National Cooperative Highway Research Program

NCHRP Report 372

Support Under Portland Cement Concrete Pavements

Transportation Research Board
National Research Council

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Report 372

Support Under Portland Cement Concrete Pavements

MICHAEL I. DARTER, KATHLEEN T. HALL, and
CHEN-MING KUO
Department of Civil Engineering
University of Illinois at Urbana-Champaign

Subject Areas

Pavement Design, Management, and Performance

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
Washington, D.C. 1995

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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

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

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

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

Note: The Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers names appear herein solely because they are considered essential to the object of this report.

NCHRP REPORT 372

Project 1-30 FY'93

ISSN 0077-5614

ISBN 0-309-05373-0

L. C. Catalog Card No. 95-60667

Price $21.00

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

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

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

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study that was performed to provide guidance for consideration of structural support in the design of portland cement concrete pavements. The report provides a comprehensive description of the research, including an examination of support concepts and causes of loss of support, a review of related data from numerous field studies, a description of a three-dimensional finite element model for the analysis of the effects of support on pavement response, and recommended revisions to the American Association of State Highway and Transportation Officials (AASHTO) *Guide for Design of Pavement Structures*. The contents of this report will be of immediate interest to pavement engineers, researchers, and others concerned with the design, construction, and rehabilitation of concrete pavements.

There is no general agreement among pavement engineers on how to select values for the modulus of subgrade reaction and the loss of support due to erosion, curling, and warping for use in design of rigid pavements and rigid pavement overlays. A perceived lack of adequate guidance in the AASHTO *Guide for Design of Pavement Structures* has resulted in inconsistent design practices for concrete pavements. Therefore, guidelines are needed to ensure proper consideration of support in pavement design.

Under NCHRP Project 1-30, "Support Under Portland Cement Concrete Pavement," the University of Illinois at Urbana-Champaign was assigned the task of developing guidelines for improved consideration of support in rigid pavement design. To accomplish this objective, the researchers examined support concepts and causes of loss of support, reviewed related data from numerous field studies, adapted a three-dimensional finite element model to analyze the effects of support on pavement response to traffic and climatic conditions, and proposed guidelines for improving the consideration of support in the AASHTO design procedure for concrete pavements. This report documents the work performed under Project 1-30 and discusses the validation and the finite element model analysis performed in preparing the proposed guidelines.

The proposed guidelines, summarized in this report, represent comprehensive revisions to the following portions of the AASHTO *Guide for Design of Pavement Structures*: Section 3.2, "Rigid Pavement Design," and Section 3.3, "Rigid Pavement Joint Design," of Chapter 3, Highway Pavement Structural Design, of Part II, Pavement Design Procedures for New Construction and Reconstruction; Appendix I, Rigid Pavement Design Example; and Appendix HH, Development of Effective Roadbed Soil Moduli. The revisions proposed by the University of Illinois research team will be considered by AASHTO's Joint Task Force on Pavements for inclusion in the AASHTO *Guide* during 1995.

CONTENTS

ACKNOWLEDGMENTS

This research was conducted under NCHRP Project 1-30 by the Civil Engineering Department of the University of Illinois at Urbana-Champaign, under subcontract to the Illinois Universities Transporation Research Consortium.

The authors extend their grateful appreciation to Professors Barry J. Dempsey, Samuel H. Carpenter, and Marshall R. Thompson of the University of Illinois at Urbana-Champaign for their enthusiastic support and many helpful comments and suggestions.

SUPPORT UNDER PORTLAND CEMENT CONCRETE PAVEMENTS

SUMMARY

The support that the base and embankment or roadbed subgrade provide to a Portland Cement Concrete pavement was found to have a significant effect on the performance of the pavement. The findings of this study relate to the appropriate *k*-value for design, the effect of support on performance, and improved consideration of support in the current empirical AASHTO design methodology and in mechanistic design.

The *k*-value, as measured on top of the finished embankment upon which the base and slab will eventually be constructed, is considered most appropriate for design. The concept of a composite "top-of-the-base" *k*-value is not valid (unrealistically high) and is not recommended for design. The *k*-value test recommended is the "elastic" *k*-value as conducted extensively at the AASHO Road Test and before that at the Arlington Road Test. When the elastic *k*-value was used in a three-dimensional (3-D) finite element pavement model, the computed slab stresses were very close to those computed from strains measured at the AASHO Road Test under creep speed loadings. An elastic *k*-value can be determined from test results obtained from standard plate bearing tests specified by ASTM, AASHTO, or Corps of Engineers.

Guidelines are provided for three categories of methods for determining a *k*-value for use in concrete pavement design: 1) correlation methods (correlation of elastic *k* with soil types, properties, and tests); 2) deflection testing and backcalculation methods (which represents the most rapid, cost-effective, and reliable method to obtain an adequate sample size of *k*-values for pavement design); and 3) plate testing methods (ASTM, AASHTO, and Corps of Engineers nonrepetitive and repetitive plate loading test methods with recommended *k*-value calculation procedures).

Practical guidelines were developed for estimating the *k*-value for situations where the existing subgrade is a very poor soil and an embankment layer of improved soil is placed on top. Guidelines are also provided for increasing an estimated *k*-value (i.e., determined from soil property correlations) when there is a stiff layer (such as bedrock) close to the surface. Seasonal *k*-value support can be estimated through knowledge of the variation in moisture level and frost depth throughout the year.

Several major deficiencies related to concrete pavement support were found to exist in the current version of the AASHTO design procedure for concrete pavements. These deficiencies are summarized as follows.

• **Use of the gross k-value measured on top of the base.** In this study, the elastic k-value measured on the subgrade was found to be the appropriate input for design. The gross k-value is unrealistically low for design.

• **Use of the lowest gross k-value measured in springtime instead of a seasonally adjusted k-value.** The concept of a seasonally adjusted k-value—incorporated in the AASHTO design procedure in the 1986 *Guide*—is a meaningful way to consider the relative amounts of damage done in different seasons, but is incompatible with the springtime gross k-value with which the current AASHTO design equation was derived. In this study, a seasonally adjusted k-value for the AASHO Road Test site was determined for use in deriving a proposed new rigid pavement design equation.

• **Consideration of the base in terms of a composite k-value rather than a more realistic consideration of the base's effect on slab stress and performance**. In this study, the base layer was modelled as a structural layer of the pavement structure using a 3-D finite element model. The modulus of elasticity, thickness, and coefficient of friction of the base course are important inputs.

• **Use of a loss of support adjustment factor to reduce k.** Substantial loss of support occurred at the AASHO site, which led to increased slab cracking and loss of serviceability; thus, the performance data already represent considerable loss of support. No additional loss of support adjustments are included in the proposed revisions to the design procedure. However, transverse joint faulting (from erosion) is predicted and the joint design adjusted if necessary to prevent faulting.

• **Use of Spangler's corner equation.** This equation when used with dowelled joints did not model the critical stress and crack initiation location, and thus could not possibly provide accurate indications of the effect of slab support on cracking, especially when thermal curling and moisture warping are considered. The 3-D finite element model was used to compute critical stresses at the midslab location that considers slab thermal gradients, base stiffness and thickness, coefficient of friction, elastic k-value, slab modulus and strength, and joint spacing. Also, a design check for critical stress is provided for undowelled pavements at the joint (corner) loading position.

• **No guidelines to design a pavement with undowelled joints.** The J-factor in the current AASHTO design procedure only considers tensile stress that controls cracking, not faulting. In the proposed revisions, design checks are provided for undowelled joints for both joint faulting and slab cracking (corner/joint load position). A check is also provided for dowelled joints to ensure their adequacy.

• **No consideration of joint spacing other than that at the AASHO Road Test (15 ft [4.6 m]).** Joint spacing is known to have a major effect on slab cracking and faulting. Slab support is a very important variable in the new procedure in the selection of joint spacing to minimize transverse cracking.

• **No representation of climates other than that of the Road Test site.** Thus, other climates that result, for example, in different slab curling (temperature differential from top to bottom of slab) or warping (moisture gradient) cannot be considered in the current AASHTO procedure. This limitation alone has led to many pavement failures from premature cracking. In the proposed revision, temperature differentials were incorporated into the slab thickness design procedure: positive for the midslab location (i.e., warmer on top than bottom) and negative for the corner/joint loading position that are project site specific. Moisture gradients are considered through an equivalent negative temperature gradient at the joint (corner) load position.

• **No consideration of the effect of faulting on performance,** because faulting of transverse joints did not occur at the Road Test, which demonstrates that even with extensive erosion and pumping, faulting can be controlled through properly sized dowel bars. Contrary to popular belief, the J-factor reflects the effect of load transfer on tensile stress in the slab under corner loading; it does not reflect the effect of load transfer on faulting at the joint. Increasing slab thickness in an attempt to reduce faulting has been shown to be an exercise in futility. In the proposed revision to the design procedure, joint faulting is predicted for the selected design and then checked against the maximum allowable. If predicted faulting exceeds the allowable faulting, a redesign of the joint load transfer, base, subdrainage, or other features is required, not an increase in slab thickness.

These deficiencies were addressed in this study, and an improved methodology was developed for better consideration of slab support. Proposed revisions to the AASHTO design procedure for concrete pavements were developed. These revisions were necessary given the deficiencies in consideration of pavement support.

A 3-D finite element model for concrete pavements (3DPAVE) was developed in this study in order to analyze the many complex and interacting factors which influence the support provided to a concrete pavement. The 3-D model was validated by comparison with deflections and strains measured under traffic loadings and temperature differentials. In every comparison with measured field data, 3DPAVE's calculated responses were found to be in very good agreement with the measured responses.

"Loss of support" refers to any gap or void that may occur between the base and the slab, or between a stabilized base and the subgrade, causing increased deflection of the slab surface. There are three basic types of loss of support that a concrete slab exhibits over time.

- Erosion of the base or subgrade or both from beneath the slab, resulting in increased deflections (faulting) and stresses (cracking) in the slab.
- Settlement or consolidation of the base or subgrade or both, usually resulting in slab cracking in the vicinity of the settlement.
- Temperature curling and moisture warping of the slab, resulting in increased deflections and stresses in the slab. Permanent construction curling presents a potential for very serious loss of support and early failure of jointed concrete pavements.

Loss of support can have a major impact on slab deflections and stresses, and thus pavement life. Temperature curling and moisture warping can be reasonably considered in the design process. Loss of support from erosion of the base or subgrade cannot be predicted at the present time for any given design project. This is currently an obstacle to predicting pavement life as a function of progressive loss of support and increasing slab stresses in a mechanistic design procedure.

In the AASHTO empirical design methodology, the critical stress computed for a fully supported slab is calibrated to the performance of the concrete slabs with granular bases at the AASHO Road Test, which were initially fully supported when constructed but experienced substantial loss of support over time because of erosion. This is true of the existing AASHTO design equation, in which the critical stress is calculated from Spangler's corner equation. This is also true of the proposed revision to the design equation, in which the critical stress is calculated as a function not only of slab thickness, slab modulus, and subgrade k, but also of base thickness and modulus, slab/base friction, temperature and moisture gradients, and joint spacing.

One would expect that the performance of a pavement with a dense treated (erosion resistant) or open-graded base would be different than the performance of the same pavement with a dense graded (low permeability) granular base. The proposed revision to the AASHTO design procedure accounts for the effect of base type on performance in two ways: first, base modulus and friction coefficient are considered in the design equation for computing slab stress, and second, base type is a factor in predicting faulting.

Specification of adequate material properties to limit the potential for erosion is also recommended and guidelines are provided.

The degree of friction between the slab and base has a significant effect on the critical stress in the slab, especially for higher-strength bases. Little is known at this time about the degrees of friction that various base types exhibit under wheel loads and how the friction between the slab and base may change over time. Tentative recommended friction coefficients are provided based on tests in which slabs were pushed over base surfaces.

In the proposed design procedure, the critical stress in the slab is computed from a closed form equation (to replace the Spangler equation) developed from the results of many 3-D finite element runs. The critical stress is a function of several important design variables.

Improved support is very important to performance as documented from several studies (reduced roughness, faulting, and cracking). However, greatly increasing support with a very stiff base and or very stiff embankment may not necessarily improve performance or cost-effectiveness of design. Under these conditions, it may be necessary to shorten the joint spacing to avoid premature transverse cracks in the slab. This determination is facilitated by the proposed design equation, which has joint spacing as an input to the stress calculation.

Analyses showed that two different loading positions could produce critical tensile stresses for a given pavement: midslab and joint. This is not a new finding as there have been some well-documented research efforts that have shown that both of these locations could be a critical condition under various design and climatic conditions.

Joint (corner) loading may be critical for undowelled or inadequately dowelled pavements, especially in combination with curling or warping. Some undowelled pavements that had substantial permanent construction curling have experienced many broken corners, diagonal cracks, and even transverse cracks after just a few months or years. The critical load position for a given pavement geometry and design may be identified through finite element modelling and the stresses checked using mechanistic design procedures.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT AND RESEARCH OBJECTIVE

Support under concrete pavements plays a very important role in performance. Slab support depends upon several variables, including the following:

- Stiffness of underlying layers (resistance to deflection under load);
- Uniformity of support along the pavement over time and climate (including lack of localized settlements and heaves);
- Friction between the slab and the base layer;
- Drainability of the pavement structure and subgrade;
- Erosion of the base or subgrade (causing loss of support over time at edges and corners); and
- Temperature curling and moisture warping of the concrete slab.

The support that a foundation (including the subgrade, embankment, and constructed base layers) provides to the concrete slab influences the magnitude of deflections and stresses in the slab induced by traffic loads and environmental forces. These critical stresses and deflections produce two of the major distresses (slab cracking and joint faulting), which greatly affect concrete pavement performance. Another major distress, joint spalling, can be caused in part by concrete durability problems that depend in part on the degree of saturation of the concrete slab, which is affected by base and subgrade permeability.

In the American Association of State Highway and Transportation Officials (AASHTO) *Guide for Design of Pavement Structures* (hereafter referred to as the AASHTO *Guide*), and in all other accepted concrete pavement design procedures, the support a foundation provides a concrete pavement is characterized by a *k*-value, which represents a "dense liquid" (elastic springs) foundation. The AASHTO *Guide* also has a "loss of support" input that is intended to represent the potential for reduction of support at slab corners over the design life of the pavement, and also an empirical factor for drainability of the base. It is very difficult to select appropriate values for these three inputs, which are truly representative of the support the concrete slab will experience over its design life, and which are truly representative of the effect of support on the performance of the concrete pavement.

Design Subgrade *k*-Value

The rigid pavement thickness design equation used in the AASHTO *Guide* was developed from the results of the American Association of State Highway Officials (AASHO) Road Test, at which plate bearing tests were conducted to determine "elastic" *k*-values and "gross" *k*-values (computed from elastic plus substantial permanent deformation). The springtime gross *k*-value was incorporated into the development of the AASHTO rigid pavement design equation. However, previous research and this study have shown that it is the elastic *k*-value that produces calculated values of slab stress and deflection that agree well with those measured in the field.

Conventional plate bearing tests were often conducted to determine subgrade *k*-values through the 1950s, and even up to the 1980s by some agencies, particularly for airfields. However, these tests are extremely expensive and time-consuming, and thus are rarely conducted today.

Other approaches exist for estimating *k*-values for design, including correlation with soil properties and other soil tests, and backcalculation from deflection testing on concrete pavements. These different approaches to selecting *k*-values frequently give different results. There is no general agreement among engineers as to which of these methods is most appropriate as well as most practical.

Since the 1960s, *k*-values intended to represent plate bearing test values estimated on top of the base have commonly been used in concrete pavement design. These "top-of-base" or "composite" *k*-values overestimate the support the slab actually experiences in the field. This is particularly true for treated base layers or existing concrete or asphalt pavement structures. The top-of-base *k*-value that the AASHTO design procedure and other current concrete pavement design procedures would assign to such a stiff base are totally unrealistic, as Figure 1 illustrates.

The composite *k* concept also does not realistically reflect the effect that a base layer has on stress in a concrete slab due to load, friction with base, temperature, and moisture influences. A more realistic approach to characterizing concrete pavement support would assign elastic *k*-values to the subgrade or embankment and consider the base course as a structural layer.

Loss of Support

The loss of support (LOS) factor was introduced in the 1986 AASHTO *Guide* to attempt to account for reduced slab support due to base erosion. The LOS factor applies a drastic reduction to the composite *k*-value if the pavement being designed will be built on an erodible base. However, it is well-documented that the concrete pavements at the AASHO Road Test, which were built on dense graded granular bases, developed extensive pumping. Thus, the existing rigid pavement design equation already includes the effect of a significant amount of loss of support on

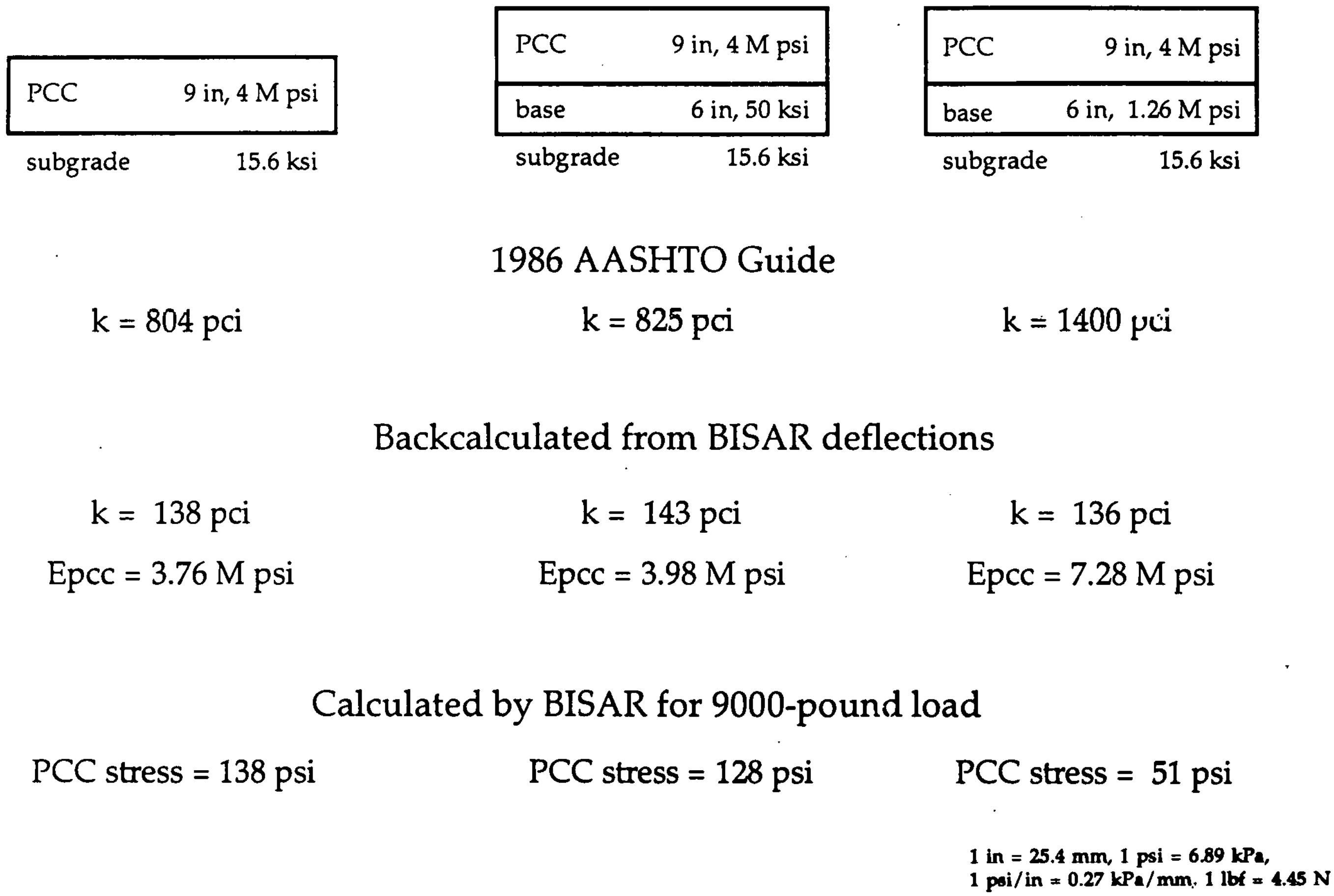


Figure 1. Examples of subgrade and base effects on k-value and slab response.

slab stress and performance. One of the issues investigated in this study was whether the *k*-value reduction applied by the AASHTO *Guide's* LOS factor result in unnecessary increases in designed slab thicknesses for granular bases.

Influence of Support on Performance

The stresses and deflections that affect performance in a concrete slab depend on several support factors, including the following:

1. Subgrade soil stiffness;
2. Base type, stiffness, and thickness;
3. Frictional resistance between the slab and base;
4. Seasonal moisture levels in the subgrade and untreated base;
5. Seasonal freezing and thawing in the base and subgrade;
6. Load transfer at joints and cracks (effects erosion);
7. Erosion of base or subgrade material over time due to traffic action, poor drainage, or foundation movement; and
8. Temperature and moisture gradients in the slab.

All of these factors interact to influence the magnitude of stresses and deflections experienced in concrete slabs under traffic loads and in response to environmental forces (temperature cycles, moisture cycles, and freeze-thaw cycles). Repeated stresses and deflections at critical slab edge and corner locations may produce transverse cracking, longitudinal cracking, corner breaks, and faulting. Concrete pavement performance is measured by these distresses (plus joint spalling), either directly (i.e., distress severity and quantity) or indirectly (i.e., roughness, which in concrete pavements is primarily a function of cracking, faulting, and spalling).

Most of these support factors are either not considered at all or considered in an unrealistic manner in current concrete pavement design procedures. Improved methods for characterizing concrete pavement support parameters and their effects on performance are needed to improve design.

Research Objectives

This study addressed the urgent need to produce practical guidelines for selection of appropriate *k*-values, consideration of loss of support over time, and consideration of other support factors for use in design of concrete pavements and overlays. NCHRP Project 1-30 had the following specific objectives: (1) to develop and recommend improved guidelines for the selection of *k*-values and (2) to identify and assess loss-of-support values for use in the design of rigid pavements and pavement overlays.

SCOPE OF STUDY

This study addressed concrete pavement support characterization for two purposes: improvement of the guidelines for consideration of support parameters in the current AASHTO *Guide* design methodology and development of improved methods for characterizing support in a mechanistic design methodology.

Substantial efforts were expended in the development of improved methods for considering support for concrete pavements in the AASHTO *Guide*. The products of these efforts include detailed guidelines for selection of the design subgrade *k*-value, an equation for slab stress due to wheel loads and climatic influences with a realistic modelling of the base course, and a proposed revision to the AASHTO design equation, which is compatible with the *k*-value guidelines and incorporates the new slab stress equation.

The second focus of this study was development of improved methods for characterizing concrete pavement support in a mechanistic design methodology. Recommendations were developed for characterizing support and considering the influence of support on slab stress and performance in mechanistic design.

RESEARCH APPROACH

The research objectives were accomplished through intensive examination of the support concepts, which have evolved over the last 100 years; use of available test data from many past field studies; development and extensive use of a 3-D finite element model for multilayered jointed concrete pavement; analysis of the effects of support on slab response to traffic loading and climatic influences; and analysis of the effects of support on concrete pavement performance.

Three categories of procedures for estimating the *k*-value were documented and evaluated. These include (1) correlations with soil type, soil properties, and soil tests; (2) backcalculation from nondestructive deflection test measurements; and (3) plate bearing test methods.

A review was conducted of the causes of loss of support, including temperature curling, moisture warping, and erosion of supporting material. Recommendations were developed for consideration of the various aspects of loss of support in design.

A detailed and through review of the AASHO Road Test concrete pavement analyses was conducted, and many interesting discoveries made regarding the extension of the original rigid pavement equation. These discoveries led to the necessity of modifying the rigid pavement design equation to more realistically account for the effects of the subgrade and base layers on pavement performance.

A 3-D finite element model for investigating the effect of support on slab stress and deformation response was developed using the ABAQUS general-purpose finite element software package. The model was extensively tested and validated using measured strain and deflection data from the AASHO Road Test and other experimental studies. Based on these results, procedures for improved consideration of support in the current AASHTO methodology were developed.

A framework and tools for considering support in a mechanistic-empirical methodology were developed. This framework includes the 3DPAVE model to realistically model stresses in the concrete slab for any type of loading, slab dimensions, thermal and moisture gradients, dowelled or undowelled joints, a treated or untreated base layer with a realistic friction coefficient between the slab and base, the stiffness of the subgrade, and related design features such as joint spacing and load transfer.

ORGANIZATION OF REPORT

Chapters 2 through 4 discuss the findings, interpretation, and conclusions of this study. The report appendixes as submitted by the research agency are not published herein but are available for loan on request to the NCHRP (see page 50).

CHAPTER 2

FINDINGS

The key findings of this study are summarized in this chapter. These findings relate to the evolution of the *k*-value concept, recommended methods for determining *k*-value, the three-dimensional finite element model used to characterize concrete pavements, methods for assessing support, and recommendations for improved consideration of support in the AASHTO *Guide* and in mechanistic design.

EVOLUTION OF THE *k*-VALUE

A detailed review of the evolution of the *k*-value concept is provided in Appendix A. The main findings of this review are summarized in this section.

Introduction of Dense Liquid Support Model

The conceptual model of a plate supported by a "dense liquid" foundation is attributed to Winkler (*1*) in 1867, although it may have been suggested earlier. Such a foundation is assumed to deflect under an applied vertical force in direct proportion to the force, without shear transmission to adjacent areas of the foundation not under the loaded area. Applications of this concept to plates such as ice sheets and concrete slabs were proposed by Hertz (*2*) in 1884; Föppl (*3*) in 1907; and Koch (*4*), Schleicher (*5*), and Westergaard (*6*) in 1925.

The dense liquid model represents one end of the spectrum of elastic soil response. At the other end of the spectrum is the elastic solid model, according to which a load applied to the surface of a foundation is assumed to produce a continuous and infinite deflection basin. The elastic response of real soils (i.e., unbound sands, silts, and clays) lies somewhere between these two extremes, as illustrated in Figure 2. The dense liquid model has traditionally been favored over the elastic solid model in concrete pavement analysis. One reason for this is that the dense liquid model simplifies slab stress and deflection calculations. In the era of high-speed computers, this is a less significant concern than it was in the past. Another reason the dense liquid model is preferred for concrete pavement analysis is that, for slabs on natural soil subgrades or granular bases, the dense liquid model more accurately predicts slab responses at edges and corners.

In addition to elastic behavior, real soils exhibit irreversible and time-dependent behavior. The latter is a concern for concrete pavement design, particularly for cohesive saturated soils. The *k*-value of these soils may be substantially higher under rapid loading (e.g., moving vehicles or impulse loads) than under slow loading, because under rapid loading pore water pressures are not dissipated (*7*). However, the available concrete pavement performance models are based on *k*-values determined from static load tests, while the actual loads applied by traffic are usually dynamic, as are the loads applied by deflection testing devices used for evaluation of in-service pavements and foundations.

Under slow loading, primary consolidation occurs gradually as pore water pressures dissipate, until in most cases the deformation of the soil reaches some stable value (*7*). However, it is possible for soils to exhibit secondary (creep) deformation, if the magnitude of load exceeds the creep strength of the soil (*7,8*). In such a case the deformation may not reach a stable value, even in a load test which is allowed to continue for several hours. The consolidation and creep responses of soils to slow loading necessitate some standardization in soil load test methods. For example, a typical load test procedure may specify that the test may be stopped when the rate of change of deformation has slowed to some given value.

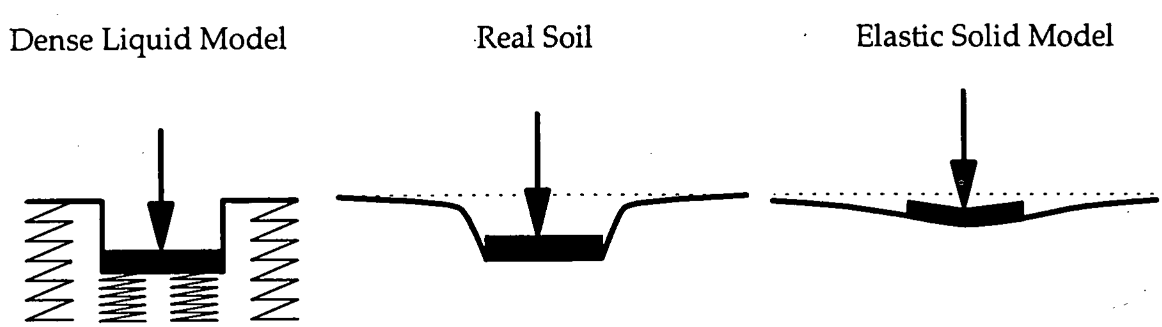


Figure 2. Dense liquid and elastic solid extremes of elastic soil response.

Westergaard's Equations for a Slab on a Dense Liquid

In 1925, Westergaard presented equations for deflection of a concrete slab on a dense liquid foundation, for interior, edge, and corner loading conditions (*6*). Westergaard introduced the term "modulus of subgrade reaction" for the *k*-value, and also introduced the term "radius of relative stiffness," ℓ, to describe the stiffness of a concrete slab relative to that of the subgrade. Westergaard suggested that the subgrade *k*-value could be backcalculated from deflections of the slab surface rather than from load tests on the subgrade (*6,9*). Westergaard envisioned his equations as being useful for evaluating existing pavements in order to gain insight for design of new pavements. Westergaard did not, however, explicitly address determination of subgrade *k*-values for use in new design. Thus, he did not shed light on the problem of reconciling *k*-values from plate load tests with *k*-values backcalculated from pavement deflections.

Westergaard also identified some of the major concerns in characterizing subgrade support which still persist, namely: slab curling and warping, nonuniformity of support, friction at the slab/foundation interface, and dynamic versus static loading, and encouraged further analysis of all of these influences. Westergaard suggested that the dynamic loading response "may possibly be expressed approximately in terms of an increased value of the modulus *k*" (*9*).

Arlington Road Tests

In the early 1930s, the Bureau of Public Roads conducted extensive field tests at the Arlington Experiment Farm in Virginia to investigate several aspects of concrete pavement behavior. These investigations were documented in a series of reports by Teller and Sutherland (*10,11,12,13,14*). One of the objectives of these field tests was to verify Westergaard's equations. Among the many valuable findings of the Arlington tests are those concerning measurement of subgrade *k*-values, effects of seasonal moisture variation on *k*-values, effects of slab curling on corner *k*-values, and effect of subgrade "improvement" on *k*-values.

In order to verify Westergaard's equations with experimental results, it was necessary for the Arlington researchers to develop some way to determine the subgrade *k*-value. Rigid plate testing and full-size top of slab testing were employed to investigate necessary test procedures, which would produce compatible results from the two methods. Several series of soil tests were run with a range of plate diameters and loads. Each load was applied and released several times in order to "reach a condition such that each succeeding application of a given load would produce the same vertical displacement of the bearing plate. This might be termed a state of approximate elastic equilibrium" (*14*).

Figure 3 is an example load-displacement plot for the repetitive loading procedure used in the Arlington tests. The *k*-value was determined by dividing the plate pressure at a given load level by the elastic deflection at equilibrium. The load-deflection tests clearly showed the effects of plate size and displacement magnitude on *k*. Teller and Sutherland concluded that plate tests for *k* should be conducted with rigid plates of fairly large diameter, and maximum displacements limited to the range which a concrete pavement might be expected to experience under traffic.

Deflection testing was also conducted on top of concrete slabs in the Arlington tests to experimentally verify the stresses and deflections predicted by Westergaard's equations. This was the first backcalculation of subgrade *k*-values and slab E-values from deflections measured on top of concrete slabs under interior, edge, and corner loading conditions. Radiuses of relative stiffness values were determined by matching slab deflection basin measurements to contours drawn by Westergaard (*6*) for deflection versus distance from load. The *k*-values determined from repeated loads on a 30-in. (762-mm)-diameter plate at a deflection at 0.05 in. [1.27 mm] on the subgrade gave values that agreed well with those backcalculated from deflections induced by loads on top of concrete slabs (no base course existed on these slabs). The backcalculated concrete elastic moduli were also in good agreement with values obtained from lab tests on samples cut from the slabs.

The researchers also investigated the effect of seasonal moisture content on *k*-values measured by plate bearing tests. At each displacement magnitude, the lower moisture content in the summer corresponded to a significant increase in *k*-value.

Teller and Sutherland described some rather surprising results concerning plate load tests on modified subgrade. In one test area, the subgrade was modified by mixing several inches of sand with the silty subgrade. The *k*-value obtained from plate tests on the modified subgrade was 400 psi/in. [108 kPa/mm], whereas the *k*-value of the original subgrade had been about 280 psi/in. [76 kPa/mm] for summer conditions. After a concrete slab was constructed on the modified subgrade, the *k*-value backcalculated from slab deflections was found to be 285 psi/in. (77 kPa/mm), "considerably lower than that indicated by the bearing tests with the 36-in. (914-mm)-diameter rigid plate but essentially the same as that found for the original unmodified subgrade under similar summer conditions" (*14*). Teller and Sutherland concluded that when the load was applied over a large area (i.e., distributed by a concrete slab), "the influence of the strengthened upper layer on the load support offered by the subgrade as a whole tended to disappear."

Effect of Loads and Temperature on Total Slab Stress

A summary of the findings of the Arlington experiments was written by Kelley in 1939 (*15*). Kelley pointed out that the stresses produced in concrete slabs by the combined effects of wheel loads and temperature variation could be much greater than the stresses predicted by Westergaard's equations for wheel loads only, and that for short slabs (e.g., less than 17 ft [5.2 m] for k = 100 psi/in. [27 kPa/mm] and less than 13 ft [4 m] for k = 300 psi/in. [81 kPa/mm]), temperature curling stresses actually increase with increasing *k*-value.

These observations are reasonable if one considers that the stiffer the foundation is, the less a curled slab can settle into the foundation, thus the greater proportion of the slab area will be unsupported by the foundation, and thus the higher the slab stresses will be. Nonetheless, the idea of total slab stress increasing with *k*-value is somewhat difficult to accept if one is accustomed to thinking of *k*-value only in terms of its effect on stresses due to traffic loads.

Kelley felt that a default *k*-value of 100 psi/in. (27 kPa/mm) was a somewhat conservative value, which provided a tradeoff

1 in = 25.4 mm, 1 lb/ft^2 = 47.85 Pa

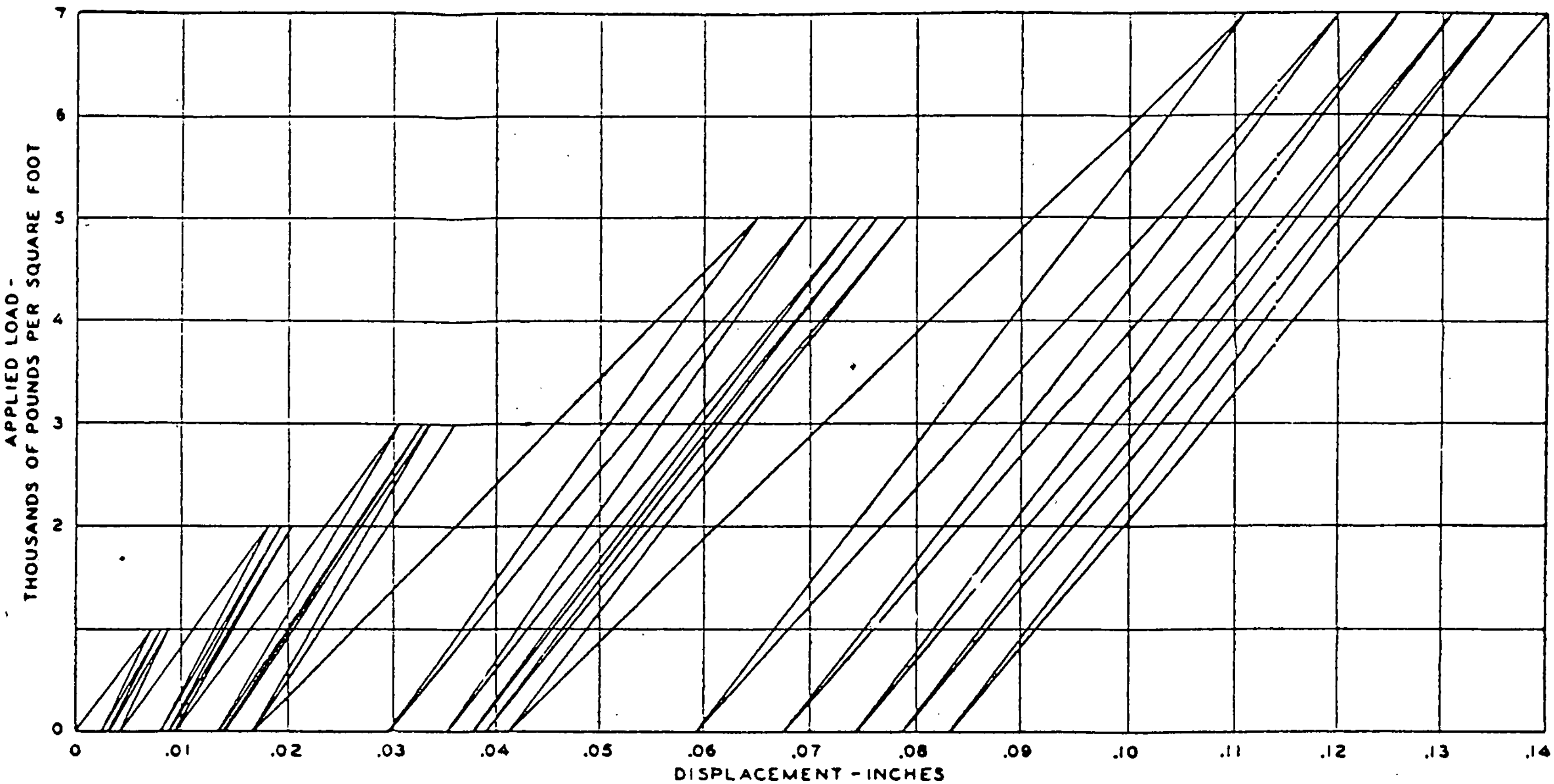


Figure 3. Typical load-displacement data from Arlington plate tests (14).

between load and curling stresses. If the actual *k*-value of a subgrade was higher than the assumed value, for example, the stress equations would overestimate the load stress and underestimate the curling stress. However, consideration of curling stresses has not been a part of concrete pavement design practice in the 50 or more years after Kelley's recommendations were published, although curling stresses certainly have been significant to the performance of concrete pavements.

Corps of Engineers Field Studies

In the 1940s, the U.S. Army Corps of Engineers conducted load tests on subgrades and concrete slabs several airfields (*16,17,18,19,20,21*). One of the objectives of the Wright Field slab tests was to develop a standard procedure for determining subgrade *k*-values (*22*). The *k*-values obtained from load tests on the subgrade with plates ranging from 12 to 72 in. (305 to 1829 mm) diameter were compared with *k*-values obtained from tests on top of slabs. The volumetric method of calculating *k*, which is explained in Appendix A, involves dividing the applied load by the volume of the deflection basin. The 30-in. (762-mm) diameter plate consistently yielded subgrade *k*-values in close agreement with the volumetric *k*-values. The 30-in. (762-mm) plate-bearing test became the Corps of Engineers' standard test method for *k*, and also became the basis for the American Society for Testing and Materials (ASTM) and AASHTO standard test methods which were developed later.

In addition, an insightful comment on the effect of a base course was made by Sale: "The only exception to this pattern [plate *k* agreeing with volumetric *k*] is the high *k*-value obtained on moderate base course thicknesses which generally must be adjusted downward to match full-size slab performance" (*22*).

Correlation of *k*-Value and CBR and Soil Classification

In 1942, Middlebrooks and Bertram (*23*) published a paper summarizing many aspects of the Corps of Engineers' subgrade studies, including perhaps the first published correlation of *k*-value to California Bearing Ratio (CBR) and to the Unified and Public Roads (now AASHTO) soil classification groups, shown in Figure 4.

An important detail of the Corps of Engineers' *k*-value test method on which these correlations are based is the selection of 0.05 in. (1.27 mm) as the deflection at which *k* is defined. The Corps' plate-bearing test procedure does not involve repeated loading and unloading, as was done at the Arlington Road Test, so it is reasonable to ask whether the *k*-value obtained from the Corps' test procedure is an "elastic" *k*, or whether it includes both elastic and plastic deformation. It is a significant question because the correlation chart shown in Figure 4 formed the basis for similar *k*-value correlation charts and tables, which were later incorporated in the U. S. Army's design manuals (*24,25*) and the Portland Cement Association's design manuals for highway and airport concrete pavements (*26,27*), and have been widely used ever since. Phillippe (*19*) and Middlebrooks and Bertram (*23*) state that the deflection value of 0.05 in. (1.27 mm) was selected because the results of many tests indicated that this deflection corresponded to *k*-values that agreed with the *k*-values obtained from deflection testing on top of full-size slabs. If one presumes that *k*-values calculated from top of slab deflections represent elastic response of the subgrade, then one may conclude that the *k*-value obtained from the Corps' definition (at a deflection of 0.05 in. [1.27 mm]) is the equivalent of an elastic *k*, and thus that the correlations shown in Figure 4 are correlations of CBR and soil classification to elastic *k*-values.

Effect of Base Layers on *k*

In the 1940s, numerous reports appeared in the literature concerning plate load tests on subgrades and on base layers (*28,29,30*). These and other studies of the time illustrate a developing trend to consider base layers as an effective means of improving subgrade *k* values, and to consider this improvement as a function of base thickness and base material.

The Corps of Engineers also apparently changed its position on the effect of base layers on *k*-value during this time. As Ahlvin describes in his report on the historical development of the Corps' pavement design procedures (*31*), airfield pavements were constructed directly on natural subgrades throughout the 1940s, but base materials came into use in the early 1950s to combat pumping. However, the Corps also began to attribute an improved *k*-value to the base. "Limited early experience," Ahlvin states, "had been interpreted to indicate that subbase or base under rigid pavement had no structural advantage," a conclusion that is consistent with Sale's description of the results of the Wright Field Tests.

In the 1950s, however, the Corps modified its design practice to require plate bearing tests on top of bases. This led eventually to development of curves for top-of-base *k*-values. Ahlvin's historical review does not mention any attempts by the Corps to validate the base *k*-value curves by deflection testing on top of concrete pavements. Had this been attempted, the results obtained earlier at the Arlington and Wright field tests would have been reaffirmed and the erroneous concept of top-of-base *k*-value might not have been perpetuated.

ASTM Plate Bearing Test Methods

The first standard ASTM test methods for plate bearing tests on soils were published in 1952. Two tests, based largely on the Corps of Engineers' procedure, were published: D 1195, Repetitive Static Plate Load Test, and D 1196, Nonrepetitive Static Plate Load Test. These two tests have changed very little since they were originally published. Details of these and other plate bearing test procedures are provided in Appendix B. Interestingly, neither of the ASTM test methods give any guidance on calculation of the subgrade *k*-value from the load and deflection data obtained. Calculation of *k*-value is covered in the Corps of Engineers' test method, and in the AASHTO test methods T221 and T222, which were not standardized until the 1960s.

AASHO Road Test

This major field test was conducted by the Highway Research Board in cooperation with AASHO between 1958 and 1960, near Ottawa, Illinois. The AASHO Road Test is documented in

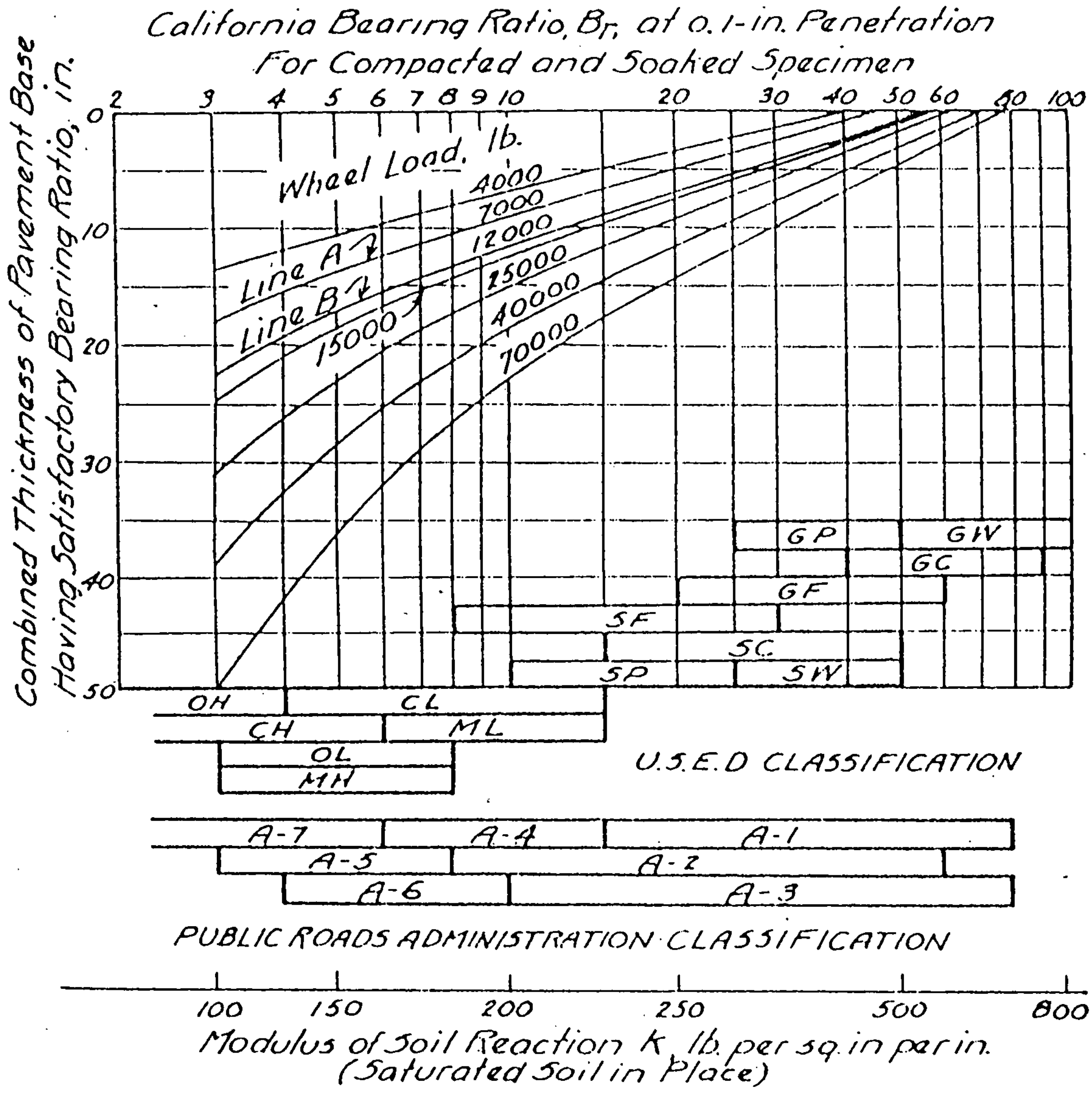


Figure 4. Correlation of k-value, CBR, and soil classification (23).

great detail in a series of Highway Research Board reports and many related documents. Detailed investigations of base and subgrade properties and their variation throughout the seasons were conducted at the road test (*32*). Plate load, CBR, moisture content, and density tests were made on the subbase and the embankment.

Trends of increasing CBR and decreasing moisture content were noticeable. Although no particular relationship between k and dry density was noted, k did show a tendency to decrease with increasing moisture content. A clearer trend of decreasing k with increasing percent saturation (which represents a combination of dry density and moisture content) was evident. The same was true for CBR versus percent saturation.

At the time of the Road Test, AASHO did not have standard test methods for plate bearing tests, and the test procedure used did not conform to the ASTM or Corps of Engineers standards. The procedure used was similar to that used at the Arlington Test (an elastic k-value), and involved three cycles of loading and unloading at each of three load levels, using a 30-in. (762-mm)-diameter plate. An average *elastic* k-value (k_E) was determined by dividing each of the individual loads by the elastic deformations they produced (not including any permanent deformation). The k_E is the value plotted in all graphs in the AASHO Road Test reports. In addition, a *gross* k-value (k_G) was determined for each load level by dividing the load by the total deformation produced, including permanent deformation. A schematic illustration of the load-deformation results obtained from the plate tests is shown in Figure 5.

The k_E-values exceeded the corresponding k_G-values by an average ratio of 1.77. Although the k_E was the main test shown in all figures and tables of the Road Test report, the springtime k_G-value of the Road Test site was used to develop the concrete

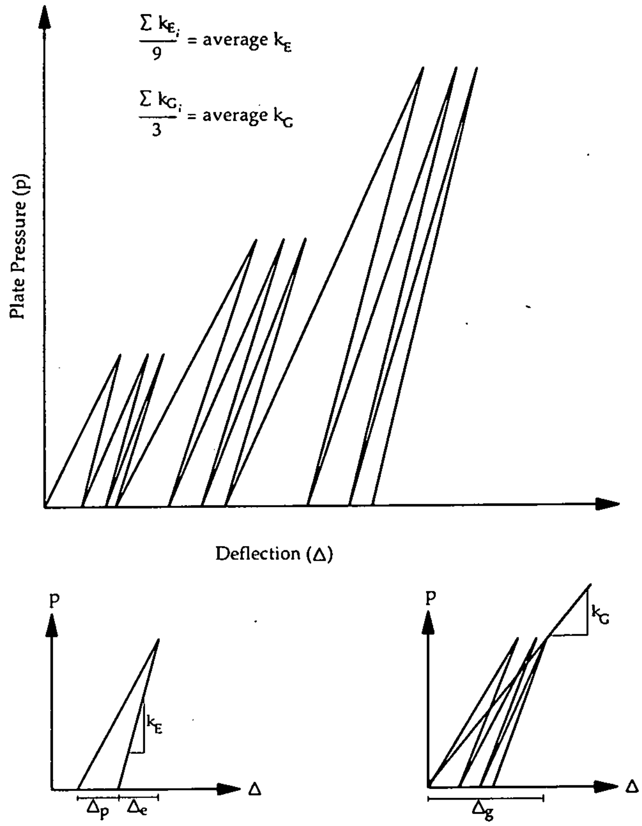


Figure 5. Schematic illustration of AASHO Road Test plate load test results.

pavement design equation although no confirmation has been found in prior literature that gross *k*-values determined in the manner employed at the AASHO Road Test were commonly used in design.

On the contrary, the elastic *k*-value was recommended by the Bureau of Public Roads after the Arlington Tests, the elastic *k*-value test was conducted at the AASHO Road Test, and the *k*-values obtained from the Corps of Engineers test method represent the elastic *k*-value. However, because the Corps of Engineers method for determining *k*-value is a nonrepetitive test, one might presume that the AASHO Road Test researchers felt that the gross *k* was more representative of the *k* obtained in a nonrepetitive test than the elastic *k*.

A *k*-value of 60 psi/in. (16 kPa/mm) was used to represent AASHO Road Test conditions in the development of the AASHO rigid pavement design equation (*33*). This value is equivalent to the mean *springtime gross k-value* from tests on top of the aggregate subbase. It was a conservative value picked to represent the Road Test conditions. The only more conservative value would have been the springtime gross *k*-value of 49 psi/in. (13 kPa/mm) on top of the subgrade. Why the subbase k_G was selected rather than the subgrade k_G is not documented.

In 1962, the Corps of Engineers conducted load tests on top of the existing slabs at the AASHO Road Test site and calculated volumetric *k*-values between 25 and 92 psi/in. (7 and 26 kPa/mm) from the slab deflection basins (*34,35,36*).

Portland Cement Association

In the 1960s, the Portland Cement Association (PCA) conducted a series of laboratory experiments with full-sized concrete slabs. These experiments included plate load tests on prepared subgrade soils, untreated gravel and crushed stone bases, cement-treated bases (CTB), and soil-cement pavements (*37,38,39*). The plate tests on top of the granular bases yielded slightly higher *k*-values than the subgrade plate tests, but the plate tests on the cement-treated bases yielded considerably higher *k*-values, which increased linearly with cement-treated base thickness. Subsequent load tests on concrete slabs constructed on the cement-treated bases showed a decrease in maximum edge and interior deflections with increasing base thickness. The PCA used these results to develop curves for top-of-base *k*-values for granular and cement-treated bases, which were incorporated in PCA's concrete pavement design procedures and published in 1966.

An intriguing aspect of the PCA's guidelines on effect of base layers on slab behavior is an alternate procedure offered for determining an adjusted ℓ-value as a function of the concrete-to-base stiffness ratio (E_1/E_2), the concrete-to-base thickness ratio (h_1/h_2), and the ℓ calculated for the concrete slab and subgrade *k* alone. According to the PCA manual's recommendation to increase *k* when a base is present, the radius of relative stiffness would decrease. The alternate procedure suggests that the opposite result, an increase in ℓ, may be more realistic when a strong base is used (*27*). The PCA manual indicates that the two approaches to defining ℓ (combining the base with the *k* versus combining the base with the slab), although diametrically opposed in concept, produce reasonably similar results for thinner pavements, weak bases, and small load sizes. However, the latter approach was considered more appropriate for thicker pavements, stiffer bases, and larger load sizes (i.e., multiwheel aircraft gear configurations).

The load tests run on concrete slabs in the PCA experiments showed decreases in deflection with increasing thickness of cement-treated base, which was interpreted as being the result of an increase in the *k*-value. These experimental results would then appear to be contrary to the statements quoted above concerning the effect of the base on increasing ℓ. However, the PCA studies did not report that any *k*-values were backcalculated from the slab deflections to determine whether the top-of-base *k*-values were confirmed. This was done for this study using the deflection data reported by PCA. The *k*-values backcalculated from the slab deflections are much closer to those obtained from plate tests on the subgrade than to those obtained from plate tests on the CTB. This does not mean, however that a cement-treated base has no effect on stresses in the concrete slab. A cement-treated base may significantly reduce stress at the bottom of the concrete slab, especially if normal frictional resistance exists between the slab and base.

1972 AASHTO Interim Guide

In the evolution of the AASHTO rigid pavement design methodology following the AASHO Road Test, a series of modifications was made to the process of selecting a *k*-value for design. The 1972 AASHTO Interim Guide specified the use of the subgrade gross *k*-value (measured on top of the aggregate subbase) in the main section of the manual (*40*). An alternate procedure to determine the design k-value, termed a composite k-value on top of the subbase, was also given. According to this procedure, the subbase stiffness and the modulus of subgrade reaction are used in a nomograph, developed using elastic layer theory, to determine the composite k-value on top of the subbase. This seems to be a discrepancy, because the AASHO Road Test's top of granular subbase k-value (k_G = 60 psi/in. [16 kPa/mm]) was already incorporated in the rigid pavement design equation.

The 1972 *Guide* suggested that an adjustment to the k-value might be warranted to reflect loss of support. This too appears to be a discrepancy, because loss of support was already incorporated into the AASHTO model: the rigid pavement design equation was developed from the performance data for the AASHO Road Test's concrete pavement sections with granular bases, and these pavements experienced substantial pumping and loss of support beneath the slabs (*32*).

Correlation of k to Soil Type and Degree of Saturation

The concrete pavement design procedure developed in the 1977 Zero-Maintenance study (*41,42*) recommended that a soil's k-value in various seasons be determined from its AASHTO classification and the degree of saturation in the upper 1 to 5 ft (0.3 to 1.5 m) of soil. The k-values were obtained using correlations between resilient modulus, static elastic modulus, and degree of saturation developed from an extensive field and laboratory study of Illinois soils (*43*).

1986 AASHTO *Guide* k-Value Methods

Five modifications to the 1972 Interim Guide's k-value guidelines were introduced in the 1986 version of the AASHTO *Guide* (*44*). Each of these modifications has been examined in great detail in this study (see Appendix A), and the following conclusions reached:

1. **Equation for k-value for an unprotected subgrade:** The equation ($k = M_R/19.4$, where laboratory resilient modulus M_R is assumed in the *Guide* to be equal to the in-place elastic modulus E of the subgrade), developed from elastic layer simulation of plate testing on an elastic half-space, produces unrealistically high k-values.
2. **Nomograph for composite (top-of-base) k:** This nomograph, also developed from elastic layer simulation of plate tests on base/subgrade combinations, produces unrealistically high k-values.
3. **Adjustment to k for rigid foundation within 10 ft (3 m) depth:** The basis for this nomograph is not documented, although it is presumed to have been developed using elastic layer simulation in a manner similar to the development of the composite k nomograph. Analyses have been conducted to replace this nomograph with one that would adjust k not only for a rigid layer beneath the subgrade but also for significant thicknesses of improved embankment material (e.g., more than 1 ft [0.3 m] placed above the subgrade).
4. **Seasonal adjustment procedure for k:** The AASHTO *Guide* provides a method for determining a design k-value which represents the range of k-values expected in various seasons, weighted with respect to the relative damage done to the pavement in those seasons. The relative damage is calculated using the AASHTO rigid pavement design equation. This damage-weighted seasonal adjustment seems reasonable in concept, except that it suggests that relative damage is sensitive to slab thickness, and close examination of the nomograph and equations reveals that thickness has little or no effect. An inconsistency of the seasonal adjustment procedure is that the design equation itself is not calibrated to a seasonal average k for the AASHO Road Test site, but rather the springtime lowest of the year k_G value.
5. **Loss of support adjustment to the k-value:** This nomograph, developed using discrete element analysis of various sizes of voids under a concrete pavement joint, produces dramatic reductions in k-value for erodible bases. This loss of support adjustment is a major discrepancy in the design procedure, as mentioned before, because the performance prediction model is based on the AASHO Road Test pavements, which had granular bases and experienced substantial loss of support.

Development of k-Value Backcalculation Methods

The concept of characterizing a deflection basin by its maximum deflection and its AREA (cross-sectional area computed from deflections at 0, 12, 24, and 36 in. [0, 305, 610, and 914 mm], normalized to the maximum deflection) was proposed by Hoffman and Thompson in 1981 for flexible pavements (*45*). This concept was subsequently applied to backcalculation of PCC slab elastic modulus values and subgrade k-values for many airport and highway projects (*46*). The ILLI-SLAB finite element program was used to compute a matrix of maximum deflections and AREA solutions by varying the k-value and E for a given slab thickness and slab size. A family of curves was then plotted against AREA and d_0 axes. Individual midslab deflection basins (AREA and d_0) measured with a Falling Weight Deflectometer (FWD) could then be plotted on the matrix, and the slab E and foundation k-value interpolated. In 1985, Foxworthy adapted this backcalculation scheme to a computerized solution (*47*).

The k-values obtained from the FWD were typically about twice as high as the static k-values, which would be expected for the same soils in standard plate bearing tests. Foxworthy's research included FWD deflection testing at several U.S. Air Force bases, and comparison of the backcalculation results obtained with results of plate load tests and laboratory tests of concrete samples. Foxworthy observed that k-values backcalculated from deflections exceeded k-values from plate load tests by a mean ratio of 2.7 (ranging from 1.6 to 4.4).

Further investigation of the AREA concept by Barenberg and Petros (*48*) and by Ioannides (*49*) produced a forward solution procedure to replace the iterative and graphical procedures used previously. This solution is based on the fact that a unique relationship exists between AREA, defined for a given load radius and sensor arrangement, and the dense liquid radius of relative stiffness (ℓ_k) of the pavement system, in which the subgrade is characterized by a k-value. In 1989, Ioannides, Barenberg, and Lary (*50*) demonstrated the application of this closed-form approach with the ILLI-BACK program, which was developed to permit rapid analysis of deflection basins. This

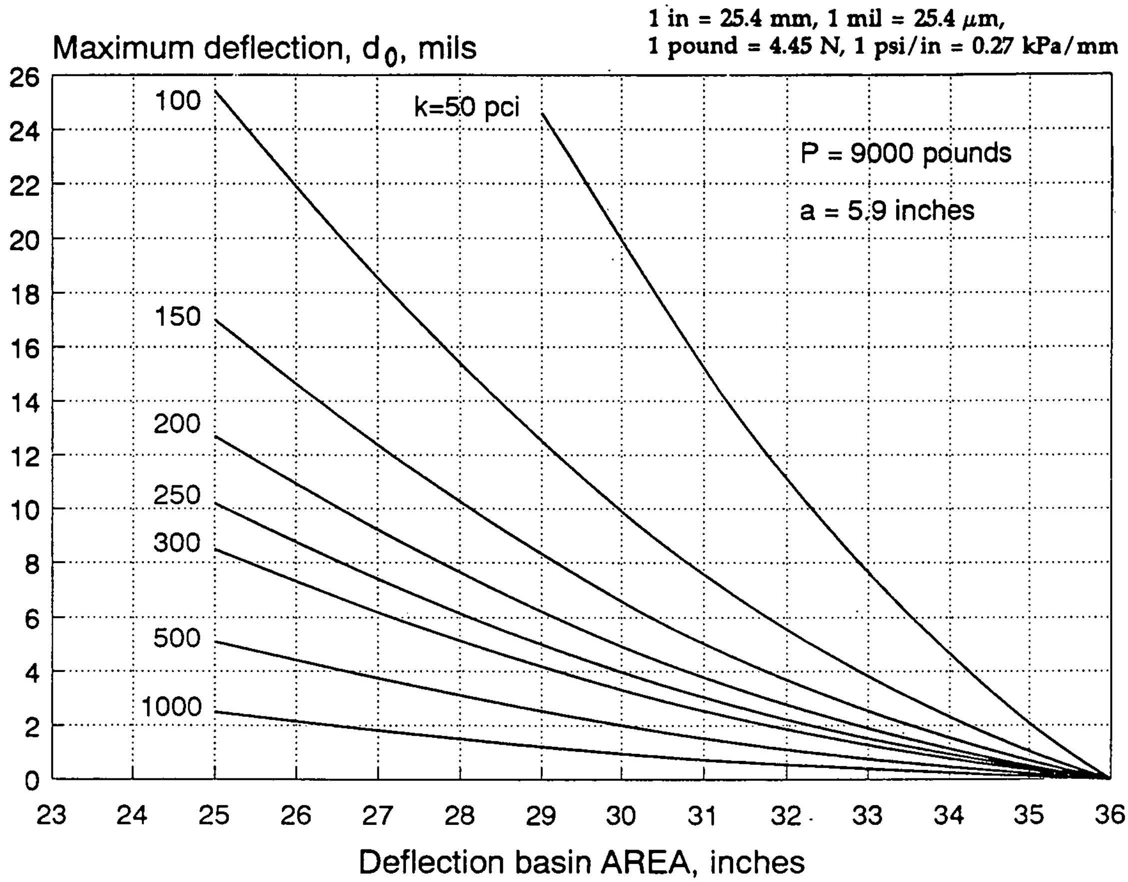


Figure 6. Backcalculated k-value determination from d_0 and AREA (52).

solution method is much faster than the graphical methods used before, but it does have a drawback: the solution for k is based on Westergaard's equations for deflection of an infinite slab. The deflection of an actual highway pavement slab (12 ft [3.7 m] wide, perhaps only 15 ft [4.6 m] long) may be quite different from that predicted from theory for infinite slabs. Also, as with the ILLI-SLAB solution, the k-values backcalculated from FWD data are higher than static k-values. The backcalculated concrete modulus will also be higher than the actual concrete modulus if a treated base is present.

Nomographs and equations for backcalculation of concrete elastic moduli and subgrade k-values (Figure 6) and concrete E values (Figure 7) for concrete or composite pavements were developed by Hall (*51*) and incorporated into the 1993 AASHTO *Guide* (*52*). This solution method is also based on deflection of an infinite slab. For composite (asphalt concrete [AC]-overlaid PCC) pavements, an adjustment must be applied to the d_0, the deflection under the load plate, to account for compression in the AC layer. The equations, which may be used in a spreadsheet, permit rapid determination of subgrade k- and slab E values. The 1993 *Guide* recommended that the dynamic load backcalculated k-value be divided by two to estimate a static elastic k-value for use in design. This backcalculation method is described more fully in Appendix B.

In 1993, Crovetti developed equations for backcalculation of foundation k-values from interior, edge, and corner deflection measurements, and correction of infinite slab solutions for finite slab size effects (*53*). Crovetti demonstrated that Westergaard's equations for interior, edge, and corner deflection could be represented by quadratic functions of load radius divided by radius of relative stiffness (a/ℓ), and that and k-values could be calculated for these locations by an iterative process. In this study, a method was developed to rearrange these equations to solve for ℓ and k directly without iteration. Crovetti also developed algorithms for correction of backcalculated ℓ and k-values for a single finite (square or rectangular) slab, and for finite slabs with partial joint load transfer. These corrections were developed with the ILLI-SLAB finite element program. These and other recent developments in k-value backcalculation (including equations for use with the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) deflection data and equations for use with any sensor positions) are described in Appendix B.

The Iowa DOT has developed a procedure for determining springtime static k-values from Road Rater deflection data (*54*). The procedure was developed over several years by correlating Road Rater deflections on concrete pavements of known subgrade types to k-values obtained from static plate load test data and other correlations for the same subgrade types. The procedure is described in Appendix B.

All of the k-value backcalculation methods developed to date (except the Iowa procedure, based on direct correlation of deflections to static k-values) are based on plate theory, assuming pure bending of the concrete slab. A base layer, if it is considered,

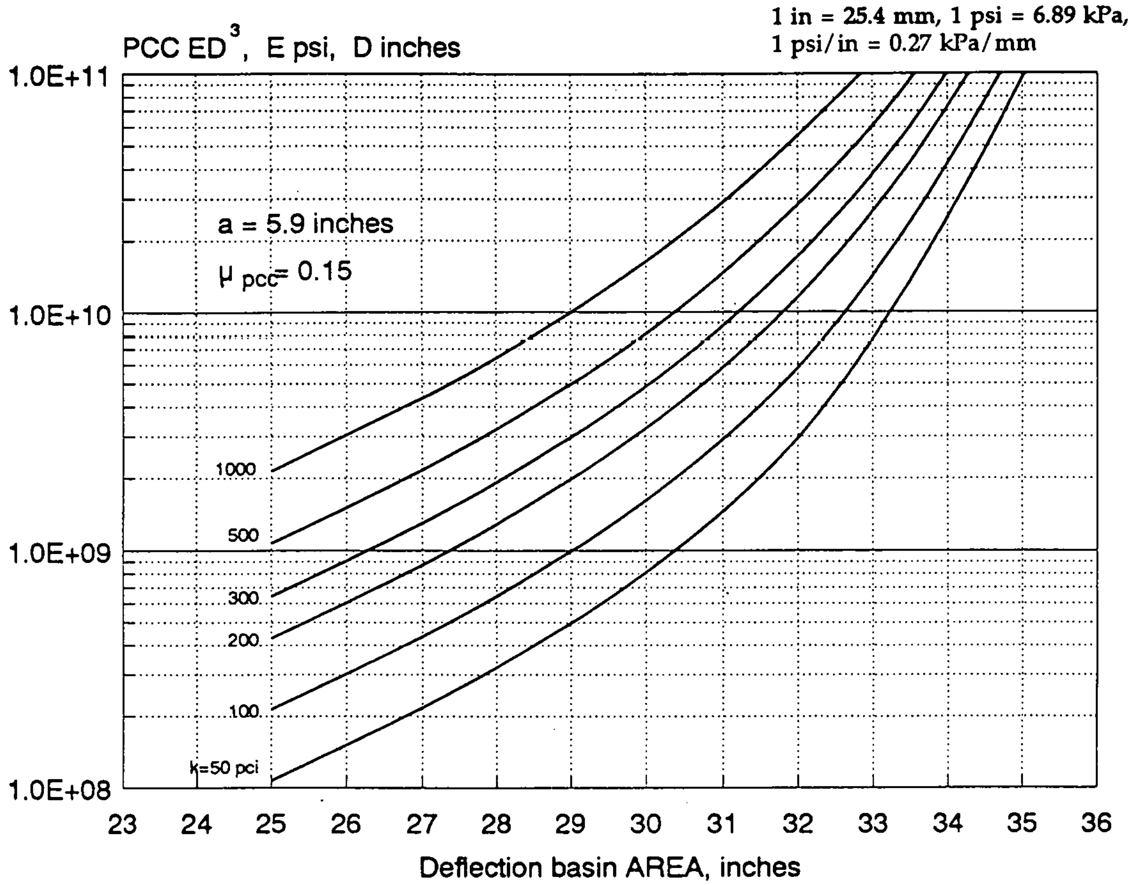


Figure 7. Concrete E determination from k-value, AREA, and slab thickness (52).

is generally also considered to exhibit plate behavior. In future work on *k*-value backcalculation methods for concrete pavements, three-dimensional finite element analysis is recommended to model the behavior of the slab and base as elastic layers on a *k*-foundation. The effects on backcalculated *k*-value of slab size, joint load transfer, base thickness and stiffness, slab/base interface friction, slab compressibility, and slab deformation due to temperature or moisture gradients could also be examined more realistically using three-dimensional finite element analysis.

BACKCALCULATION FIELD RESULTS

This section presents some backcalculation results from field testing that provide insight into subgrade *k*-values and the factors that influence them. More detail on these field studies is provided in Appendix A.

Backcalculated Versus Static *k*: AASHO Road Test Loop 1

Loop 1 of the AASHO Road Test was not trafficked during the experiment conducted between 1958 and 1960. It was used for strain measurements under vibratory loading and for materials sampling. Unlike the other AASHO Road Test loops, Loop 1 was not incorporated in the alignment of Interstate 80, and is still accessible today alongside I-80. Deflection testing of Loop 1 using a Falling Weight Deflectometer (FWD) was conducted in May 1992.

The Loop 1 deflection data were analyzed in great detail. Careful efforts were made to account for the effects of temperature, load transfer, slab size, and concrete slab compressibility. The results are shown in Table 1. The mean backcalculated *k* of 148 psi/in. (40 kPa/mm), when divided by 2, yields an estimated static *k* of 74 psi/in. (20 kPa/mm), which is within the range of 63 to 105 psi/in. (17 to 28 kPa/mm) obtained from elastic k_E plate load tests on the subgrade, and within the 25 to 92 psi/in. (7 to 25 kPa/mm) range of volumetric *k*-values obtained by the Corps of Engineers from static load tests on top of the slabs. The mean backcalculated concrete *E* of 6.37 million psi (43890 MPa) is also very similar to the value of 6.25 million psi (43062 MPa) obtained from dynamic tests on beam samples.

Backcalculated Versus Static *k*: Willard Airport

Because plate bearing tests are rarely conducted today, very little data are available to compare static *k*-values from plate bearing tests with backcalculated *k*-values from pavement testing at the same site. Foxworthy's analysis of the U.S. Air Force data is one of the few available examples of such a comparison.

TABLE 1. AASHO Road Test Loop 1 rigid pavement backcalculation results

Slab Thickness (in)	Mean Backcalculated k (psi/in)	Mean Concrete E (10^6 psi)
5.0	111	7.42
9.5	142	5.44
12.5	192	6.26
Overall Mean:	**148**	**6.37**

Note: 1 in = 25.4 mm, 1 psi/in = 0.27 kPa/mm, 1 million psi = 6890 MPa

Another example is the University of Illinois' Willard Airport in Savoy, Illinois. The results of ASTM D1196 nonrepetitive static-plate load tests conducted on the silty clay subgrade after slab removal yielded static *k*-values of 60, 75, and 120 psi/in. (16, 20, 33 kPa/mm). The subgrade soil has a density of about 93 lbs/ft^3 (1490 kg/m^3) and a CBR value of about 5 (*55*).

FWD testing was conducted at several locations on a nearby pavement having a relatively thin slab (8 in. [203 mm]) and an aggregate base. The backcalculated dynamic *k*-values for several locations near the plate load tests ranged from 111 to 163 psi/in. (30 to 44 kPa/mm) with a mean of 135 psi/in. (37 kPa/mm). Dividing this value by two to obtain an approximate static *k*-value gives 68 psi/in. (18 kPa/mm), which is close to the mean static-plate load test result of 85 psi/in. (23 kPa/mm).

Subgrade and Base Type Versus *k*: RPPR Field Studies

Field evaluations of 95 in-service jointed-plain concrete pavements (JPCP) and jointed-reinforced concrete pavements (JRCP) highway pavements located throughout the United States were conducted for the FHWA's "Rigid Pavement Performance and Rehabilitation" (RPPR) study (*56*). Deflection testing was conducted on these pavements using an FWD. The backcalculated *k*-values are summarized by subgrade soil classification in Table 2. The values shown are estimated static values obtained by dividing the mean backcalculated values by 2. The ranges of estimated static *k*-values for the various soil classes appear to be very reasonable in most cases.

Data were also obtained that provide backcalculated *k*-values for six different groups of experimental test sections located in six different states. At each experimental test site, the sections were grouped into those with an aggregate base and those with a treated base (asphalt treated, cement treated, and lean concrete). FWD backcalculation *k*-value results for these sections with different types of bases but the same subgrades are given in Table 3. The ratio of the *k*-value for treated base sections divided by the *k*-value for untreated base sections is shown to vary from 1.0 to 1.5 with an average of 1.2.

Thus, it appears that the treated base may have about a 20 percent effect on increasing the backcalculated *k*-value, although it is still believed that the backcalculated *k*-value is primarily that of the subgrade or embankment beneath the base course because that is where a large majority of deflection occurs.

The magnitude of increase in *k*-value for the treated base sections over the same subgrade is far less than would be predicted by the conventional top-of-base *k*-value charts, and much less than the composite *k*-values that would be predicted for treated bases by the 1986 *Guide* procedure. For example, a subgrade *k*-value of 100 psi/in. (27 kPa/mm) and a 5 in. (127 mm) treated aggregate base course would produce a composite *k*-value of 340 psi/in. (92 kPa/mm) using the PCA procedure and about 250 psi/in. (68 kPa/mm) for AASHTO, for a ratio of 2.5 to 3.4!

One possible explanation for the difference is slab size effect: the L/ℓ ratio for a slab with a bonded, stabilized base is lower than the L/ℓ ratio for a slab of the same dimensions on a granular base or no base at all. The lower the L/ℓ ratio is below 8, the less applicable are the infinite slab theory backcalculation methods, without adjustment.

Subgrade Type Versus *k*: LTPP Study

Subgrade *k*-values were backcalculated from deflection data collected from the General Pavement Studies (GPS) experiments 3 and 4 in the LTPP study, using the SHRP LTPP method described in Appendix B. The subgrade *k*-value results are presented by subgrade class in Table 4. The LTPP data include pavements with untreated granular bases and a variety of treated base types. These results are similar to those obtained from the RPPR deflection data, and are in reasonably good agreement with the *k*-value correlations suggested by the PCA and Corps of Engineers charts.

Effect of *k*-Value on Performance: LTPP

Preliminary analysis of the concrete pavement LTPP data has shown some relationships between backcalculated *k*-value and concrete pavement performance (*57*). Prediction models were developed for various distresses for JPCP, JRCP, and continuously reinforced concrete pavements (CRCP). The following effects of backcalculated *k*-value were noted in the prediction models developed:

TABLE 2. Backcalculated *k*-values by soil class, from RPPR data (*56*)

Subgrade Class	Number of Projects	Minimum Static k value (psi/in)	Maximum Static k value (psi/in)
A-1-a	10	116	310
A-1-b	0	---	---
A-2-4	22	64	374
A-2-5	0	---	---
A-2-6	8	64	336
A-2-7	1	128	128
A-3	2	189	265
A-4	17	95	314
A-5	0	---	---
A-6	22	78	311
A-7	1	170	170

1 psi/in = 0.27 kPa/mm

TABLE 3. Effect of treated and untreated bases on *k*, for projects with different bases at same location, from RPPR data (*56*)

Location (State) of Project	Backcalculated k (psi/in)		
	Aggregate Base	Treated Base	Ratio
North Carolina	554	535	< 1.0
Ohio	395	482	1.2
New York	577	560	< 1.0
Michigan	304	468	1.5
Minnesota	200	270	1.3
Arizona	425	602	1.4

1 psi/in = 0.27 kPa/mm

- **As *k*-value increased**: Faulting of dowelled joints decreased,
  Transverse cracking of JPCP decreased,
  Deteriorated transverse cracks in JRCP decreased,
  Lower International Roughness Index (IRI) resulted for JPCP, and
  Lower IRI resulted for JRCP.

Lower IRI also resulted for CRCP when the subgrade type changed from fine-grained to coarse-grained. Although these results are tentative, they do indicate that the backcalculated *k*-value had some influence on the performance of all types of concrete pavement. As the *k*-value increased, the performance generally improved.

Plate Load *k* on High-Strength Base: Japan

A Japanese study of deflection and strain measurements on concrete airfield pavements provides an interesting comparison of top-of-base plate load *k*-values and *k*-values backcalculated from slab deflections (*58*). Each section of concrete pavement was constructed on a crushed stone layer either 4 or 8 in. (102 or 203 mm) thick, which was in turn placed on a 12-in. (305-mm) layer of pit gravel. Plate load tests were conducted on top of the base to determine *k*-values.

TABLE 4. **Backcalculated *k*-values by soil class, from LTPP GPS 3 and 4**

Subgrade Class	Number of Projects	Minimum Static k (psi/in)	Maximum Static k (psi/in)	Average Static k (psi/in)
A-1-a	5	108	181	142
A-1-b	5	92	334	208
A-2-4	16	48	535	188
A-2-5	0	---	---	---
A-2-6	2	101	370	235
A-2-7	4	68	239	133
A-3	13	60	271	135
A-4	24	54	395	154
A-5	2	66	102	84
A-6	26	61	512	146
A-7-5	5	79	181	117
A-7-6	18	48	248	126

1 psi/in = 0.27 kPa/mm

After the concrete slabs were constructed, static load tests were conducted on the slabs with a 12-in. (305-mm)-diameter plate. A 45-ton (400-kN) truck provided the reaction force for the load tests at the slab interiors, edges, and corners. Westergaard's equations were used to backcalculate *k*-values using the maximum deflections and the concrete *E* determined from laboratory tests.

The *k*-values measured on top of the base were consistently two to four or more times the *k*-values backcalculated from top of slab deflections, and this discrepancy increased with increasing load level. These results confirm again that plate tests on base layers yield misleadingly high *k*-values, shown by slab deflection.

Effects of Fill and Bedrock: Dulles Airport

Dulles International Airport near Washington, D. C. provides an interesting example of the effect of a shallow rigid layer on deflections and backcalculated *k*-values (*59*). The site upon which the airport was built has shallow bedrock (stratified red shale), varying in depth from 0 to 6 ft (1.8 m) throughout most of the airport property. The overlying soils are red clayey silt and silty clay (FAA class E-7, liquid limit 34, plasticity index 17, dry density 104 lb/ft^3 [1666 kg/m^3]). If the bedrock layer were not present and this soil were present to a substantial depth, it might be expected to have a static *k*-value in the range of about 100 to 250 psi/in. (27 to 67 kPa/mm).

The concrete slabs are all 15 in. (381 mm) thick. The elastic modulus of the concrete was estimated at 5.4 million psi (37200 MPa) from sonic modulus tests on cores and from backcalculation of Waterways Experiment Station (WES) Vibrator and FWD deflection data. The backcalculated dynamic *k*-values for the 35 pavement sections tested ranged from 260 to 1000 psi/in. (70 to 270 kPa/mm), and averaged 480 psi/in. (130 kPa/mm). These correspond to an estimated static *k*-value range of 130 to 500 psi/in. (35 to 135 kPa/mm), and an estimated average static *k*-value of 240 psi/in. (65 kPa/mm). There is, however, considerable variation in fill heights and depths to bedrock within a section, and a corresponding variation in the *k*-values, as illustrated in Figure 8.

The average static *k*-value is within but near the high end of the range of values predicted for the subgrade soil type, which means that about half of the values are above the range expected for the subgrade type. The highest values (500 psi/in. [135 kPa/mm]) are twice the expected upper limit (about 250 psi/in. [67 kPa/mm]) for the subgrade type. The highest *k*-values were reported for areas with bedrock at a very shallow depth (0 to 2 ft [0.6 m]).

These results suggest that bedrock or a similar stiff layer at a shallow depth (i.e., within 10 ft of the subgrade surface) may increase *k*-values to as much as twice the level that would otherwise be assigned to the subgrade soil based on its classification, density, and other properties. This type of field information is valuable because a shallow rigid layer is considered to be significant in producing an effectively stiffer foundation, but the magnitude of the increase that should be expected due to a rigid layer is extremely difficult to quantify. Simulating the effect of a rigid layer in an elastic layer computer analysis can yield much greater changes in *k*-value (e.g., by a factor of five or more), which may be very erroneous. Additional collection and analysis of field data on rigid layer depth and its effect on subgrade *k*-values are needed to more accurately quantify this effect.

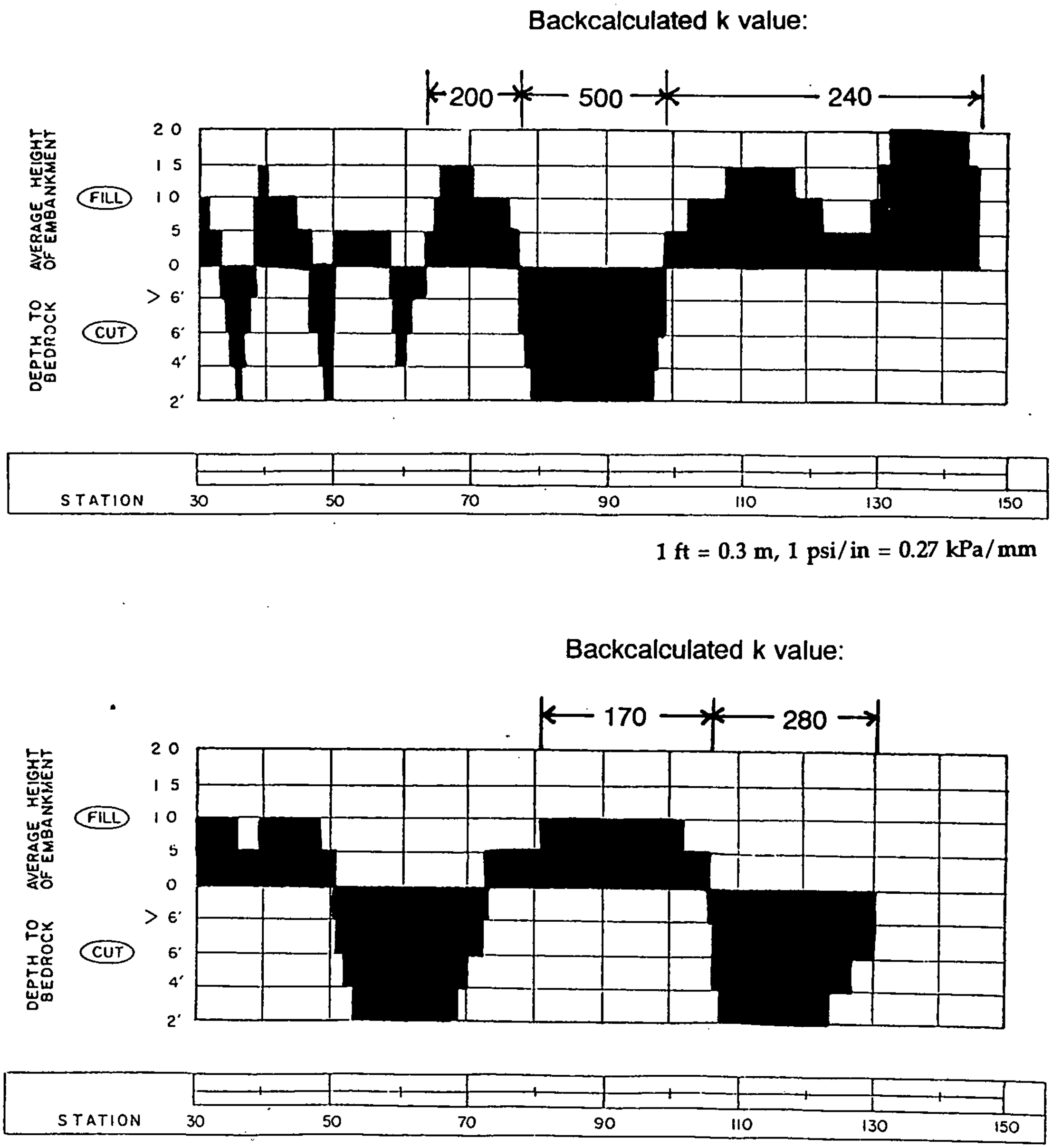


Figure 8. Effect of fill and bedrock on backcalculated k-values at Dulles International Airport (59).

Soft Subgrade With High Friction Treated Base: Utah I-15 in Salt Lake City (*60*)

This highway pavement consists of a 9-in. (229-mm) undowelled JPCP over a 4-in. (102-mm) cement-treated base, over a 12-in. (305-mm) granular borrow layer consisting of AASHTO A-1 and A-2 materials. Beneath the borrow layer is selected embankment material and below this are very soft lake bed clays (CBR 0-3 percent). During the construction of I-15, vertical sand drains were installed in these soft soils to drain water as the soils consolidated under the weight of the embankment. The embankment had to be constructed in 1-ft (0.3 m) increments because of the large volume of water that was drained from the underlying soils during consolidation.

This pavement is over 25 years old and has shown excellent performance with virtually no cracking and little faulting. The pavement was tested with an FWD, and the *k*-value was backcalculated using the 1993 AASHTO *Guide* procedure described in Appendix B. The backcalculated dynamic *k*-values ranged from 49 to 234 psi/in. (13 to 63 kPa/mm), corresponding to a range of estimated static *k*-values of 25 to 115 psi/in. (7 to 31 kPa/mm). These are among the lowest *k*-values observed anywhere, despite the presence of the embankment, borrow layer, and cement-treated base. The *k*-values were higher in one end of the project where the embankments were over 10 ft (3 m) high, and lower in the other end where the embankments were shallower. Thus, the extremely soft lake bed clays, even buried beneath several feet of improved material and a stiff base has a large effect on the backcalculated *k*-value.

It is also interesting to note that this pavement has shown no fatigue-related cracking in over 25 years of heavy traffic for a

TABLE 5. Mean backcalculated *k*-values from 20 Chilean JPCP test sections (*61*)

New Alignment Subgrade	Constructed Over Old AC or PCC Pavement
Mean k = 185 psi/in n = 5 sections Slabs Cracked = 6 %	Mean k = 368 psi/in n = 15 sections Slabs Cracked = 23 %

1 psi/in = 0.27 kPa/mm

relatively thin slab. A structural analysis conducted assuming that the slab and base had no frictional resistance indicated that the pavement should have failed from fatigue damage. A second analysis conducted assuming that there was high friction indicated that there should be very little fatigue damage, which is consistent with the performance observed (*60*).

Effect of *k*-Value on Slab Cracking: Chile

A major research study has been underway in Chile for several years to monitor the deterioration of undowelled JPCP (*61*). Twenty of these pavements were instrumented and tested in a variety of ways. Results from this monitoring show a "permanent upward curling of slabs in all pavement sections. . . . The curling is demonstrated in the field by the perceptible rocking of the slabs under the early morning traffic and by the systematic transverse cracking and corner breaks of some rather new pavements with no signs of pumping. Cracking seems to start from the surface downward and from the edges inward" (*61*). The researchers have also concluded that moisture gradients in the concrete slabs have produced slab stresses, deformations, and increased deflections under load as significant as those caused by temperature gradients (*62*).

Static axle deflection data at the slab centers and an estimate of the concrete modulus of elasticity (from strength) were obtained and the static *k*-value was backcalculated using Westergaard's center deflection equation. The backcalculated static *k*-values ranged from 87 to 675 psi/in. (23.5 to 182 kPa/mm) and seemed to vary considerably depending on the type of foundation. Some of the JPCP were constructed on new alignments having a normal subgrade and some were constructed over old AC or PCC pavement. The base type was mostly cement treated aggregates placed over the subgrade or over the existing old pavements.

Results shown in Table 5 indicates some important results. Pavements constructed over old deteriorated pavements had backcalculated *k*-values about two times the *k*-value of pavements constructed on a new alignment with a typical subgrade. The percentage of cracked slabs of pavements constructed over old pavements was about four times that of pavements constructed on new alignments. The underlying existing pavements have provided a much stiffer foundation, which may have contributed to increased stresses in the slabs when deformed by load, and temperature and moisture gradients, resulting in increased cracking. These results are consistent with the 3-D finite element analyses conducted in this study and proposed revised AASHTO design procedure.

RECOMMENDATIONS FOR DETERMINING *k*-VALUE

Three categories of methods for determining a *k*-value for use in concrete pavement design are recommended. Further details on the procedures are given in Appendixes B and F.

Correlation Methods

The *k*-value can be estimated using soil classification, resilient modulus, moisture level, density, CBR, Hveem R-value, and the Dynamic Cone Penetrometer (DCP). These correlation methods are anticipated to be used routinely for design.

Deflection Testing and Backcalculation Methods

These methods are suitable for determining *k*-value for design of overlays of existing pavements, or for design of reconstructed pavements on existing alignments, or for design of similar pavements in the same general location on the same type of subgrade. An agency may also use backcalculation methods to develop correlations between nondestructive deflection testing results and subgrade types and properties.

Plate Testing Methods

The standard ASTM, AASHTO, and Corps of Engineers nonrepetitive and repetitive plate-loading test methods, as well as the German plate-load test, are summarized in Appendix B. The American standard test methods are the most direct methods of determining the elastic *k*-value of the soil under static loading, but because these tests are costly and time consuming, it is not anticipated that they will be conducted routinely.

Adjustment for Fill and Rigid Layer

A nomograph was developed to adjust the *k*-value that would be assigned to a subgrade using the correlation methods to account for the effects of a substantial embankment thickness above the natural subgrade and/or a rigid layer (e.g., bedrock or hardpan clay) at a shallow depth beneath the natural subgrade. The nomograph for embankment and rigid layer effects is provided in Appendix B.

3-D FINITE ELEMENT MODEL OF CONCRETE PAVEMENT

A 3-D finite element model for concrete pavements was developed in this study in order to analyze accurately the many complex and interacting factors that influence the support provided to a concrete pavement, including the following:

- foundation support (subgrade *k*-value);
- base thickness, stiffness, and interface friction;
- slab curling and warping due to temperature and moisture gradients;
- dowel and aggregate interlock load transfer action at joints; and
- improved support with a widened lane, widened base, or tied concrete shoulder.

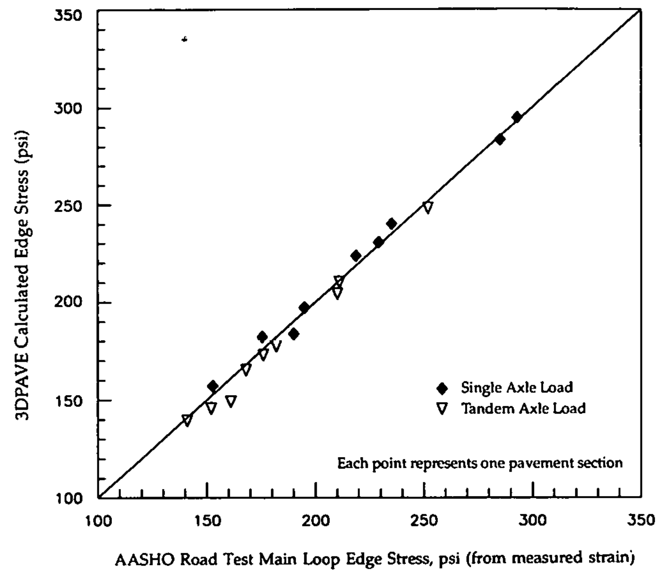
The ABAQUS general-purpose finite element software was used to develop a very powerful and versatile 3-D model for analysis of concrete pavements. The 3DPAVE model easily overcomes many of the inherent limitations of 2-D finite element models, which reduce the accuracy of the results obtained from 2-D models.

The 3-D model was built in steps, first by building and validating a simpler 2-D model, and then extending the 2-D model. A careful analysis of ABAQUS' many element types, features, and options was conducted to select the components that would produce a robust and efficient model. During its development, 3DPAVE was checked against the existing 2-D finite element model ILLI-SLAB and against available theoretical solutions (e.g., Westergaard's equations). The 3DPAVE consistently outperformed the 2-D model in accuracy over wide ranges of inputs for a variety of problems.

The 3-D model was validated by comparison with deflection and strain measurements for traffic loadings and temperature variations from the AASHO Road Test, the Arlington Road Test, and the Portland Cement Association's slab experiments. A comparison of stresses computed with 3DPAVE and stresses computed from strains measured at the AASHO Road Test is shown in Figure 9. In every comparison with measured field data, 3DPAVE's calculated responses were found to be in very good agreement with the measured responses, and significantly closer to the measured responses than those calculated by the 2-D program. The development and validation of the 3DPAVE model is documented in Appendix D.

LOSS OF SUPPORT

Loss of support refers to any gap or void that may occur between the base and the slab, or between a stabilized base and the subgrade, causing increased deflection of the slab surface. There are three basic types of "loss of support" that a concrete slab exhibits over time.



PCC E = 6.25 million psi [43063 MPa]
PCC μ = 0.28
k = 170 psi/in [46 kPa/mm]
Speed = 30 mph [48 km/hr]
Axle load range = 12 to 48 kips [53 to 214 kN]
Slab thickness range = 6.5 to 11 inches [165 to 279 mm]
1 psi = 6.89 kPa
3DPAVE/AASHO stress ratio range is from 0.93 to 1.03
Mean 3DPAVE/AASHO stress ratio is 1.00
R^2 = 0.99
n = 18

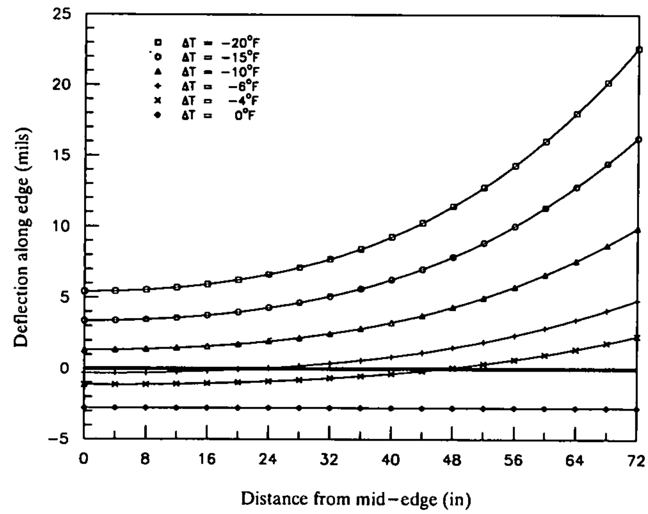
Figure 9. 3DPAVE calculated stresses versus stresses calculated from strains measured at the AASHO Road Test.

- Erosion of the base and/or subgrade from beneath the slab, resulting in increased deflections and stresses in the slab.
- Settlement or consolidation of the base and/or subgrade, usually resulting in slab cracking in the vicinity of the settlement.
- Temperature curling and moisture warping of the slab, resulting in increased deflections and stresses in the slab. Permanent construction curling presents a potential for very serious loss of support and early failure of jointed concrete pavement.

Loss of support can have a major impact on slab deflections and stresses, and thus pavement life. These phenomena are discussed below and in more detail in Appendix C.

Loss of Support from Erosion

Pumping results in loss of support either beneath the slab itself or beneath a treated base that is bonded to the slab. Either of these situations can lead to increased deflections and stresses in the concrete slab and is of concern to the design engineer. The extensive amount of loss of support that occurred at the AASHO Road Test site is well documented (see Appendix C). If this loss of support had not occurred, many of the sections would have carried additional traffic loadings. Note that this erosion occurred even though the transverse joints were adequately dowelled to prevent faulting. If the joints were not dow-



Notes: Midslab is at 0 inches and the corner is at 72 inches [1829 mm]
Joint spacing L = 12 ft [3.7 m]
k = 250 psi/in [68 kPa/mm]
Slab thickness D = 8 in [203 mm]
Concrete elastic modulus E_c = 4 million psi [27560 MPa]
ΔT = top temperature - bottom temperature, °F
1 °F = 0.55 °C, 1 in = 25.4 mm, 1 mil = 25.4 μm

Figure 10. Illustration of slab surface deflection due to negative temperature differentials through the slab thickness, computed using 3DPAVE.

elled, erosion would have been even greater and led to earlier failure.

No methodology currently exists to predict the amount of loss that may occur from erosion. To directly consider this in design, the designer needs to be able to approximately predict the amount of loss of support that may develop over time so that the increased stresses can be predicted and their effect on cracking included. A second main effect of pumping is joint faulting. An erodible base normally results in greater faulting and decreased life.

As far as the AASHTO design method is concerned, the extensive loss of support that occurred during the test period had a very significant effect on pavement load-carrying capacity. It is hard to imagine a greater amount of loss of support than that which occurred at the AASHO Road Test site. Therefore, the AASHTO performance model is calibrated to the performance of pavements that experienced extensive loss of support. Further increasing the slab thickness for anticipated loss of support represents an overdesign, which is not recommended. The current AASHTO design procedure does not provide any way, other than an unrealistic increase in the *k*-value input, to account for the benefit that an improved base would have on performance. If the pavement is designed for a base type that is stronger and less erodible (or more permeable) than a dense-graded granular base such as that to which the AASHTO performance model is calibrated, the effects of the improved base on reduced slab stress and reduced faulting are accounted for in the proposed revised AASHTO design model, which is presented in this report. Joint faulting can be predicted from several available prediction models. A design check can be done to determine if joint faulting is excessive, and if so, a modified joint design can be proposed.

For other design procedures, however, the impact of erosion and loss of support on the load-carrying capacity should be considered differently. This involves a complex analysis of the erodibility of underlying base and subgrade materials, erosion of the concrete slab itself, the friction condition between the base and concrete slab, the magnitude of deflections and load transfer at the joints, the number of axles, and a subdrainage analysis of the pavement section. Several of these factors must be predicted not only for the newly constructed pavement, but also over the life of the pavement (e.g., loss of load transfer over time). The development of a predictive model for this complex phenomenon would be extremely difficult. Furthermore, any such model, if developed, would need to be validated with field performance data to assess its predictive capability. The lack of good, field-validated models for loss of support is currently an obstacle to predicting pavement life as a function of progressive loss of support and increasing slab stress in a mechanistic design procedure.

The most comprehensive design and construction guidelines available on ways to minimize erosion and loss of support are provided in the manual "Combatting Concrete Pavement Slab Pumping" by the Permanent International Association of Road Congresses (PIARC) Technical Committee on Concrete Roads from eight European countries and the United States (*63*). Following are the general principles:

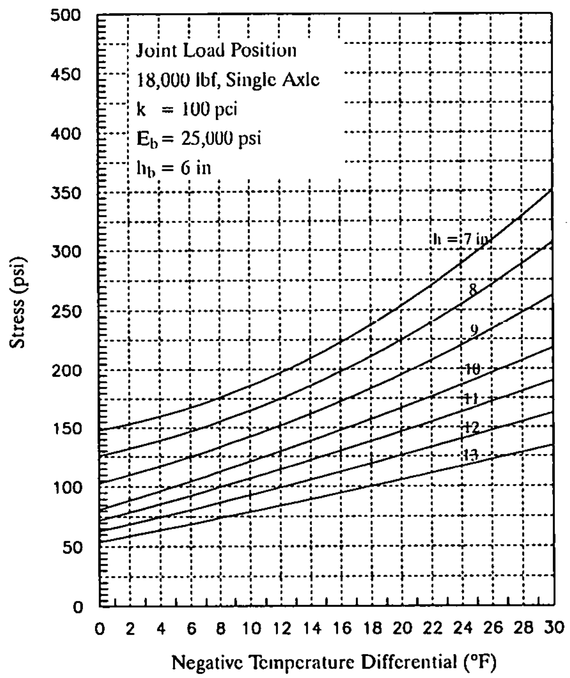
- The erodibility of subbase and shoulder materials is an important property that must be taken into account in the design of new pavements and evaluation of existing pavements, because the erosion of materials at interfaces causes pumping and destabilizes concrete slabs.
- Simple tests have been developed for characterizing the erodibility of materials. They offer data required for comparing results using a common language understood in all countries (rotary brush, jetting, rotational shear).

Five classes of erodibility resistance are defined in terms of material type and binder content (see Appendix C).

Loss of Support from Temperature Curling

Curling of a slab is caused by temperature differential through the slab, which occurs on a daily basis. A negative (top cooler than bottom) temperature differential occurs at night and results in the corners and edges displacing upward, creating the potential for a gap or void between the slab and base or subgrade. Figure 10 illustrates the magnitude of slab curling for a range of nighttime temperature differentials. When this happens, any load near the corner or joint will cause an increased stress on the surface of the slab that could lead to corner breaks, diagonal cracks, or even transverse cracks several feet from the joint. Figure 11 illustrates the stress caused by a load at the joint for a given combination of subgrade *k*, base modulus, and base thickness, for a range of nighttime temperature differentials.

A permanent form of slab curling caused by a temperature differential at construction has recently been identified by researchers in Chile and Germany (*62, 64, 65, 66*). Permanent upward corner and edge curling may occur if a high positive



1°F = 0.55°C, 1 psi = 6.89 kPa, 1 in = 25.4 mm, 1 lbf = 4.45 N, 1 psi/in = 0.27 kPa/mm

Figure 11. Increase in tensile stress at top of slab from increased negative temperature differential from top to bottom of slab.

(top warmer than bottom) temperature differential exists through the slab as it hardens. This occurs particularly on sunny days and unless extra precautions are taken to keep the top of the slab cool. This temperature differential has not been measured extensively in the past and its magnitude is not well known at the present time. One set of data from Germany shows that 5 hours after placement in sunshine, the top of a 8-in. (203-mm) slab had a temperature of 116°F (47°C) and the bottom had a temperature of 80°F (27°C). If the slab solidifies in a flat position with this large positive thermal gradient, the corners and edges will be permanently curled upward for any lower temperature gradient. Figure 12 illustrates the permanent construction curling which developed within 48 hours after placement for a concrete pavement in Germany.

"Construction curling" is defined as the temperature differential that would be required to produce a flat slab (note that this is before any moisture shrinkage at the top of the slab is taken into account). A study of a concrete highway pavement in Florida found that a 9°F (5°C) temperature gradient was required through the slab to flatten it (*67*). Obviously, any such permanent upward curling would create a serious loss of support beneath the corners and edges of the slab. Such a phenomenon has been identified in Chile where several undowelled pavements exhibited excessive corner cracking within a few years after placement.

Loss of Support from Moisture Shrinkage Warping

Warping of the slab due to a moisture gradient (top drier than the bottom) occurs seasonally (*56, 62, 64, 67*). Data from Illinois showed that substantial drying occurs only at the top surface to a depth of less than 2 in. (51 mm). The rest of the slab remains at 80 percent saturation or higher (*68*). However, in dryer, less humid climates, greater drying and upward warping of the slab may occur. Figure 13 shows an example of seasonal warping of JPCP with no temperature differential for a typical JPCP in Chile (dryer climate area). The corners are warped upward almost 0.05 in. (1.27 mm) during the dry season (*66*). From the German and Chilean data it appears that moisture warping results in deformation equivalent to that which would be caused by a negative temperature gradient of about 0.5 to 0.7°F per in. (0.011 to 0.0015°C per mm) of slab thickness.

These three climatic effects (temperature curling, construction curling, and moisture shrinkage warping) can all combine to cause a large tensile stress at the top of the slab near the joint, which in combination with axle loads could eventually lead to serious slab cracking. Combined stresses from negative temperature differentials and from load can be estimated using the 3-D finite element model 3DPAVE.

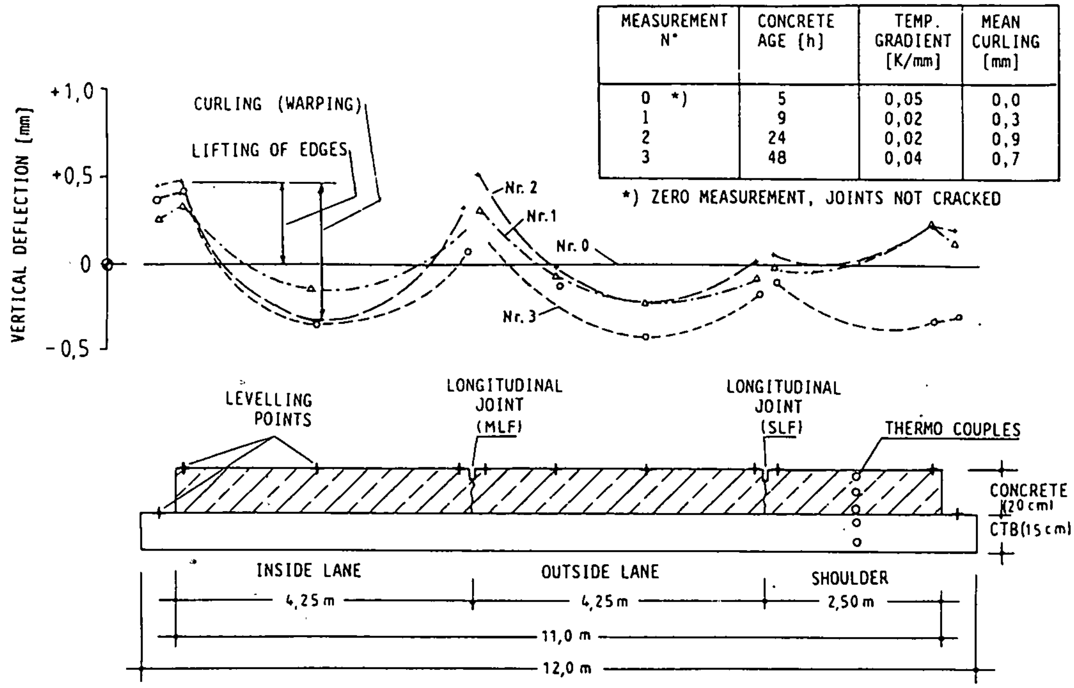
Consideration of Loss of Support in Design

The following recommendations were developed in this study for considering loss of support in the design process.

AASHTO Guide. The loss-of-support factor in the current AASHTO design procedures is not recommended because it is inconsistent with the performance of the AASHTO pavements to which the design equation is calibrated. Extensive loss of support occurred at the Road Test and thus is already built into the rigid pavement design model for slabs on dense-graded granular bases. The potential benefit to performance of a stiffer, less erodible, or more permeable base cannot be considered in the current AASHTO methodology.

In the proposed revision to the AASHTO procedure, the base thickness, stiffness, and friction with the slab are considered in calculation of slab stress due to midslab loading, and a design check is provided for joint/corner loading for undowelled pavements to identify cases for which this position may be critical. If the maximum stress due to joint/corner loading is greater than the maximum stress for midslab loading, a design modification is required. This check is not required for dowelled pavements because corner breaks or diagonal cracks have rarely occurred with dowelled joints. A design check for joint faulting is also recommended to ensure that for the design being considered, the load transfer, base type, and subdrainage are adequate to limit faulting to an acceptable level.

Mechanistic Design. Prediction of loss of support caused by thermal curling or moisture warping is possible and could be considered in design using 3-D finite element models. Prediction of additional loss of support from erosion is extremely difficult and would require a major research effort. Currently, it is recommended to determine the minimum material requirements that would minimize the occurrence of erosion under varying climatic, traffic, and design conditions, using the PIARC recommendations for example (*63*). These conditions should be met in the design and construction of the pavement. The effects of



MEASUREMENT N°	CONCRETE AGE [h]	TEMP. GRADIENT [K/mm]	MEAN CURLING [mm]
0 *)	5	0,05	0,0
1	9	0,02	0,3
2	24	0,02	0,9
3	48	0,04	0,7

1 in = 2.54 mm, 1 ft = 0.305 m, 1°F/in = 0.022°K/mm

Figure 12. Development of permanent construction curling in a concrete slab within 48 hours after paving, in Germany (66).

thermal gradients, construction curling, and moisture warping should be considered in the design process for corner loading. A design check should be employed for joint faulting that would ensure that the load transfer, base type, and subdrainage are adequate to limit faulting to an acceptable level.

IMPROVED CONSIDERATION OF SUPPORT IN AASHTO METHODOLOGY

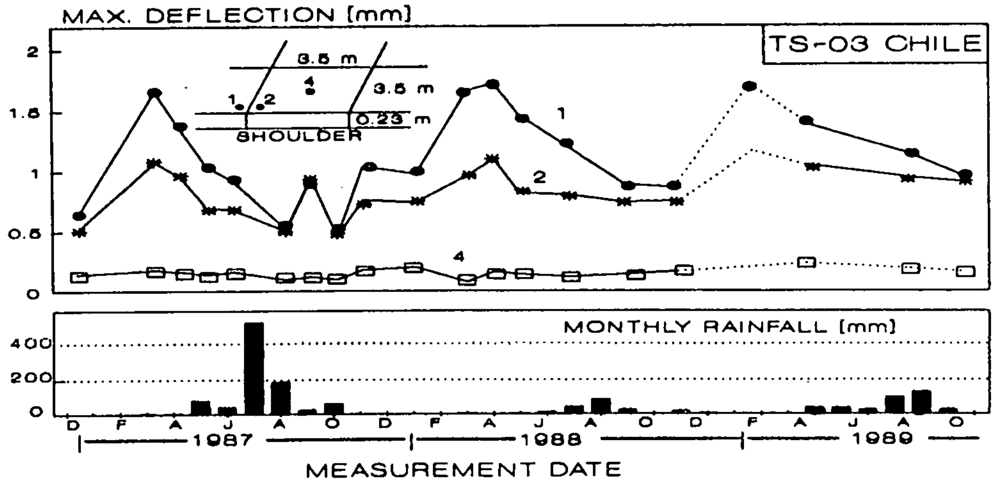
A comprehensive evaluation of the AASHO Road Test and the resulting concrete pavement design models revealed several major deficiencies related to pavement support conditions. Because of the nature of these deficiencies, a major effort was required to develop procedures for improved consideration of support into the AASHTO design methodology. Details of the development are given in Appendix E.

This effort required an extensive examination of the design and subsequent performance of the test pavements at the AASHO Road Test, a detailed examination of the original development of the concrete pavement design model and its subsequent "extensions" over time, the formulation of recommended improvements for pavement support, and finally the incorporation of these improvements into a proposed revision to the AASHTO design model with different support inputs. Efforts were then made to verify the proposed revised AASHTO design model using long-term performance data from the extended AASHO Road Test and other in-service pavements in a variety of climatic zones. The proposed revisions to the relevant portions of the AASHTO *Guide* are provided in Appendix F.

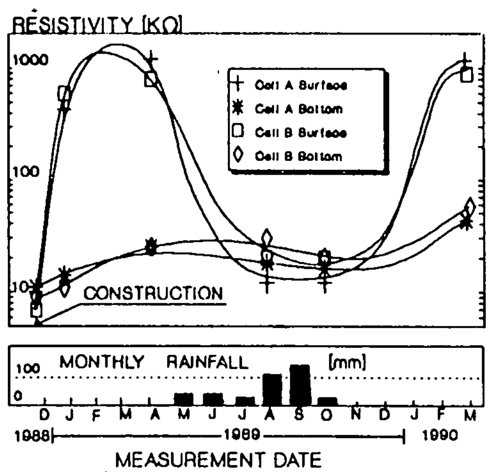
Concrete Pavement Performance at the AASHO Road Test

Details of the physical design and materials of the AASHO Road Test pavements are given in Appendix E. Traffic loadings were applied over a 2-year test period. Visible distress and roughness were recorded over this time period. Key performance aspects related to support conditions are summarized as follows.

- Extensive pumping and erosion of the sand-gravel base occurred, causing loss of support beneath the corners and edges of the slab. The amount of material pumped onto the shoulder was so great that it was actually measured in a cubic foot container. A "pumping index" (PI) was computed as cubic inches of pumped material per inch along the pavement. The PI ranged from 0 to over 200 depending on slab thickness and axle loading.
- "By removing the concrete from a few failed sections and sampling the underlying material, it was observed that subbase material had apparently been removed by erosive action of water moving across the top of the subbase, and that the remaining subbase material was relatively undisturbed ... Inasmuch as the great majority of the sections which failed pumped severely prior to failure, many of these sections would have survived the



Annual cyclic variation of slab deflections and its relation to rainfall. No temperature differential through the slab thickness.



Annual cyclic variation of electric resistivity at the slabs surface and bottom cells in the experimental pavement, and its relation to rainfall.

Figure 13. Annual cyclic variation of slab deflections and its relation to rainfall (Note: Measurements were made when no temperature gradient was present in the slab.) (62).

two years of traffic had the subbase material been stabilized effectively to resist erosion by water (*32*)."

- Slab cracking occurred on the thinner sections within each loop. Thinner slabs (i.e., 2.5 to 5 in. [63.5 to 127 mm]) developed mostly longitudinal cracks in the wheel paths. Thicker slabs developed transverse cracks that initiated mostly in the middle third of the 15-ft (4.6-m) slabs. Almost none of the JPCP 11-in. and 12.5-in. [279- and 317.5-mm] slabs cracked during the 2 years of the test, nor under an additional 14 years of I-80 traffic.
- No faulting of the dowelled transverse joints occurred during the 2-year period. Some faulting occurred later in the 8-in. (203-mm) slabs (with 1-in.-diameter [25-mm] dowels) and 9.5-in. (229-mm) slabs in the I-80 extended traffic tests. Thicker slabs with larger dowel bars did not fault during the extended tests.

Original Empirical Concrete Pavement Performance Model

At the end of the 2-year traffic period, the performance data were analyzed and prediction models were developed. Two key prediction models were developed and incorporated into the final design model:

• EMPIRICAL MAIN LOOPS MODEL: An empirical model was developed for log W (number of axle load applications in lane) as a function of slab thickness (D), loss of serviceability (P1−P2), axle type, and axle weight, based on data from the main trafficked loops (*32*). This empirical model is limited to the design, climate, subgrade, age, and traffic conditions at the Road Test site.

• MECHANISTIC-EMPIRICAL MODEL: A mechanistic-empirical model for log W was developed as a function of the ratio of concrete flexural strength (S_c) and tensile stress (σ) in the slab and terminal serviceability P2. This model was used to "extend" or incorporate theory into the empirical model so that other design features such as concrete modulus of elasticity (E), concrete strength (S_c), and subgrade k-value could be included (*32*).

The "extended" 1961 AASHTO design model was obtained by combining the empirical and mechanistic models described above. This provided improved capabilities to design pavements with some different design features than those of the AASHO Road Test pavements. However, this model had some serious deficiencies. The following assumptions were made or are inherent in its derivation:

1. The variation in load applications (W) required to reach a certain S_c/σ level for variable loads is properly evaluated by the Road Test equations and is adequately expressed by the use of equivalence factors to express all loads in terms of 18,000-lbf (40-kN) single-axle loads.
2. Any change in S_c/σ due to variations in physical constants (such as E, k, D, and S_c) will have the same effect as varying slab thickness, and this relationship is defined by the mechanistic-empirical model.
3. Thermal and moisture gradients existed in the Road Test slabs and their effects are included in both equations; however, the effect of a different climate with different thermal and moisture gradients is not considered in the extension of the equation. Thus, the effects of different design features such as joint spacing, a stiff base, or a stiffer subgrade (higher k-value) when thermal curling and moisture warping stresses exist are not considered at all.
4. Faulting at the dowelled transverse joints did not occur during the AASHO Road Test, even though extensive pumping and erosion occurred. The extended equation still does not include the consideration of faulting. The J-factor considers only corner stresses that lead to cracking, not joint faulting.

Additions to the 1961 Extended Model Through 1993 Related to Support

There have been several additions to the final 1961 design model over the years that are related to pavement support. As noted below, the ways in which these additions were made has resulted in serious deficiencies in the current concrete pavement design model.

Composite k-value. The k-value input defined in 1961 was the "gross" k-value of 60 psi/in. (16 kPa/mm), which was actually a typical value in the spring of the year, on top of the granular base layer. In the 1972 version of the *Guide,* an alternate graphical procedure was added whereby the k-value on top of the base course (called a composite k) could be determined if the resilient modulus and thickness of the base were known, along with the k-value or resilient modulus of the subgrade. In the 1986 *Guide* the composite k-value approach became the standard method. Thus, the effect of the base layer on slab thickness design was accounted for through the composite k-value of the foundation. This approach to consideration of the base through increased k-value results in (a) the base course having very little effect on performance or slab thickness design, and (b) k-values being assigned to the foundation which are unrealistic, compared to measured deflections.

Use of a composite k-value is thus neither consistent with the AASHTO methodology nor realistic. The concrete slab does not actually respond to loading as if it were supported by a foundation with the stiffness that the composite k concept suggests. The true benefit of a base layer is in its bending response with the slab. The ability of a base to reduce stresses in a concrete slab is primarily a function of the base thickness, base stiffness, and friction coefficient between the base and the slab.

Loss of support. A procedure was added to the 1986 *Guide* whereby the composite k-value was reduced considerably depending on the relative erodibility of the base material. The baseline, LS = 1.0 was a very stiff, relatively unerodible base (high-strength cement-treated or lean concrete base). With only a moderate degree of erodibility, the design k-value is reduced tremendously. For example, a composite k-value of 300 psi/in. (81 kPa/mm) would be reduced as shown for different LS values:

LS	**Reduced *k*-value**
1	100 psi/in. (27 kPa/mm)
2	31 psi/in. (8 kPa/mm)
3	13 psi/in. (3.5 kPa/mm)

This reduction is inappropriate because the original AASHO Road Test model was developed for pavements that experienced extensive erosion of the dense-graded base course during the Road Test as previously described. Photographs in the Road Test reports show persons pushing yard sticks into voids, which developed under the pavement slabs. An adjustment for even more loss of support would result in much greater slab thickness requirements than predicted by the AASHO Road Test results.

Effective k-value over seasons. A procedure to compute a seasonally adjusted k-value was included in 1986. The seasonally adjusted k-value was called the "effective k-value." However, the k-value built into the 1961 design equation, and retained in the equation as presented in the 1986 *Guide,* was the gross k-value of 60 psi/in. (16 kPa/mm) measured in the springtime, the lowest of the entire year at the Road Test site, not the seasonally adjusted "effective k-value." Thus, the 1986 revision should have included a basic revision to the concrete pavement extended equation to incorporate a seasonally adjusted effective k-value. The use of a seasonally adjusted effective k-value is not truly appropriate until this revision to the design equation is accomplished.

Drainage coefficient, C_d. According to the *Guide,* this factor depends on the percent time the subgrade approaches saturation and the drainage time for the base course. The Road Test pavements obviously had very poor subdrainage, as evidenced by the extensive erosion, pumping, and loss of support that occurred. This poor subdrainage and loss of support is built into the design equation through the use of the data from the Road Test sections that had extensive erosion and loss of support. Thus, the 1986 *Guide* not only added an unnecessary loss of support factor that results in additional slab thickness requirement, but also added a drainage factor that when applied could result in an additional slab thickness requirement.

A pavement with better subdrainage than the AASHO Road Test pavements had may also have improved support over time and may perform better from a cracking standpoint. Presumably, a C_d factor greater than 1.0 reflects this benefit, but the 1986 AASHTO *Guide* does not specifically state what the C_d is intended to adjust: cracking, faulting or some other distress. Because faulting did not occur at the Road Test, the C_d obviously could not be used to improve on faulting.

Joint load transfer factor, J. The reference value of 3.2 for *J* is a constant from an equation derived by Spangler for stress due to loading at unprotected (free) corner conditions, based on slab theory and laboratory test results. The corner stresses (computed from measured strains) actually experienced by the AASHO Road Test pavements were linearly correlated to the free corner stress predicted by Spangler's equation. (The actual magnitude of the corner stresses in the dowelled Road Test pavements was about one third of the magnitude predicted for free corner conditions by Spangler's equation.) Thus, by incorporating Spangler's equation into the AASHO design model (that is, calibrating it to the Road Test pavement stresses), the *J*-value of 3.2 was made to represent a protected (dowelled) joint and no tied shoulders, as existed at the Road Test.

A value greater than 3.2 means higher tensile stress at the top of the slab is expected due to corner loading because the joint load transfer is less than dowels would provide. A value less than 3.2 means the joint has better load transfer than dowels would provide, from improved joint load transfer (e.g., CRCP or the addition of a tied concrete shoulder). It is very important to remember that the *J*-factor is an adjustment for slab stresses that cause corner breaks, and has absolutely nothing to do with joint faulting. No joint faulting existed at the Road Test. One cannot design a reduction or an increase in joint faulting by changing the *J*-factor. This has been a point of major confusion among pavement engineers for years.

It is also important to realize that Spangler's corner equation considers only load stress for a flat corner and does not include thermal or moisture gradients that cause upward curl and warp of the corner. Different climates or construction methods that result in curling or warping magnitudes different than those that occurred at the Road Test are not considered in the AASHTO design model.

Deficiencies in 1993 AASHTO Procedure Related to Pavement Support

The following summary is a list of the specific deficiencies that were found to exist in the current version of the AASHTO design procedure for concrete pavements that are related to pavement support.

- The gross *k*-value input assumes a large amount of permanent deformation and does not represent the support that the pavements actually experience during traffic loading. An elastic *k*-value provides a far more realistic match to measured strains. In analysis of AASHO Road Test pavements, the elastic *k*-value was found to reduce the stress in the slab equal to that computed from measured strains under creep speed axle loading, as shown in Appendix D.
- The lowest gross *k*-value that was measured on top of the base during the spring (60 psi/in. [16 kPa/mm]) was incorporated into the AASHTO model in 1961 and has not been changed. The 1986 version provided a procedure to consider seasonal variation in selection of a design *k*-value; however, the design equation was not modified to incorporate the effective *k*-value that existed at the Road Test site. Thus, the current seasonal adjustment procedure is incompatible with the current design model.
- The effect of the base course on performance is not properly considered through the composite top-of-the-base *k*-value. This is especially true for stiff treated bases that act as structural layers in reducing stress in the slab. An improved way to model the effect of the base layer on slab stress is needed.
- Substantial loss of support existed for many sections at the AASHO site, which led to increased slab cracking and loss of serviceability; thus, the performance data and design equation already incorporate considerable loss of support. Incorporation of an additional loss of support factor results in overdesign. What is needed is a way to consider the benefit of an improved base on performance in terms of cracking and faulting.
- The 1961 extension used Spangler's unprotected corner equation. The critical stress location at the AASHO Road test was along the slab edge for slabs 6.5 in. [165 mm] and greater, and resulted in transverse fatigue cracks initiating at the bottom of the slab. The stresses in the vicinity of the corner were much lower than those at midslab due to the well-dowelled joints. Use of Spangler's corner equation with dowelled joints does not model the critical stress and crack initiation location, and thus cannot possibly provide accurate indications of the effect of slab support on cracking, especially when thermal curling and moisture warping are considered.
- The current AASHTO procedure does not provide a methodology to design a pavement with undowelled joints. The *J*-factor only considers tensile stress that controls cracking, not faulting. An undowelled joint requires improved slab support from the base and a more erosion-resistant base material to prevent loss of support over time and premature failure. Thermal curling and moisture warping, which become much more critical to performance with undowelled joints, are not considered in the current AASHTO procedure.
- Joint spacing other than that of the Road Test slabs is not considered at all in the current design procedure. It is known from many other studies that joint spacing has a major effect on slab cracking and faulting (*12,56*). Subgrade and base support interact with joint spacing to affect combined slab stresses from load, temperature, and moisture gradients. Thus, slab support is a very important variable in the selection of joint spacing to minimize transverse cracking.
- The original 1961 model reflects the climate of the

AASHO Road Test site only. The 1993 version does not include any variable that adjusts for different climates. Thus, other climates that cause different magnitudes of slab curling or warping cannot be considered. This limitation alone has led to many pavement failures from premature cracking.

- The only distress manifestation considered directly by the design procedure is transverse slab cracking because that is basically the only distress which occurred at the Road Test (other than erosion and loss of support which contributes to slab cracking). Thus, the loss of serviceability was due almost entirely to slab cracking and the subsequent deterioration of those cracks resulting in roughness and loss of serviceability. Some sections had excessive loss of support prior to failure from slab cracking. Cracking is related to slab support, and the Spangler corner equation incorporated into the AASHO design equation is not a realistic model for predicting the cracking that occurred, as noted above.
- Faulting of transverse joints did not occur during the 2 years of the Road Test because the joints all had dowels; thus, the performance predicted by the design model does not consider the effect of faulting on loss of serviceability. The *J*-factor, often thought to control faulting, has nothing to do with joint faulting.
- Although thermal curling and moisture warping of slabs occurred during the 2-year Road Test, the effects of these important factors were not considered in any of the extensions. This is important because any design feature that would increase stresses from either of these stresses cannot be considered in design of that pavement. For example, joint spacing, base stiffness, and subgrade stiffness all affect stresses from thermal curling and moisture gradients through the slab. None of these can be considered in pavement design using the current AASHTO *Guide* procedure.

Given these major deficiencies, the following sections describe the research and development efforts that led to a recommended improved methodology for better consideration of slab support in the AASHTO design procedure.

Improved AASHTO Methodology Recommended

Improved technology exists today that was not available in 1961, including the capabilities of 3-D finite element models to compute slab stresses, larger and faster computers, and advanced mechanistic and statistical modeling. This technology was applied to the original AASHO model to develop an extended and improved design model for concrete pavements that more fully considers pavement "support" aspects. Specific improvements in the proposed revision to the AASHTO design procedure include the following:

1. Defining the *k*-value specifically as the value determined on the finished roadbed soil or embankment, upon which the base and slab will eventually be constructed. A composite top-of-the-base *k*-value is not valid and is not recommended for design.
2. The *k*-value input recommended is the elastic *k*-value as tested extensively at the AASHO Road Test and similarly at the Arlington test site. The elastic *k*-value was found to result in slab stresses similar to those produced in the field by axle loads at creep speed (see Appendix D).

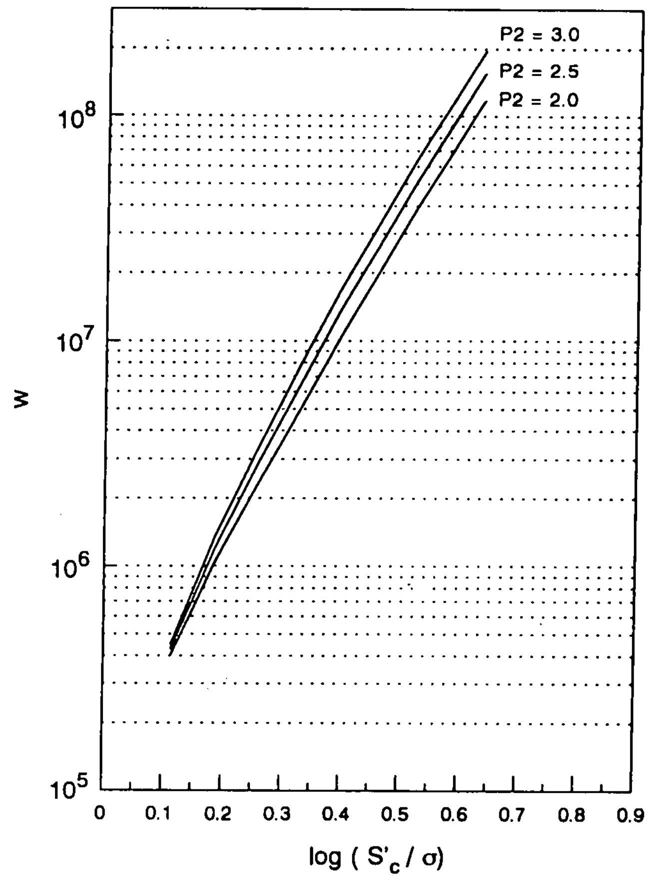


Figure 14. Relationship of W to log S_c'/σ for three terminal serviceability levels for the proposed revised AASHTO extended concrete pavement design model.

3. Seasonal support variations are considered through the determination of an effective yearly elastic *k*-value of the embankment/subgrade (Appendix E). A procedure was developed to determine the effective *k*-value for design.
4. The effect of the base course on slab stress due to load and temperature and moisture gradients is directly considered. The base thickness, stiffness, and friction coefficient (between the slab and the base) are direct inputs to the design procedure.
5. Temperature gradients and moisture gradients (as equivalent temperature gradients) are directly considered as inputs to the design procedure.
6. A procedure was developed for checking joint faulting and adjusting joint design if deficient, rather than increasing slab thickness.
7. Joint spacing is directly considered through consideration of its interaction with slab support and effect on combined load and temperature curling stresses.
8. The effects of longitudinal edge load transfer or a widened traffic lane on critical stress reduction are considered directly.
9. Joint (corner) load position stresses are checked for undowelled joints in slab design.

A new design model for concrete pavement design was developed using the same general approach used in 1961 to extend the original empirical model and also incorporate the above capabilities. Figure 14 shows this mechanistic-empirical type of

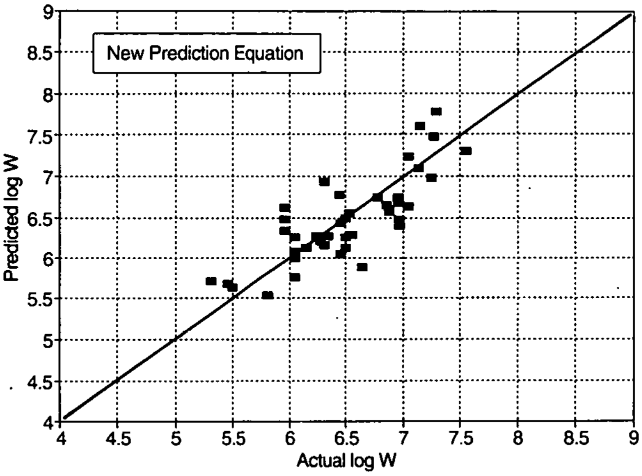


Figure 15. Predicted versus actual log W for test sections from the 2-year AASHO Road Test, the extended I-80 tests, and the FHWA database using the proposed revised concrete pavement design model.

model, in which log W is linearly related to the logarithm of the strength-to-stress ratio S'_c/σ. The new concrete pavement log W model (for 50 percent reliability) was obtained by combining the empirical model and the mechanistic-empirical model as follows (see Appendix E for derivation):

$$\log W' = \log W + (5.065 - 0.03295\, P2^{2.4}) * \left[\log\left(\frac{S'_c}{\sigma'}\right) - \log\left(\frac{690}{\sigma}\right)\right] \quad (1)$$

where

W' = number of design 18-kip equivalent single-axle loads (ESALs) in traffic lane

σ' = maximum tensile slab stress for the midslab load position due to combined load and effective temperature curl (with inputs for the new pavement design)

W = number of 18-kip ESALs estimated using the original empirical AASHO design model from the main loops (with inputs from original AASHO Road Test)

σ = Maximum tensile slab stress for the midslab load position due to combined load and effective temperature curl (with inputs from original AASHO Road Test)

Equation 1 represents the best fit relationship between design features and log W. Reliability can be added in a manner similar to that in the current AASHTO *Guide*.

Field Verification of New Models

Data were obtained from the 14-year extended AASHO Road Test (*69*) and the RPPR database (*56*). This database provides performance data from sections with various base types, subgrades, climates, and designs from many states. The number of 18-kip (40-kN) ESALs (log W) were predicted from the initial serviceability ($P1$) to the current serviceability ($P2$). The actual number of ESALs were computed from the traffic data on each section. The results shown in Figure 15 indicate a reasonable prediction of log W for a wide variety of pavement designs across the United States with no particular bias of over or under prediction.

Sensitivity of Proposed New AASHTO Concrete Pavement Design Model

Sensitivity analyses were conducted to show the relative effects of the design features, particularly those related to pavement support, on slab thickness and traffic load-carrying capacity. A standard pavement design section, described below, was used to conduct the sensitivity analyses:

Standard Pavement Section Features

P1 = 4.5
P2 = 2.5
Slab thickness D = 9 in. (229 mm), also 11 in. (279 mm)
Slab elastic modulus E_c = 4,200,000 psi (28938 MPa)
Base thickness = 5 in. (127 mm), also 4 and 6 in. (102 and 152 mm)
Base elastic modulus E_b = Aggregate, 25,000 psi (172 MPa), also Treated, 500,000 and 1 million psi (3445 and 6890 MPa)
k = 100 psi/in. (27 kPa/mm), also 250 and 400 psi/in. (67.5 and 108 kPa/mm)
Joint spacing L = 15 ft (4.6 m), also 10 and 20 ft (3.05 and 6.1 m)
Concrete flexural strength S'_c = 690 psi (4754 kPa)
Effective temperature gradient DT = Varies with slab thickness; Urbana IL climate
Base friction coefficient f = Varies with base type (modulus);
1.5 for E_b = 25,000 psi (172 MPa)
6 for E_b = 500,000 psi (3445 MPa)
35 for E_b = 1 million psi (6890 MPa)
PCC Shoulder = No
Widened Lane = No

Effect of Subgrade k-Value and Base Stiffness

The effects of the subgrade stiffness (k-value) and base stiffness (E_b) on load-carrying capacity are complex, as they strongly interact with each other and other design features. Table 6 illustrates some results from varying subgrade k-value and base elastic moduli (with corresponding variation in slab/base friction). The friction coefficient used for each base type is shown in

TABLE 6. Sensitivity analysis for *k*-value and base modulus

Base Modulus (psi)	k value (psi/in)		
	100	250	400
25,000 (f = 1.5)	13	15	16
500,000 (f = 6)	20	20	20
1,000,000 (f = 35)	24	20	20

Values shown in table are 18-kip [80 kN] ESALs, millions
1 psi = 6.89 kPa, 1 psi/in = 0.27 kPa/mm

TABLE 7. Sensitivity analysis for base thickness and *k*-value

Base Modulus (psi)	Base Thickness (in)	k value (psi/in)		
		100	250	400
25,000	4	13	15	16
25,000	6	13	15	17
1,000,000	4	20	18	18
1,000,000	6	28	21	23

Values shown in table are 18-kip [80 kN] ESALs, millions
1 psi = 6.89 kPa, 1 in = 25.4 mm, 1 psi/in = 0.27 kPa/mm

Table 6 and represents a typical value of peak frictional resistance.

These results show that on a soft subgrade (k = 100 psi/in. [27 kPa/mm]), changing from an aggregate base to a treated base produces a large increase in the load-carrying capacity (13 to 24 million ESALs). The stiffer the subgrade, the less the effect of base stiffness on load-carrying capacity.

Given an aggregate base (E_b = 25,000 psi [172 MPa]), an increased subgrade stiffness results in some increase in load-carrying capacity. Given a stiffer treated base, however, the stiffness of the subgrade has little effect on load-carrying capacity.

The coefficient of friction does not appear to have much of an effect on the load-carrying capacity for any given base type, unless a very high value is used. For example, given a treated base with modulus E_b = 1,000,000 psi (6890 MPa), a change in the friction coefficient from 1.5 (low friction) to 35 (typical for cement-treated base) results in vey little change in the ESALs. Thus, it is not necessary to quantify the friction coefficient f with great precision.

Effect of Base Thickness and Embankment Stiffness

The effect of base thickness is shown in Table 7 for ranges of subgrade stiffness and base stiffness. Base thickness does not affect traffic life when the base has a low modulus (untreated aggregate base). This same conclusion was reached at the Road Test, where "the effect on performance of varying the thickness of the subbase between 3 and 9 in was not significant . . ." (*32*). However, when the base is treated and has a modulus such as 1,000,000 psi (6890 MPa), the thickness has a very significant effect as shown. The effect of base thickness is less with stiffer subgrades.

Effect of No Base

Most of the AASHO Road Test sections had a dense-graded aggregate base course. The thickness of the base did not influence the load-carrying capacity of the pavements. Some sections, however, were constructed directly on the silty-clay soil subgrade. Analysis of the performance of these sections led the AASHO research staff to conclude that "sections with subbase had an average life about one third longer than that of sections without subbase" (*32*). The increase in life between sections with base and matching sections without base ranged from 0 to 100 percent.

The proposed revision to the AASHTO design procedure includes thickness of base as an input. The results shown in Table 8 were obtained for pavements with no base and pavements with a 6-in. (152-mm) aggregate base. The results are similar to those obtained at the AASHO Road Test: for k = 100 psi/in. (27 kPa/mm) the predicted traffic life of sections with a granular base are about 35 to 50 percent greater than similar sections with no base.

TABLE 8. Sensitivity analysis for slab thickness, aggregate base thickness and *k*-value

Slab Thickness (in)	Aggregate Base Thickness (in)	k value (psi/in)		
		100	250	400
9	0	9	11	12
9	6	13	18	17
11	0	37	35	32
11	6	51	45	41

Values shown in table are 18-kip [80 kN] ESALs, millions
1 in = 25.4 mm, 1 psi/in = 0.27 kPa/mm

TABLE 9. Sensitivity analysis for joint spacing and *k*-value for aggregate base

Joint Spacing (ft)	k value (psi/in)		
	100	250	400
10	15	20	23
15	13	15	16
20	10	11	11

Values shown in table are 18-kip [80 kN] ESALs, millions
1 ft = 0.305 m, 1 psi/in = 0.27 kPa/mm

Effect of Joint Spacing

As joint spacing increases, stresses due to thermal curling and moisture warping increase. The JPCP at the Road Test had a 15-ft [4.6-m] joint spacing and none of the extensions to the empirical design model included curling or warping stresses. Table 9 shows some results for the standard pavement section.

These results show that for any level of subgrade stiffness, the number of load applications decreases as joint spacing increases. The results also show that as subgrade stiffness increases for a shorter joint spacing, the traffic applications to terminal serviceability increases. However, for longer (20-ft [6.1-m]) joint spacing the traffic remains about the same. These results again show the significant interactions between support and design features.

Alternative Designs

The proposed revised design procedure permits the comparison of a variety of pavement designs that are supposed to show approximately the same traffic life. Several of these designs are shown in Table 10. All of these designs meet the same design ESAL traffic level for the standard example (13 million).

As joint spacing increases from 15 to 20 ft (4.6 to 6.1 m), the required slab thickness increases from 9.0 to 9.3 in. (229 to 236 mm) for the untreated aggregate base and from 8.2 to 8.7 in. (208 to 221 mm) for the treated base. Use of a 5-in. (127-mm) treated base in lieu of a 5-in. (127-mm) aggregate base decreases required slab thickness by about 0.8 in. (20 mm) for the 15 ft (4.6 m) joint spacing.

Effect of Climate

The effect of climate was brought directly into the proposed revised AASHTO design model through consideration of a positive (daytime) effective temperature gradient from top to bottom of the slab. Values were computed for various locations across the United States and the results show that certain areas of the United States have much more critical values than the AASHO Road Test site. Note that the negative temperature gradient and moisture gradient was brought into the design through the joint loading position check described later.

Given the standard pavement section and a given design traffic, the results in Table 11 represent the effect of different climates on slab thickness at 50 percent reliability, holding all other inputs constant. As the effective temperature differential inceases, slab thickness must be increased if joint spacing is held constant to carry the same amount of traffic.

A similar illustration could be made by holding slab thickness constant at 9 in. (229 mm) (where practical) and varying joint spacing for the different climates. Table 12 shows that as the effective temperature differential increases for different regions, the joint spacing required to control curling stresses (to achieve the same loss in serviceability) decreases.

TABLE 10. JPCP pavement designs using proposed revised design model for 50 percent reliability, 13 million ESALs, k = 100 psi/in. (27 kPa/mm)

Slab Thickness Required (in)	Base Type	Base Thickness (in)	Joint Spacing (ft)
9.0	Aggregate (E_b = 25,000 psi)	5	15
9.3	Aggregate (E_b = 25,000 psi)	5	20
8.2	Treated (E_b = 1,000,000 psi)	5	15
8.7	Treated (E_b = 1,000,000 psi)	5	20

1 in = 25.4 mm, 1 psi = 6.89 kPa, 1 ft = 0.305 m

TABLE 11. Slab thickness and joint spacing required for various effective temperature differentials existing at various locations

Location	Effective Temperature Differential* (°F)	Slab Thickness Required (in)	Joint Spacing Required (ft)
Syracuse, NY	5.2	8.6	15
Salem, OR	6.4	8.8	15
Raleigh, NC	7.2	8.9	15
Urbana, IL	7.9	9.0	15
Tallahassee, FL	8.7	9.2	15
Sacramento, CA	10.0	9.3	15
Las Vegas, NV	11.5	9.6	15

* Effective temperature differential between top and bottom of 9 in [229 mm] slab
1°F = 0.55°C, 1 in = 25.4 mm, 1 ft = 0.305 m

Comparison with 1993 AASHTO Guide

A comparison was made to see how the proposed revised design procedure compared with the existing AASHTO procedure. The standard pavement section defined previously was examined over a range of traffic loadings to make this comparison.

Table 13 shows the comparison for a pavement located at the AASHO site but having a stiff (E_b = 1,000,000 psi [6890 MPa]) treated aggregate base course. The 1993 AASHTO method handles this case by increasing the k-value on top of the base. A "composite" k of 400 psi/in. (108 kPa/mm) was determined from the AASHTO *Guide's* nomograph for base thickness = 5 in. (127 mm), base E = 1,000,000 psi (6890 MPa), and subgrade M_R = 3000 psi (20.7 MPa). Two sets of results are shown: the first set of thicknesses obtained using the composite k of 400 psi/in. (108 kPa/mm), assuming no loss of support, and the second set of thicknesses obtained using a k-value of 130 psi/in. (35 kPa/mm), the value which would be obtained by reducing the composite k-value for loss of support using LS = 1.0.

The proposed revised procedure handles the case of a treated-base course by considering the stiff base as a structural layer, with a degree of friction between the slab and base which is appropriate for the base type (see Appendix E), and a k-value input that represents the embankment beneath the base. The results show the thicknesses to be closer to the composite k-value with no loss of support, except at very high traffic levels.

However, many different comparisons could be made. For example, the existing AASHTO design does not consider the effect of joint spacing. A second comparison was made for a design ESAL of 50 million. The existing AASHTO procedure would require the same thickness for any joint spacing, whereas the proposed revised procedure would require a thicker slab for a longer joint spacing (or conversely a shorter joint spacing for a thinner slab) to provide the same performance. This is commonly understood and provided for in various design manuals that thicker slabs can have somewhat longer joint spacings due to the reduction in thermal and moisture gradient effects. Table 14 illustrates some results for this example. An increase in joint spacing requires an increase in slab thickness.

TABLE 12. Joint spacing and slab thickness required for varying effective temperature differential

Location	Effective Temperature Differential (°F)	Slab Thickness Required (in)	Joint Spacing Required (ft)
Syracuse, NY	5.2	8.8*	20 (max)
Salem, OR	6.4	9.0*	20 (max)
Raleigh, NC	7.2	9.0	17
Urbana, IL	7.9	9.0	15
Tallahassee, FL	8.7	9.0	12.5
Sacramento, CA	10.0	9.1*	12 (min)
Las Vegas, NV	11.5	9.4*	12 (min)

* Minimum and maximum recommended joint spacings are 12 and 20 ft [3.7 and 6.1 m] respectively. Slab thicknesses with asterisks were adjusted to meet these joint spacing limits.

1°F = 0.55°C, 1 in = 25.4 mm, 1 ft = 0.305 m

TABLE 13. Comparison of slab thickness required for 1993 AASHTO *Guide* procedure and proposed revised design procedure for pavement with treated aggregate base

Design ESALs (millions)	1993 AASHTO Guide*		Proposed Design Model**
	k = 400 psi/in	k = 130 psi/in	
5	6.6	7.4	6.6
10	7.6	8.3	7.8
20	8.7	9.3	8.9
50	10.1	10.7	10.1
100	11.3	11.9	11.1
200	12.6	13.2	12.1

Values shown in table are slab thicknesses, inches
1 in = 25.4 mm, 1 psi/in = 0.27 kPa/mm

* AASHTO inputs: k = 400 psi/in (composite top-of-base springtime value with no loss of support), and k = 130 psi/in (composite top-of-base springtime value with loss of support = 1.0)

** Revised Inputs: k = 110 psi/in (seasonally adjusted embankment value incorporated into equation for AASHO Road Test site)

Other inputs: 5-in [127 mm] base, E_b = 1 million psi [6890 MPa], Friction coefficient = 35, Design reliability = 50 percent

Design Tables

The proposed revised design equations are too complex to put into nomograph form as was done with the original equation in 1961 and later versions. However, the design equations can easily be solved in a spreadsheet or computer program. Tables were prepared that make it possible to quickly determine the required design thickness for a wide range of design features and climate.

Note that the design examples presented in the preceding sensitivity analysis were developed assuming that there will be no significant joint faulting. Because no faulting occurred during the AASHO Road Test, a fair comparison of the existing AASHTO model and the proposed revised model should be

TABLE 14. Comparison of slab thickness required for 1993 AASHTO *Guide* procedure and proposed revised design procedure for a range of joint spacings

Joint Spacing (ft)	1993 AASHTO Guide*		Proposed Design Model**
	k = 400 psi/in	k = 130 psi/in	
15	10.1	10.7	10.1
20	10.1	10.7	10.7
25*	10.1	10.7	11.3

Values shown in table are slab thicknesses, inches
1 in = 25.4 mm, 1 psi/in = 0.27 kPa/mm
Inputs same as specified for Table 13
Design ESAL = 50 million
* A joint spacing greater than 20 ft [6.1 m] is not recommended for this range of slab thickness but these are the values obtained.

made considering only loss of serviceability due to cracking that occurs as a function of stress repetitions. In reality, jointed concrete pavements in service for several years may develop faulting as well, which further reduces the serviceability and thereby reduces the number of axle loads that the pavement can carry. The next section discusses this aspect of performance and provides a design check for faulting.

Design of Joint Load Transfer to Control Faulting

Because all joints were adequately dowelled at the Road Test, no significant faulting occurred during the 2-year test. If the joints were not properly dowelled, a large amount of faulting would have occurred. According to the Road Test report:

> One joint faulted seriously, but investigation showed that the joint had been accidentally sawed at some distance beyond the end of the dowels intended to protect it. Over the 2-year period of the test there were no other cases of measurable faulting at joints, all of which were dowelled. (*32*)

Faulting is one of the most important distresses affecting rideability and serviceability of jointed concrete pavements. Therefore, any pavement that faults significantly will have reduced serviceability and carry fewer traffic loads to terminal serviceability. The current (1993) AASHTO design concept is to design for different load transfer levels by selection of the J-factor. A higher J-factor will result in an increase in slab thickness according to the 1993 AASHTO equation. However, field studies have demonstrated that slab thickness does not affect faulting significantly (*56,69*). Thus, joints must be prevented from significant faulting through good joint load transfer, joint spacing, base design, and subdrainage design, not slab thickness design. The following procedure is recommended to determine the adequacy of a proposed joint load transfer design.

STEP 1: Initial Slab Thickness Design. Develop an initial slab thickness design. Note that the degree of joint load transfer is not an input at this stage since the midslab load position is used. However, joint spacing, base properties (type, thickness, stiffness, friction), and other design features must be chosen for the slab thickness design. These can be modified later if necessary and a redesign made to achieve a better joint design.

STEP 2: Initial Joint Design. Develop an initial joint design, including the following: base type, joint spacing, subdrainage presence, and dowel diameter, if dowels are to be used.

STEP 3: Joint Faulting Prediction. Mean joint faulting is predicted using models in Reference 56, and the adequacy of the design to control faulting below an acceptable level is evaluated.

Tables were prepared to show the faulting predictions for pavements with and without dowel bars. The mean joint faulting is predicted and compared with recommended critical levels. If the predicted faulting is greater than the recommended level, an adjustment to joint design is made. Adjustments include use of dowels, or if dowels already exist, an increase in the diameter, selection of a different base type and permeability, and a decrease in the joint spacing (for undowelled joints). Slab thickness is not adjusted because it has only a minimal effect on joint faulting.

Design of the Base Course

The base course is considered a structural layer in the proposed revision to the AASHTO design procedure, as opposed to the current AASHTO procedure in which the base is considered a part of the foundation and thus affects the k-value input. In the proposed revision to the design procedure, a coefficient of friction between the slab and base is also an input. An equation was developed, using the results of many 3DPAVE runs, for slab stress due to midslab loading assuming full friction (i.e., a "bonded" interface). The stress in the slab due to a degree of friction less than full friction is computed by multiplying the full friction stress by an adjustment factor which is a function of the slab thickness, base modulus, and friction coefficient. The equation for the friction adjustment factor was also developed using the results of a factorial of 3DPAVE runs.

Ranges of values for friction coefficients for a variety of base types and interface treatments are given in Appendix E, summarized from the available literature. Most of the available data on slab/base interface friction comes from laboratory tests in which small-scale concrete slabs are constructed on bases and pushed horizontally with a measured force. The vertical force in such tests is generally only the weight of the slab. How well such tests represent field conditions, in which full-scale slabs also bend under the weight of applied loads, is not clear. It is also not clear at this time how much the friction coefficient of various bases and interface treatments changes over time and how a coefficient that best represents the typical value over the service life of the pavement should be selected. The addition of a friction coefficient as an explicit input to the slab thickness design process is believed to be a significant improvement to the AASHTO design procedure and a significant need in any mechanistic design procedure. However, long-term field performance studies data essential to better establish how slab/base interface friction over the life of the pavement should be characterized.

Friction between the slab and base affects the amount of erosion between the layers also. A high degree of friction will greatly reduce or eliminate erosion between the slab and the base. Reasonable friction of the slab to almost any type of base course can be achieved without extraordinary means. To avoid reflection cracking in a slab due to high friction with a cement-treated or lean concrete base, transverse and longitudinal joints should be cut in the base to a depth of approximately one fourth of the thickness of the base prior to slab placement. Joints are not needed in asphalt-treated bases.

The friction between the slab and base is also an input to the calculation of stress due to joint (corner) loading. A check on this stress is included in the proposed revision to the design procedure to identify situations in which the corner loading position might control the design. This is described in the next section.

The effect of the base is directly considered in joint design as well as the slab thickness design. If the initial joint design and base features are not adequate to control faulting below a design value, revisions must be made to the load transfer system and/or the base and subdrainage.

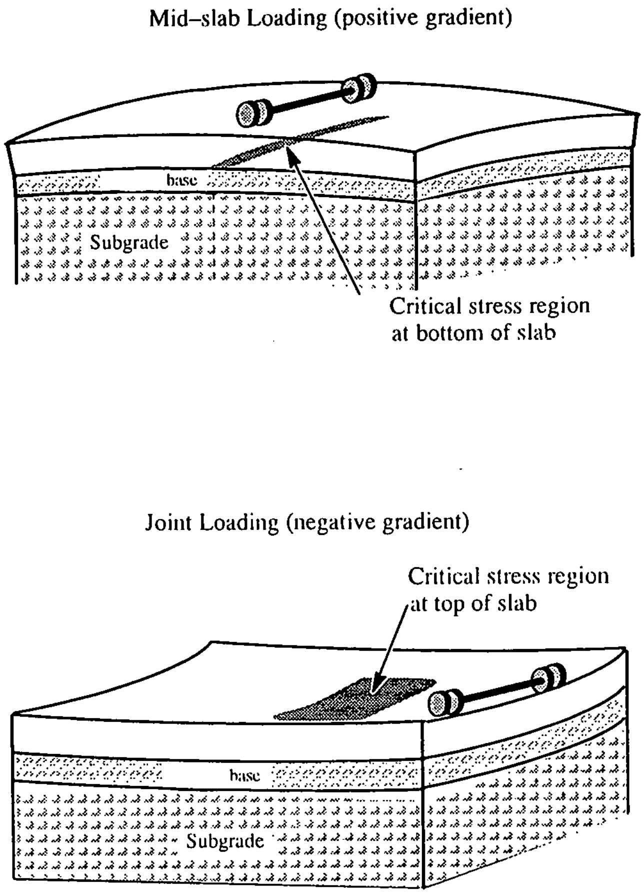


Figure 16. Critical location of maximum tensile stresses for the midslab load position and joint load position.

Design Check for Critical Joint Load Position Stresses

The proposed revision to the AASHTO procedure uses the midslab loading position shown in Figure 16, because this was the critical position (maximum stress) at the AASHO Road Test. This occurred because all of the transverse joints were well dowelled to provide good load transfer. Strain measurements from Loop 1 showed that the maximum stress in the slab occurred "along the pavement edge with the center of the outer loaded area at the distance of 1 ft (0.3 m) from the edge and 4 to 6 ft (1.2 to 1.8 m) from the nearest transverse joint" (*70*). This maximum stress approximately matched the stress computed from strain measurements at the slab edge for midslab loading from moving truck axles (30 mph [48 km/hr]):

> For a constant axle weight and slab thickness, it was estimated that the maximum compressive stress at the (top) edge due to edge loading exceeded, in absolute value, the maximum tensile stress due to corner loading by 51 to 112 percent. The exact percentage depended on the thickness of the slab (*32*).

Similar results were obtained from 3DPAVE for the dowelled AASHO Road Test pavements. For dowelled load transfer and typical positive and negative temperature differentials appropriate for the AASHO Road Test site, the maximum stress is much greater for the midslab loading position. Figure 16 shows the maximum critical stress region at the bottom of the slab for loading combined with a positive temperature differential. This explains why corner breaks are almost never observed for properly dowelled joints, as many pavement surveys have shown (*56,69*).

However, cracks often do occur near the joints in pavements with no mechanical load transfer such as dowels. Under certain design and climatic conditions, truck axle loadings near the transverse joint (see Figure 16) may produce even higher tensile stresses in the slab than the midslab load position. These high tensile stresses could result in corner breaks, diagonal cracks, or even transverse cracks several feet from the joint. This mechanism has been well analyzed and described by Poblete et al. for undowelled jointed plain pavements in Chile (*61,62*).

Figure 16 shows the critical stress region on top of the slab, found using 3DPAVE, to be the maximum critical tensile stresses for the joint loading position when no load transfer exists at the transverse joint. With load transfer, these stresses decrease significantly to values less than those obtained for the midslab position.

Given these findings and concerns, a design check for the joint loading position with negative equivalent temperature differentials was developed. The joint loading position shown in Figure 16 requires a different analysis due to the additive effects of the following contributors to slab stresses.

- AXLE LOAD STRESS: When the axle load is near the transverse joint a tensile stress occurs at the top of the slab.
- NEGATIVE TEMPERATURE DIFFERENTIAL STRESS: Negative (nighttime) temperature differentials cause corners to curl upward, creating a tensile stress at the slab surface. An effective negative temperature differential stress was computed for each climatic site using a procedure similar to that used for the daytime positive temperature differentials.
- CONSTRUCTION CURLING STRESS: Upward curling of corners occurs shortly after concrete slab placement if a high positive temperature differential through the slab is present as the concrete sets (*62,64*). This positive differential occurs particularly on sunny days when conventional curing procedures are used. This temperature differential has not been measured extensively in the past and its magnitude is not well known at the present time (*62,64*). This is defined as the temperature differential that would be required to produce a flat slab (note that this is before any moisture shrinkage occurred at the top of the slab).
- MOISTURE GRADIENT STRESS: Moisture shrinkage warping of the top of the slab occurs over time (*62,64,67,68*). The stress induced by this type of warping can be determined by representing the moisture warping by an equivalent temperature gradient (see Appendix C).

Applied loads and the three climatic factors previously described can lead to large tensile stresses at the top of the slab near the joint. Combined stresses from negative temperature differentials and from load can be estimated using 3DPAVE.

Many jointed plain concrete pavements without dowel bars or other mechanical load transfer devices have been constructed in the United States and other countries. These pavements are often built in warm dry climates (e.g., the western United States, Chile, Spain) where the potential for construction curling and moisture shrinkage warping is greater. When the joints are open in cooler weather, the degree of load transfer at the joints provided by aggregate interlock is very low.

Analyses were conducted using 3DPAVE of pavement sections no load transfer, loaded at the joint position. The maximum stress in the slab due to load and temperature differential was computed and plotted as shown in Figure 11 for a range of design features. The results showed that under conditions of extreme negative temperature differential and poor load transfer, the tensile stresses due to joint loading can equal or exceed stresses due to midslab loading.

A procedure was developed to check for critical stress for the joint loading position for pavements that do not have mechanical load transfer devices equivalent to dowel bars. Pavements that have adequate load transfer devices such as properly sized and spaced dowels would not experience significantly high stress at the joint. The slab stress design check is accomplished by comparing the stress obtained for the midslab loading position with the stress obtained for the joint loading position. If the joint loading stress is greater, then corner breaks, diagonal cracks, and even transverse cracks may initiate at the top of the slab first. The higher of the two stresses is the one which should be used in the slab thickness design.

Design features that provide a defense against critical joint loading stresses are the use of properly sized and spaced dowels and to a lesser degree, a widened slab (i.e., slab paved wider than 12 ft [3.7 m] but traffic lane striped 12 ft [3.7 m] wide), or a tied concrete shoulder. The other effect that good load transfer has on performance is that corner deflections are reduced. High differential deflections at the corner can lead to erosion and loss of support, resulting in even greater stresses under corner loading.

Design of Different Types of Concrete Pavement

The current AASHTO *Guide* does not distinguish between JPCP, JRCP, and CRCP as far as thickness design is concerned. The proposed revision to the AASHTO procedure applies specifically to JPCP with relatively short joint spacing. Required slab thickness for JRCP and CRCP may be different; however, in keeping with current AASHTO design philosophy, the thickness designed for JPCP should be adequate for JRCP and CRCP.

Summary Comparison of Existing AASHTO Design Procedure and Proposed Revised Procedure with Improved Support Considerations

Table 15 summarizes the key differences between the way in which design features related to pavement support are handled in the current AASHTO procedure and the proposed revised procedure developed under NCHRP Project 1-30.

TABLE 15. Summary of comparison between existing and proposed revised design considerations

Design Feature	Existing AASHTO Procedure	NCHRP 1-30 Proposed Revision
Subgrade Support	Gross k value required, lowest springtime value incorporated into equation, NOT seasonally adjusted k value. Effect of subgrade stiffness not considered in thermal curling stresses in slab.	Elastic k value of subgrade, seasonal adjustment if needed. Subgrade stiffness directly considered in slab design for load and thermal curling stresses. Brings climate into design process. Ability to estimate k value for variety of soils and bedrock.
Base Course	Considered only through a composite (top-of-base) k value. Base stiffness and friction are not considered in load or curling stresses in slab.	Direct consideration of base as structural layer (thickness, stiffness and friction). Effect of base on both load and thermal curling stresses.
Joint Spacing	Built-in 15 ft [4.6 m] JPCP. Built-in 40 ft [12.2 m] JRCP. Not considered otherwise.	Direct consideration of joint spacing effect on load and curl stresses. Brings climate into design process.
Climatic Effects	AASHO site climate built into design model. Only adjustment is through seasonal composite k value. Other climates (temperature differentials) not considered.	Seasonal variation of subgrade elastic k value possible through effective k procedure. Effective temperature differentials can be determined for climates different than AASHO site.
Seasonal Variation in Support	Seasonal adjustment is possible using effective k value method, but adjustment is inconsistent with lowest springtime gross k value built into model.	Seasonally adjusted AASHO site effective k value built into design model. Seasonal adjustment possible for other locations.
Loss of Support	Substantial loss of support built into existing model. Additional reduction of k value for loss of support is overdesign.	Substantial loss of support built into model from AASHO site, no further adjustment needed.
Joint Faulting	Not considered at all in current procedure. Mistakenly thought to be considered through J factor, which results in increased slab thickness, not improved joint design or reduced faulting.	Faulting checked after slab thickness design completed. If joint design is inadequate, joint design and/or base changes allowed, but not slab thickness increase.
Joint Load Transfer	Dowelled joints built into existing model. J factor attempts to adjust corner stress for more or less load transfer. No way to consider curling or warping of corners, especially for undowelled joints.	Effect of joint load transfer on corner load, curl and moisture gradient stresses for undowelled joints is checked directly.
Widened Slab, Tied Shoulders	Inadequate stress adjustment through J factor.	Direct adjustment of critical stress through consideration of longitudinal load transfer.

CHAPTER 3

INTERPRETATION, APPRAISAL, AND APPLICATION

Many practical insights and procedures were identified and developed in this research study into support for concrete pavements. This chapter summarizes the practical guidelines for improving the consideration of support in concrete pavement design. The guidelines for selection of a design k-value are applicable to both the AASHTO *Guide* design methodology and a mechanistic design methodology.

Specific guidelines were also developed for improved consideration of support in slab thickness design and joint design in the AASHTO methodology. Proposed revisions to the relevant portions of the AASHTO *Guide* are provided in the appendixes. Finally, guidelines for improved support considerations in mechanistic design of concrete pavements are provided.

IMPROVED GUIDELINES FOR SELECTION OF DESIGN *k*-VALUE

The following guidelines for selection of k-value were developed in this study:

- Selection of values for fine-grained soils as a function of AASHTO or Unified soil class and degree of saturation;
- Selection of k-values for coarse-grained soils as a function of AASHTO or Unified soil class and density;
- Correlations of k-value to California Bearing Ratio (CBR), Hveem Stabilometer (R-value), and Dynamic Cone Penetrometer (DCP);
- Adjustment to k-value for height and density of embankment above the subgrade;
- Adjustment to k-value for a rigid layer within 10 ft (3 m) of the surface of the subgrade; and
- Adjustment to seasonal k-values on the basis of relative damage to identify an average annual effective k-value for design.

All of these k-value guidelines are directly applicable not only to the AASHTO design procedure but to any concrete pavement design procedure, with the exception of the seasonal adjustment. The seasonal adjustment procedure given in Appendix H of this report was derived for the proposed revised AASHTO performance equation. The seasonal adjustment procedure is applicable in concept to any concrete pavement design procedure, but would have to be derived for the performance model used. All of the above k-value guidelines are presented in Appendixes F and H and are documented in Appendix B.

IMPROVED SUPPORT CONSIDERATION IN AASHTO DESIGN METHODOLOGY

Improved consideration of the effect of slab support on concrete pavement performance is possible through proposed revisions to the AASHTO design procedure. The revised procedure consists of the following steps:

Step 1. Develop Effective Modulus of Subgrade Reaction (k-value).

The design k-value is defined as that on top of the finished roadbed soil or embankment upon which the base and slab will be constructed. The k-value may be determined by (a) correlation with soil types, properties, or tests; (b) deflection testing of in-service pavements; or (c) direct plate load testing. These options are described in detail in Appendixes B and F. Adjustments to the k-value for seasonal variation, embankment material, and a shallow rigid layer may also be needed.

Step 2. Determine Required Slab Thickness.

The required slab thickness is a function of the following:

- effective (seasonally adjusted) k-value,
- estimated future traffic in the design lane for the performance period, W_{18},
- design reliability, R,
- overall standard deviation, S_O,
- design serviceability loss, P1 − P2,
- concrete modulus of rupture, S_c',
- concrete elastic modulus, E_c,
- joint spacing, L,
- base modulus, E_b,
- base thickness, H_b,
- effective positive temperature differential, TD (from equation given),
- lane edge support condition (conventional width and AC shoulder, conventional width and tied PCC shoulder, or widened slab), and
- friction coefficient between the slab and base, f.

Although the equation for midslab critical stress is very complex, it was found that for a given combination of design k-value, base type (i.e., modulus and friction coefficient), joint spacing, concrete modulus of rupture, and positive temperature differential, a linear relationship exists between log W_{18R} (the logarithm of the design ESALs for the specified level of design reliability) and the slab thickness D.

Step 3. Design Check for Joint Loading Stress.

If the transverse joints will not be dowelled, stress in the corner region due to joint loading may exceed the stress due to midslab loading under some design and climatic conditions. If the joint load stress is found to be critical, changes in the slab design or joint design features may be required.

Step 4. Design Check for Joint Faulting.

A design that is adequate in terms of slab thickness is not necessarily adequate in terms of faulting, a distress which has a significant effect on serviceability loss but which is not included in the AASHTO performance model. To ensure that the design is adequate with respect to faulting potential, faulting is predicted using available models. If the predicted faulting exceeds the recommended critical level, changes in the joint design are required. Slab thickness should not be increased to attempt to compensate for faulting potential.

IMPROVED SUPPORT CHARACTERIZATION FOR MECHANISTIC DESIGN

Mechanistic design requires realistic characterization of the subgrade and base layers beneath the concrete slab so that the critical tensile stresses in the slab due to loading and climate can be accurately computed for design purposes. The key practical findings of this research related to improved support characterization for mechanistic design of concrete pavements are summarized below.

Adequate Characterization of Subgrade Soil

The elastic response of real soils lies somewhere between the two extremes of the dense liquid and elastic solid idealizations. Unbound fine-grained and granular materials with relatively low shear strength are much closer to the dense liquid end of the spectrum than to the elastic solid end. Stabilized materials behave much more like elastic layers than like a dense liquid foundation.

The dense liquid model is preferred for subgrade characterization in mechanistic concrete pavement design primarily because it more realistically represents the behavior of natural soil subgrades or granular bases than does the elastic solid model. For example, if a load were placed at the edge of slab with no physical load transfer (e.g., no dowels or aggregate interlock) and an unstabilized foundation, the loaded slab would deflect and the unloaded slab would not deflect. This is the response the dense liquid model would predict. The elastic solid model, on the other hand, would predict equal deflections on both sides of the joint, even though the shear stress produced in the foundation may be substantially higher than the shear strength of the foundation material. Because concrete slab responses at edges and corners are considered critical for design purposes, the dense liquid model is considered more appropriate.

This research has shown that the elastic *k*-value, as determined at the Arlington and AASHO Road Tests through repetitive plate load testing on the subgrade, produces a support *k*-value that when used in an appropriate 3-D finite element model of the pavement, agrees well with strains measured under full-scale field loading conditions. Results from FWD or vibratory deflection testing on top of the slab and backcalculation of the *k*-value also produces an elastic *k*-value, after an appropriate reduction to estimate a static test value.

Rate of loading was found to have a major effect on the *k*-value. In analysis of the AASHO Road Test strain data, a *k*-value of about 80 psi/in. [22 kPa/mm] was found to achieve the best match between strains computed by the 3-D finite element model and the strains measured in the field under creep speed loading. This *k*-value is close to the mean elastic *k*-value measured in plate load tests at the site. At speeds approaching 60 mph [97 km/hr], an elastic *k*-value of over 300 psi/in. [81 kPa/mm] was required to match the measured strains. Mechanistic design could consider rate of loading (i.e., traffic speed) as a design input. This requires further research into the relationship of loading rate to stiffness for different types of soils. Furthermore, mechanistic performance models must be developed using performance data from pavements for which the observed performance (e.g., cracking) is related to responses (e.g., stresses) calculated using *k*-values appropriate for the rate of loading which the pavements experienced. In contrast, current performance models like the AASHTO model which were derived with respect to the static *k*-value of the pavements tested require a static *k*-value as a design input.

Foundation support characterization has in the past been simplified by representing the foundation as a uniform subgrade of infinite depth. However, more realistic situations that should be considered in mechanistic design are (a) the possibility that a thick layer of select material may be placed over the natural subgrade, or (b) the possibility that a stiff layer (e.g., bedrock or hardpan clay) may be present at some relatively shallow depth below the natural subgrade surface. This research has addressed these issues and has produced procedures to adjust subgrade *k*-values for fill layers above the subgrade and shallow rigid layers below the subgrade.

Characterization of the Base Layer

The base layer should be characterized as a structural elastic layer along with the concrete slab. The common practice of characterizing the base layer as an enhancement to the subgrade (a so-called "composite" *k*-value) yields computed slab responses that are unrealistic.

The degree of friction between the base and slab is important to the magnitude of stress experienced in the slab. Studies indicate that a range of friction coefficient values can be identified for a specific base type or interface treatment. The analyses conducted with 3DPAVE for this study resulted in development of a stress model that permits calculation of the stress that will occur in a concrete slab under loading as a function of climate inputs and subgrade, base, and slab inputs, including the coefficient of friction between the slab and base.

Effect of Subgrade and Base Stiffness With and Without Temperature Gradient

When no temperature gradient exists through a slab, an increase in subgrade *k*-value or base modulus will always decrease tensile stress in the slab under loading, suggesting a thinner slab is needed. The effect of subgrade and base stiffness is very different when a temperature differential exists through the slab. Results of 3-D finite element analyses show that very stiff foundations may actualy increase combined load and temperature curl stresses, resulting in thicker slab requirements or a reduction in the joint spacing. Mechanistic design can fully consider these complex tradeoffs between foundation stiffness, slab thickness, and joint spacing to provide a more adequately designed pavement.

Seasonal Variation in Subgrade Support

Seasonal variation in subgrade support is known to exist with many climates and soils. Mechanistic design can accomodate the computation of slab stresses over different seasons throughout the year based upon changes in the subgrade *k*-value. Procedures are provided to estimate a *k*-value for a given fine-grained soil classification based on its degree of saturation. Thus, if the approximate degree of saturation can be estimated seasonally, the elastic *k*-value can be estimated and used for design.

Loss of Support from Erosion and Slab Curl/Warp

Loss of support at joints can be caused by erosion of the base or subgrade resulting in a void beneath the corner. Loss of support can also be caused from (1) a negative temperature differential through the slab, (2) construction-related built-in temperature curling, or (3) moisture shrinkage of the surface—all causing the corners and edges to lift up. Loss of support from any of these causes results in increased tensile slab stress at the top of the slab.

The loss of support and increased stresses caused from temperature and perhaps from moisture gradients can be calculated using 3-D finite element models. They can be reasonably included in a mechanistic design procedure as discussed in the next section.

The loss of support caused by erosion of the base or subgrade material from pumping action is far more difficult to consider. It is fairly easy to model a void beneath a slab corner using finite element analysis. It is very difficult, however, to estimate the progression of loss of support that will occur in the field for a pavement with given design features in a given climate, subject to given traffic loadings. Research has linked erodibility of treated base materials to the strength and permeability of the materials. Many concrete pavements today are being constructed over permeable bases, used for the express reason to prevent erosion and loss of support. Further research is needed to develop improved predictive models for erosion of various base types and properties. When known, this loss of support from erosion can be considered in mechanistic design through year-by-year computation of stresses.

Critical Loading Position

Analyses have shown that two different loading positions may produce critical tensile stresses in a jointed concrete pavement: the midslab position and the joint positions. This is not a new finding as there have been some well-documented research efforts that have shown that both of these locations could be a critical condition under various design and climatic conditions.

The midslab load position has been used in concrete pavement design for many years. The wheel is positioned near the edge of the traffic lane and the critical stress occurs at the bottom of the slab. Cracks initiate at the bottom of the slab. This stress is increased or decreased depending on the thermal gradient. This stress leads to transverse cracking from fatigue damage. This damage can be greatly reduced through the use of thicker slabs, higher-strength concrete, shorter joint spacing, and treated bases with high friction with the slab. Widened slabs and tied concrete shoulders also reduce this stress.

The joint load position has not been used in concrete pavement design in the past. When the axle load is near the joint and corner of the slab, tensile stresses occur in the top of the slab. These stresses are greatly increased with nighttime temperature gradients, as well as from construction curl and moisture shrinkage warping. These stresses are greatly reduced when the joints are dowelled; practically no corner breaks have been found in pavements with adequately dowelled joints. If the joints contain undersized dowels, they may become loose in their sockets and become ineffective.

The joint loading position may be critical for undowelled pavements, especially in dryer climates. Some undowelled pavements that had significant permanent construction curling have experienced many broken corners, diagonal cracks, and even transverse cracks after just a few months or years. Situations in which the joint load position may become critical may be easily identified and modelled using mechanistic design procedures.

Joint Faulting Check

Joint faulting is the result of erosion of some underlying layer beneath the slab and reduction in load transfer. Prediction models have been developed by various researchers that directly link joint faulting with design features, climate, materials, and traffic. Although these models need improvement, especially the incorporation of more mechanistic concepts and variables, they represent an initial attempt at providing improved design. They can be used to directly check a given design for faulting. If the predicted faulting is excessive, the joint design and other features can be modified so that faulting is controlled. Procedures to accomplish that design check are provided herein and improved procedures can be developed for mechanistic design.

Effect of Subgrade/Base Stiffness on Joint Spacing

Field performance has shown that joint spacing has a considerable effect on the development of transverse cracking. However, the stiffness of the base and subgrade are major factors in this mechanism as well as slab thickness and the particular climate involved. Complex interactions exist between all of these variables, which affect tensile stresses in the slab.

Mechanistic design procedures based on 3-D finite element analyses have the potential to fully consider these interactions so that the maximum joint spacing can be determined more rationally than in the past. For example, several design manuals provide rules of thumb for joint spacing as a function of slab thickness. These rules of thumb do not, however, adequately consider the effects of climate, frictional resistance, and base and subgrade stiffness.

Data and computational methods for thermal gradients, moisture gradients and construction curling are becoming more readily available, along with finite element tools that can compute stresses caused by these climatic factors to a much more accurate degree than in the past. These tools may be used in mechanistic design to more accurately model the pavement structure under load and climatic conditions.

CHAPTER 4

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

The support that the base and embankment or roadbed subgrade provide to a concrete pavement was found to have a very significant effect on the performance of the pavement. The following key conclusions are based upon the results obtained from this research. Further information related to each conclusion can be found in the previous chapters and in the appendixes to this report.

Determination of *k*-Value

1. The *k*-value that is most appropriate for design is defined as that measured on the finished roadbed subgrade (or constructed embankment) upon which the base and slab will eventually be constructed. The concept of a composite "top-of-the-base" *k*-value is not valid and is not recommended for design. Use of a composite *k*-value does not accurately represent the effects of the base and subgrade on slab stresses or deflections.

2. The recommended *k*-value input to concrete pavement design is the *static elastic* *k*-value. The elastic *k*-value as measured in plate load tests at the AASHO Road Test and Arlington Road Test was found to provide a match between calculated and measured slab strains and deflections. When the elastic *k*-value was used in the 3DPAVE finite element model, the computed slab stresses were found to match very closely the stresses computed from strains measured at the AASHO Road Test. An elastic *k*-value can be determined from the results of standard plate load test methods specified by ASTM, AASHTO, or the Corps of Engineers.

3. Three categories of methods for determining a *k*-value for use in concrete pavement design are available, with detailed guidelines provided for each.

Correlation methods. The static elastic *k*-value can be estimated using soil classification, moisture level, density, CBR, Hveem R-value, and the Dynamic Cone Penetrometer (DCP). These correlation methods (along with procedures to adjust for embankments of improved materials and underlying stiff layers) are adequate for use in routine design. Design of projects on very unusual subgrades and especially more critical high-traffic-volume projects may require field testing for further verification.

Deflection testing and backcalculation methods. These methods are suitable for determining *k*-value for design of overlays of existing pavements, for design of reconstructed pavements on existing alignments, or for design of similar pavements in the same general location on the same type of subgrade. Backcalculation represents the most rapid, cost-effective, and reliable method to obtain an adequate sample size of *k*-values for pavement design. An agency may also use backcalculation methods to develop correlations between nondestructive deflection testing results and subgrade types and properties. The *k*-value obtained from backcalculation, when divided by a factor of two, yields an estimate of the static *k*-value for design which agrees well with the static *k*-value estimated from the correlation or plate test methods.

Plate testing methods. The standard ASTM, AASHTO, and Corps of Engineers nonrepetitive and repetitive plate-loading test methods were reviewed in this study, as well as the German plate load test. The American standard test methods are the most direct methods of determining the elastic *k*-value of the soil or embankment under static loading, but because these tests are very costly, time-consuming, and thus provide a very limited sampling of the project, it is not anticipated that they will be conducted routinely. It is unfortunate that a more rapid and simple plate bearing test method for *k*-value has not yet been developed. The elastic *k*-value may be determined using a 30-in.-diameter (762-mm) bearing plate in either a repetitive static plate load test (e.g., AASHTO T 221, ASTM D 1195) or nonrepetitive static plate load test (AASHTO T 222, ASTM D 1196). In a repetitive test, the elastic *k*-value is determined from the ratio of load to elastic deformation (the recoverable portion of the total deformation measured). In a nonrepetitive test, the load-deformation ratio at a deformation of 0.05 in. (1.25 mm) is considered to represent the elastic *k*-value.

Many design manuals state that the *k*-value can be estimated with sufficient accuracy based on soil classification or correlations with soil tests because of the relative small effect that it has on slab thickness. Analyses were conducted for a variety of design conditions and the following general results obtained for the error in slab thickness due to an error in *k*-value estimation.

Percent Error In *k*-value	Typical Maximum Error in Slab Thickness
10	1 percent
25	2.5
50	5
100	10

For example, a measured mean *k*-value of 250 psi/in. (68 kPa/mm) is estimated to be 375 psi/in. (102 kPa/mm) in error (50 percent overestimation). The error in required slab thickness is approximately 5 percent (i.e., 7.6 vs 8.0 in. [193 vs 203 mm]).

Thus, the *k*-value does not need to be estimated with great accuracy, but an error of 50 percent or greater could have a significant effect on slab thickness design.

4. Practical guidelines were developed for estimating the *k*-value for situations where the existing subgrade is a very poor soil and an embankment layer of improved soil is placed on top. Guidelines are also provided for increasing an estimated *k*-value (i.e., determined from correlations) when there is a stiff layer (such as bedrock) close to the surface.

5. Guidelines are provided for estimation of seasonal *k*-values on the basis of variation in moisture level and frost depth throughout the year for various soil types.

Improved Support Considerations in AASHTO *Guide*

6. Several major deficiencies related to concrete pavement support were found to exist in the current version of the AASHTO design procedure. These deficiencies were addressed in this study and an improved methodology was developed for better consideration of slab support. Key problems and solutions are listed below.

(a) Although the elastic *k*-value was the main test conducted at the AASHO Road Test, the more conservative "gross" *k*-value (including both elastic and permanent deformation) was incorporated in the AASHTO concrete pavement design equation. The elastic *k*-values exceeded the gross *k*-values at the AASHO Road Test by an average ratio of 1.77. Extensive full-scale slab testing by the Corps of Engineers has found that it is the elastic *k*-value, not the gross *k*-value, which matches *k*-values backcalculated from slab deflection basins under static loading. In this study, 3-D finite element analyses showed that modelling the AASHO Road Test pavements with the elastic *k*-value produced an excellent match between calculated and measured slab stresses.

Solution: the elastic *k*-value is recommended as the appropriate subgrade input parameter for use in concrete pavement design.

(c) The *k*-value incorporated in the AASHTO design equation was the mean gross *k*-value measured on top of the granular base, not the mean for the subgrade. This is inconsistent with the "composite *k*" adjustment given in the current procedure, which increases the subgrade *k* to reflect the effect of the base. The concept of a composite, top-of-the-base *k*-value was judged in this study to be invalid because it does not represent the support that the slab actually receives from the subgrade and base layers, and thus does not accurately predict slab stresses or deflections. This is especially true for stiff treated bases that act more as structural layers in reducing stress in the slab.

Solution: the subgrade *k*-value is the recommended support input for design. A procedure was developed to consider the effect of the base on slab stress and performance. The base layer was modelled as a structural layer of the pavement structure using the 3-D finite element model. The modulus of elasticity, thickness, and friction coefficient of the base course are important inputs.

(d) The lowest gross *k*-value that occurred during the spring on top of the base was incorporated into the AASHTO design model in 1961. The model was not re-derived for a seasonally adjusted *k* for the AASHO site when a procedure to determine an effective seasonally adjusted *k* was added to the *Guide* in 1986.

Solution: The original rigid pavement design equation was modified to incorporate a seasonally adjusted effective elastic *k*-value that existed on the embankment at the Road Test site.

(e) A loss-of-support adjustment factor, which makes drastic reductions to *k* for erodible base types, was incorporated in the 1986 *Guide*. However, substantial loss of support existed for many sections at the AASHO site, which led to increased slab cracking and loss of serviceability. Thus, the performance data and design equation already represent the effect of considerable loss of support. Incorporation of an additional loss-of-support adjustment to *k* is unnecessary and unwarranted and produces a thicker slab design.

Solution: a loss-of-support adjustment is not recommended for the AASHTO design procedure. Erodibility should be directly evaluated in terms of the potential faulting of transverse joints, and the joint design adjusted if necessary to prevent faulting.

(f) Spangler's unprotected corner stress equation was incorporated in the 1961 extension of the AASHTO model. Critical stresses at the AASHO Road test occurred along the longitudinal slab edge for slabs 6.5. in. (165 mm) and greater, and resulted in transverse fatigue cracks. The stresses in the vicinity of the corner were much lower than those at midslab because the joints were well dowelled. Use of Spangler's corner equation when used with dowelled joints did not model the critical stress and crack initiation location, and thus could not possibly provide accurate indications of the effect of slab support on cracking, especially when thermal curling and moisture warping are considered.

Solution: A stress equation was developed using the 3-D finite element model 3DPAVE to compute critical stresses at the midslab location, including consideration of thermal gradients, base properties (stiffness, thickness, and friction), elastic *k*-value, slab modulus and strength, and joint spacing. Also, a design check for critical stress is provided for undowelled pavements at the joint loading position which may be critical in certain conditions such as dry climates.

(g) No procedure is provided in the current AASHTO *Guide* to design a pavement with undowelled joints. The *J*-factor only considers tensile stress that controls cracking, not faulting.

Solution: Design checks are provided for undowelled joints for both joint faulting and slab cracking due to joint loading. A check is also provided for dowelled joints to ensure their adequacy. If the design checks show inadequate joint design, the design may be revised and reevaluated.

(h) Joint spacing (other than that built into the procedure, i.e., 15 ft [4.6 m] at the Road Test) is not considered at all in the design procedure, and it is known that joint spacing has a major effect on slab cracking and faulting.

Solution: Subgrade and base support were found to have a significant effect on allowable joint spacing when stresses from load, temperature, and moisture gradients are considered. Thus, slab support is a very important variable in the new procedure in the selection of a joint spacing to minimize transverse cracking.

(i) The original 1961 model reflects the AASHO site climate only. The current version does not include any variable that adjusts for a different climate. Thus, other climates that result, for example, in different degrees of slab curling (temperature differential from top to bottom of slab) or warping (moisture gradient) than occurred at the AASHO site cannot be considered. This limitation alone has led to many pavement failures from premature cracking.

Solution: Temperature differentials were incorporated into the slab thickness design procedure: positive for the midslab location (i.e., warmer on top than bottom) and negative for the corner/joint loading position. Appropriate positive and negative temperature differential inputs for a given location may be determined as a function of readily available climatic data. Therefore, the stresses computed to design the pavement should be much more realistic. Moisture gradients are considered through an equivalent temperature gradient for the joint (corner) load position.

(j) The current AASHTO model does not reflect the effect of faulting on performance, because faulting of transverse joints did not occur at the Road Test because of properly sized dowels. As the Road Test results demonstrate, even with extensive erosion and pumping, faulting can be controlled through properly sized dowel bars. The *J*-factor input in the current AASHTO procedure, thought by many engineers to control faulting, has nothing to do with erosion and faulting at the joint. Increasing slab thickness in an attempt to reduce faulting has been shown to be an exercise in futility.

Solution: Joint faulting is predicted for the selected design and then checked against a defined maximum level. If predicted faulting exceeds the allowable faulting, a redesign of the joint load transfer, base, subdrainage, or other features is required, not an increase in slab thickness.

A revised AASHTO rigid pavement design procedure is proposed in this study that provides significant improvements to characterizing pavement support. This procedure overcomes many of the limitations of the existing method. Partial verification of the method was accomplished with performance data from the extended AASHO Road Test pavements and pavements with other designs located in other climatic zones.

Method to Compute Slab Stresses Accurately

7. A 3-D finite element model for concrete pavements was developed in this study in order to analyze realistically the many complex and interacting factors that influence the support provided to a concrete pavement, including:

- subgrade support (subgrade *k*-value);
- base thickness, stiffness, and interface friction;
- slab curling and warping due to temperature and moisture gradients;
- dowel and aggregate interlock load transfer action at joints; and
- improved support with a widened slab, widened base, or tied concrete shoulder.

8. The ABAQUS general-purpose finite element software was used to develop a very powerful and versatile 3-D model for analysis of concrete pavements. The 3DPAVE model easily overcomes many of the inherent limitations of 2-D finite element and Westergaard models. During its development, 3DPAVE was checked against the existing 2-D finite element models and against available theoretical solutions (e.g., Westergaard's equations). The 3DPAVE consistently outperformed the 2-D model in accuracy over wide ranges of inputs for a variety of problems.

9. The 3-D model was validated by comparison with measured deflection and strain data for traffic loadings and temperature differentials from the AASHO Road Test, the Arlington Road Test, and the Portland Cement Association's slab experiments. In every comparison with measured field data, 3DPAVE's calculated responses were found to be in very good agreement with the measured responses, and significantly closer than those calculated by the 2-D program.

Loss of Support

10. Loss of support refers to any gap or void that may occur between the base and the slab, or between a stabilized base and the subgrade, causing increased deflection of the slab surface. There are two basic types of loss of support that a concrete slab exhibits over time.

- Loss of support from erosion of the base and or subgrade from beneath the slab that results in increased deflections and stresses in the slab.
- Temperature curling and moisture warping of the slab that results in increased deflections or stresses in the slab. Permanent construction curling presents a potential for very serious loss of support and early failure of jointed concrete pavements.

Both of these causes of loss of support can have a major impact on slab deflections and stresses, and thus pavement life. Temperature curling and moisture warping can be reasonably considered in the design process and thus, to this extent, a significant amount of loss of support is included directly. Further loss of support from erosion of the base/subgrade cannot be predicted at the present time for any given design project. The specification of adequate material properties to limit the potential erosion for given classes of project applications is recommended at this time, and guidelines are provided.

Improved Support Characterization For Mechanistic Design

11. The elastic *k*-value produces a support *k*-value that when used in an appropriate 3-D finite element model of the pavement, agrees well with strains and stresses measured under full-scale field loading conditions. Results from FWD or vibratory deflection testing on top of the slab and backcalculation of the *k*-value also produces an elastic *k*-value (after appropriate reduction from dynamic to static loading).

12. Speed of loading was found to have a major effect on the *k*-value of the AASHO Road Test soil. At creep speed, a *k*-value of about 80 psi/in. produced the measured strains (which was close to the mean elastic *k*-value measured in the field), whereas at speeds approaching 60 mph, an elastic *k*-value of over 300 psi/in. was required to produce the decreased measured strains. Mechanistic design could consider speed of loading as a design input.

13. The frictional resistance between the base and the slab is

an important factor for the critical stress in the slab. It is desirable to provide high friction between the base and the slab to reduce critical tensile stresses, reduce the potential for erosion, and provide desired cracking through sawed or formed joints. It is believed that in practice, a substantial amount of bonding and high friction exists between the base and the slab and attempts to "break bond" between two layers is rarely successful as determined by coring and backcalculation. Reflection cracking problems from the base can be handled through forming joints in the treated base course.

14. The effect of subgrade and base stiffness on slab stresses is very different when a temperature differential exists through the slab. When no temperature gradient exists through a slab, increased subgrade k-value or base modulus value will always show reduced tensile stress in the slab under loading, and thus design will require a thinner slab. Results shown herein using the 3-D finite element model indicates that very stiff foundations may actually increase combined load and temperature curl stresses resulting in thicker slab requirements. Greatly increased base and subgrade stiffness may not always be beneficial. Under these conditions it may be necessary to shorten the joint spacing to avoid premature transverse cracks in the slab. Mechanistic design using 3-D finite element as a basis can consider these complex tradeoffs to provide a more adequately designed pavement.

15. Analyses showed that two different loading positions could produce critical tensile stresses for a given pavement: midslab and joint (corner). This is not a new finding as there have been some well-documented research efforts that have shown that both of these locations could be a critical condition under various design and climatic conditions.

16. The real problem is undowelled or inadequately dowelled pavements. Some undowelled pavements that had substantial permanent construction curling have experienced many broken corners, diagonal cracks, and even transverse cracks after just a few months or years. This is a critical load position that can be very well modelled and the stresses checked using mechanistic design procedures.

17. Field performance has shown that joint spacing has a very large effect on the development of transverse cracking. The stiffness of the base and subgrade (and their bonding situation) are major factors in this mechanism as well as slab thickness and the particular climate involved. Complex interactions exist between all of these variables in affecting tensile stresses in the slab and mechanistic design that utilizes 3-D finite element models has the potential to consider these interactions so that the joint spacing for projects can be determined more rationally than in the past.

18. Data and computational methods for thermal gradients, moisture gradients, and construction curling are becoming more readily available along with the finite element software to predict the resulting stresses to a much more accurate degree and to then use them to produce a more reliable and cost-effective pavement design. These can all be used in mechanistic design to more accurately model the pavement structure under load and climatic conditions.

SUGGESTED RESEARCH

The following key areas of research are suggested.

1. Further verification of the elastic k-value test and estimation procedures in the field is needed. This would include the evaluation of all three approaches to estimating the elastic k-value at specific project sites: correlations with soil properties and tests, backcalculation, and plate bearing tests. A field and analytical study is needed to verify that these procedures will provide approximately the same results.

2. The prediction of erosion and subsequent loss of support beneath concrete pavements is an area requiring additional research. The study should first concentrate on developing simple test procedures to characterize the degree of erodibility of various materials. Second, analytical methods (finite element) of modelling loss of support should be developed. Third, methods should be developed for prediction of loss of support over a pavement's life.

3. Development of a rapid method of measuring the bearing capacity of subgrades is needed. This study could concentrate on the development of a portable, hand-held device and/or a static loading version of a Falling Weight Deflectometer. In theory, this device should be relatively easy to manufacture since only the maximum deflection in the center of the plate is needed along with the weight applied. The resulting dynamic bearing value could be correlated with the standard elastic k-value measured with a large 30-in. (760-mm) diameter plate.

4. Further verification of the friction coefficients is needed. The values recommended in this study are based on those determined through pushing a slab over a base course, not a slab and base course under wheel load and thermal gradient. The main concern is determining the appropriate friction for design of different bases and interfaces that would exist over a pavement's life cycle.

5. Further verification of the revised AASHTO rigid pavement design model is needed. This could be accomplished using the Long-Term Pavement Performance database. Partial verification was accomplished under this study using the extended AASHO sections and the FHWA RPPR database. However, more verification is needed and the LTPP database includes most of the required information.

REFERENCES

1. WINKLER, E., *Die Hehre von der Elastizitat und Festigkeit* (The Theory of Elasticity and Stiffness), H. Dominicus, Prague (1867).
2. HERTZ, H., "Über das Gleichgewicht schwimmender elastischer Platten," (On the Equilibrium of Floating Elastic Plates), *Wiedemann's Annalen der Physik und Chemie,* Vol. 22 (1884).
3. FÖPPL, A., *Technische Mechanik,* Vol. 5 (1907).
4. KOCH, J.J., "Berekening van vlakke platen, ondersteund in de hoekpunten van een willekeurig rooster," *De Ingenieur,* No. 6 (1925).
5. SCHLEICHER, F., "Über Kreisplatten auf elastischer Unterlage," Festschrift sur Hundertjahrfeier der Technischen Hochschule Karisruhe (1925).
6. WESTERGAARD, H.M., "Computation of Stresses in Concrete Roads," *Proceedings,* Highway Research Board (1925). Also published as "Stresses in Concrete Pavements Computed by Theoretical Analysis," *Public Roads,* Vol. 7, No. 2 (1926).
7. SELVADURAI, A.P.S., *Elastic Analysis of Soil-Foundation Interaction,* Developments in Geotechnical Engineering, Vol. 17, Elsevier Scientific Publishing Company, the Netherlands (1979).
8. PECK, R.B., HANSON, W.E., and THORNBURN, T.H., *Foundation Engineering,* 2nd edition, John Wiley and Sons, New York (1974).
9. WESTERGAARD, H.M., "Theory of Stresses in Road Slabs," *Proceedings,* Fourth Annual Meeting of the Highway Research Board (1925).
10. TELLER, L.W. and SUTHERLAND, E.C., "The Structural Design of Concrete Pavements, Part 1, A Description of the Investigation," *Public Roads,* Vol. 16, No. 8 (1935).
11. TELLER, L.W. and SUTHERLAND, E.C., "The Structural Design of Concrete Pavements, Part 2, Observed Effects of Variations in Temperature and Moisture on the Size, Shape and Stress Resistance of Concrete Pavement Slabs," *Public Roads,* Vol. 16, No. 9 (1935).
12. TELLER, L.W. and SUTHERLAND, E.C., "The Structural Design of Concrete Pavements, Part 3, A Study of Concrete Pavement Cross Sections," *Public Roads,* Vol. 16, No. 10 (1935).
13. TELLER, L.W. and SUTHERLAND, E.C., "The Structural Design of Concrete Pavements, Part 4, A Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs," *Public Roads,* Vol. 17, Nos. 7 and 8 (1936).
14. TELLER, L.W. and SUTHERLAND, E.C., "The Structural Design of Concrete Pavements, Part 5, An Experimental Study of the Westergaard Analysis of Stress Conditions in Concrete Pavements of Uniform Thickness," *Public Roads,* Vol. 23, No. 8 (1943).
15. KELLEY, E.F., "Application of the Results of Research to the Structural Design of Concrete Pavements," *Public Roads,* Vol. 20, No. 5 (1939).
16. U.S. ARMY CORPS OF ENGINEERS, "Wright Field Slab Tests," Ohio River Division, Cincinnati Testing Laboratories, Cincinnati, OH (1942).
17. U.S. ARMY CORPS OF ENGINEERS, "Report on Special Field Bearing Tests on Natural Subgrade and Prepared Subbase Using Different Size Bearing Plates," Ohio River Division, Cincinnati Testing Laboratories (1943).
18. SALE, J.P. and HUTCHINSON, R. L., "Development of Rigid Pavement Design Criteria for Military Airfields," Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. AT3 (July 1959).
19. PHILLIPPE, R. R., "Field Bearing Tests Applied to Pavement Design," Symposium on Load Tests of Bearing Capacity of Soils, American Society for Testing and Materials, Special Technical Publication No. 79 (1947).
20. U.S. ARMY CORPS OF ENGINEERS, "Rigid Plate-Bearing Test Investigation," Waterways Experiment Station, Vicksburg, MS (1945).
21. U.S. ARMY CORPS OF ENGINEERS, "Determination of High Values of the Subgrade Modulus, k, from the Plate Bearing Test," Rigid Pavement Laboratory, Ohio River Division, Mariemont, OH (1953).
22. SALE, J.P., "Rigid Pavement Design for Airfields," *Proceedings,* First International Conference on Concrete Pavement Design, Purdue University, West Lafayette, IN (1977).
23. MIDDLEBROOKS, T.A. and BERTRAM, G. E., "Soil Tests for Design of Runway Pavements," *Proceedings,* Highway Research Board (1942).
24. U.S. DEPARTMENT OF THE ARMY, Technical Manual TM 5-824-3.
25. U.S. DEPARTMENT OF THE ARMY, "Rigid Airfield Pavements," Engineering and Design Manual EM 1110-45-303, Office of the Chief of Engineers (1958).
26. PORTLAND CEMENT ASSOCIATION, "Thickness Design for Concrete Pavement" (1966).
27. PACKARD, R.G., "Design of Concrete Airport Pavement," *Portland Cement Assoc. Engineering Bulletin* (1973).
28. CAMPEN, W.H. and SMITH, J.R., "Use of Load Tests in the Design of Flexible Pavements," Symposium on Load Tests of Bearing Capacity of Soils, American Society for Testing and Materials, *ASTM Special Publication No. 79* (1947).
29. HITTLE, J.E. and W.H. GOETZ, "Factors Influencing the Load-Carrying Capacity of Base-Subgrade Combinations," *Proceedings,* Highway Research Board (1946).
30. MCLEOD, N.W., "A Canadian Investigation of Load Testing Applied to Pavement Design," Symposium on Load Tests of Bearing Capacity of Soils, American Society for Testing and Materials, *ASTM Special Technical Publication No. 79* (1947).
31. AHLVIN, R.G., "Origin of Developments for Structural Design of Pavements," Technical Report GL-91-26, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS (1991).
32. HIGHWAY RESEARCH BOARD, "The AASHO Road Test, Report 5, Pavement Research," *Special Report 61E* (1962).
33. LANGSNER, G., HUFF, T.S., and LIDDLE, W.J., "Use of Road Test Findings by AASHO Design Committee," from "The

AASHO Road Test, Proceedings of a Conference Held May 16-18, 1962, St. Louis, MO," *HRB Special Report No. 73* (1962).
34. U.S. Army Corps of Engineers, "Results of Modulus of Subgrade Reaction Determination at the AASHO Road Test Site by Means of Pavement Volumetric Displacement Test," Ohio River Division Laboratories (April 1962).
35. Vesic, A.S. and Saxena, S.K., "Analysis of Structural Behavior of AASHO Road Test Rigid Pavements, *NCHRP Report No. 97* (1966).
36. Hudson, W.R., "Comparison of Concrete Pavement Load-Stresses at AASHO Road Test with Previous Work," *Highway Research Record No. 42* (1963).
37. Childs, L.D. and Kapernick, J.W., "Tests of Concrete Pavements on Crushed Stoned Subbases," Proceedings of the American Society of Civil Engineers, Vol. 89, No. HW-1 (1963).
38. Childs, L.D., "Cement-Treated Subbases for Concrete Pavements," *Highway Research Record No. 189* (1967).
39. Nussbaum, P.J. and Larsen, T.J., "Load-Deflection Characteristics of Soil-Cement Pavements," *Highway Research Record No. 86* (1965).
40. American Association of State Highway and Transportation Officials, "Interim Guide for Design of Pavement Structures," Washington, D. C. (1972).
41. Darter, M.I., "Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume 1—Development of Design Procedures," University of Illinois at Urbana-Champaign, *Report No. FHWA-RD-77-111* (1977).
42. Darter, M.I. and Barenberg, E.J., "Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume 2—Design Manual," University of Illinois at Urbana-Champaign, *Report No. FHWA-RD-77-112* (1977).
43. Thompson, M.R. and Robnett, Q.L., "Resilient Properties of Subgrade Soils," Illinois Cooperative Highway Research Program, *Report No. UILU-ENG-76-2009,* University of Illinois and Illinois Department of Transportation (1976).
44. American Association of State Highway and Transportation Officials, *Guide for Design of Pavement Structures,* Washington, D.C., (1986).
45. Hoffman, M.S. and Thompson, M.R., "Mechanistic Interpretation of Nondestructive Pavement Testing Deflections," Transportation Engineering Series No. 32, Illinois Cooperative Highway and Transportation Research Series No. 190, University of Illinois at Urbana-Champaign (1981).
46. ERES Consultants, Inc., "Nondestructive Structural Evaluation of Airfield Pavements," prepared for U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS (1982).
47. Foxworthy, P.T., "Concepts for the Development of a Nondestructive Testing and Evaluation System for Rigid Airfield Pavements," Ph.D. thesis, University of Illinois at Urbana-Champaign (1985).
48. Barenberg, E.J. and Petros, K.A., "Evaluation of Concrete Pavements Using NDT Results," Illinois Highway Research Project IHR-512, University of Illinois and Illinois Department of Transportation, Report No. UILU-ENG-91-2006 (1991).
49. Ioannides, A.M., "Dimensional Analysis in NDT Rigid Pavement Evaluation," *Transportation Engineering Journal,* American Society of Civil Engineers, Vol. 116, No. TE1 (1990).
50. Ioannides, A.M., Barenberg, E.J., and Lary, J.A., "Interpretation of Falling Weight Deflectometer Results Using Principles of Dimensional Analysis," *Proceedings,* Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, IN (1989).
51. Hall, K.T., "Performance, Evaluation, and Rehabilitation of Asphalt-Overlaid Concrete Pavements," Ph.D. thesis, University of Illinois at Urbana-Champaign (1991).
52. American Association of State Highway and Transportation Officials, "Guide for Design of Pavement Structures," Washington, D. C. (1993).
53. Crovetti, J.A., "Evaluation of Jointed Concrete Pavement Systems Incorporating Open-Graded Subbases," Ph.D. thesis, University of Illinois at Urbana-Champaign (1994).
54. Potter, C.J. and Dirks, K.L., "Pavement Evaluation Using the Road Rater^{TM} Deflection Dish," Final Report, Project MLR-89-2, Iowa Department of Transportation (1989).
55. Barenberg, E.J. and Ratterree, B.L., "Fully Bonded Concrete Overlay for an Airport Runway," *Proceedings,* ASCE International Air Transportation Conference, New Orleans, LA (1979).
56. Smith, K.D., Peshkin, D.G., Darter, M.I., Mueller, A.L., and Carpenter, S.H., "Performance of Jointed Concrete Pavements," Phase I, Volume IV—Appendix A, *Report No. FHWA-RD-89-139* (1990).
57. Simpson, A.L., et al., "Early Analysis of LTPP General Pavement Studies Data," Volume 3—Sensitivity Analyses for Selected Pavement Distresses, Strategic Highway Research Program (1993).
58. Kogure, K. and Sakai, T., "Measurements of Deflection and Strain of Airfield Rigid Pavement for Deterioration and Evaluation," *Proceedings,* International Conference on Bearing Capacity of Roads and Airfields, Minneapolis, MN (July 1994).
59. Crawford, Murphy, and Tilly, et al., "Evaluation of Airfield Pavements at Dulles International Airport," report for Federal Aviation Administration (1984).
60. Beckemeyer, C.B. and Darter, M. I., "Utah I-15 Pavement Evaluation," Final Report for Utah DOT, ERES Consultants, Inc. (1993).
61. Poblete, M., Salsilli, R., Valenzuela, R., Bull, A., and Spratz, P., "Field Evaluation of Thermal Deformations in Undowelled PCC Pavement Slabs," *Transportation Research Record No. 1207* (1989).
62. Poblete, M., Garcia, A., David, J., Ceza, P., and Espinosa, R., "Moisture Effects on the Behaviour of PCC Pavements," *Proceedings,* Second International Workshop on the Design and the Evaluation of Concrete Pavements, Siguenza, Spain (October 1990).
63. Permanent International Association of Road Congresses, "Combatting Concrete Pavement Pumping," Technical Committee on Concrete Roads ····.
64. Eisenmann, J. and Leykauf, G., "Effect of Paving Temperatures on Pavement Performance," *Proceedings,* Second International Workshop on the Design and the Evaluation of Concrete Pavements, Siguenza, Spain (October 1990).
65. Zachlehner, A. "Beanspruchung von Betonfahrbahnen durch Einflüse durch Hydration und Witterung," (Concrete

Pavement Stress due to the Influence of Hydration and Climatic Conditions), Ph.D. thesis, Technical University of Munich, Germany (1989).

66. ZACHLEHNER, A., "Restraint Stresses in Young Concrete Pavements," *Proceedings,* Sixth International Symposium on Concrete Roads, Madrid, Spain (October 1990).
67. ARMAGHANI, J.M., LARSEN, T.J., and SMITH, L.L., "Temperature Response of Concrete Pavements," *Transportation Research Record No. 1121* (1987).
68. JANSSEN, D.J., "Moisture in Portland Cement Concrete," *Transportation Research Record No. 1121* (1987).
69. DARTER, M.I., BECKER, J.M., SNYDER, M.B., and SMITH, R.E., "Portland Cement Concrete Pavements Evaluation System—COPES," *NCHRP Report 277,* Transportation Research Board (1985).
70. HUDSON, W.R. and SCRIVNER, F.H., "AASHO Road Test Principal Relationships—Performance with Stress, Rigid Pavements," from "The AASHO Road Test, Proceedings of a Conference Held May 16-18, 1962, St. Louis, MO," *HRB Special Report No. 73* (1962).

APPENDIXES A THROUGH H

Unpublished Material

Appendixes A through H contained in the research agency's final report are not published herein. For a limited time, copies of that report, entitled, "Support Under Concrete Pavements—Appendixes," will be available on a loan basis or for puchase ($30.00) on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C., 20055. The available appendixes are titled as follows:

APPENDIX A: Development of *k*-Value Concepts and Methods
APPENDIX B: Methods for Estimating *k*-Value
APPENDIX C: Loss-of-Support Concepts and Methods
APPENDIX D: Three-Dimensional Finite Element Model Development and Validation
APPENDIX E: Improved Consideration of Support in Current AASHTO Methodology
APPENDIX F: Proposed Revision to AASHTO *Guide,* Part II, Section 3.2, "Rigid Pavement Design," and Section 5.3, "Rigid Pavement Joint Design"
APPENDIX G: Rigid Pavement Design Example (Proposed Revision to AASHTO *Guide* Appendix I)
APPENDIX H: Development of Effective Roadbed Soil *k*-Value (Proposed Revision to AASHTO *Guide* Appendix HH)

THE TRANSPORTATION RESEARCH BOARD is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. It evolved in 1974 from the Highway Research Board which was established in 1920. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society. The Board's purpose is to stimulate research concerning the nature and performance of transportation systems, to disseminate information that the research produces, and to encourage the application of appropriate research findings. The Board's program is carried out by more than 270 committees, task forces, and panels composed of more than 3,300 administrators, engineers, social scientists, attorneys, educators, and others concerned with transportation; they serve without compensation. The program is supported by state transportation and highway departments, the modal administrations of the U.S. Department of Transportation, the Association of American Railroads, the National Highway Traffic Safety Administration, and other organizations and individuals interested in the development of transportation.

The National Academy of Sciences is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Bruce M. Alberts is president of the National Academy of Sciences.

The National Academy of Engineering was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research and recognizes the superior achievements of engineers. Dr. Robert M. White is president of the National Academy of Engineering.

The Institute of Medicine was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Kenneth I. Shine is president of the Institute of Medicine.

The National Research Council was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purpose of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Bruce M. Alberts and Dr. Robert M. White are chairman and vice chairman, respectively, of the National Research Council.

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

ADDRESS CORRECTION REQUESTED