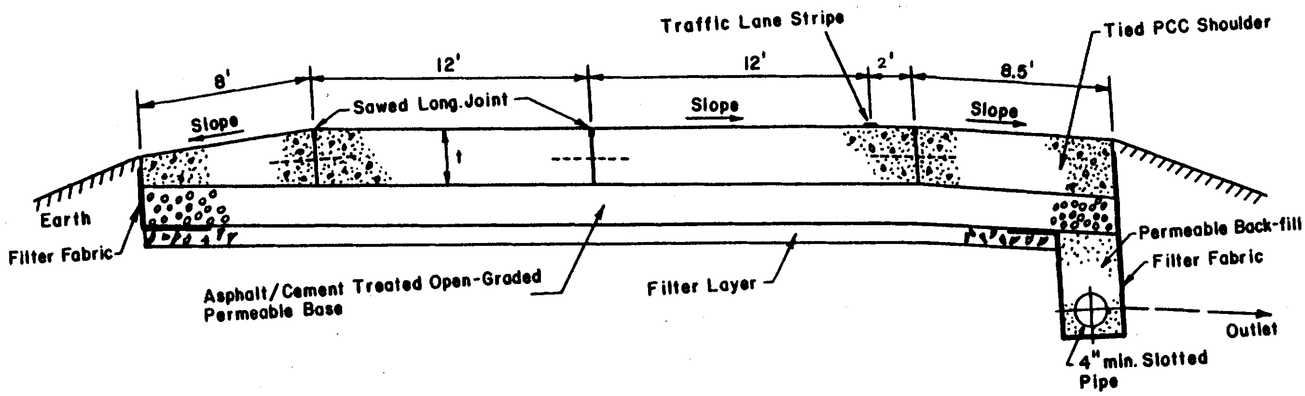
Rigid Pavement Analysis and Design

Publication No. FHWA-RD-88-068

June 1989



U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

This report presents the findings of a research study to evaluate currently available rigid pavement analysis models and design methods. The methods and models were evaluated for their limitations as well as their capabilities. A micro-computer program was developed which predicts average faulting values for doweled and non-doweled joints, based on specific design inputs. It should prove useful as a tool to check proposed pavement designs. Finally, specific experimental pavement designs were developed based on the study findings.

This report will be of interest to engineers involved in rigid pavement design and analysis.

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NOTICE

Technical Report Documentation Page

1. Report No. FHWA-RD-88-068	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle RIGID PAVEMENT ANALYSIS AND DESIGN		5. Report Date June 1989
		6. Performing Organization Code
7. Author(s) K. W. Heinrichs, M. J. Liu, M. I. Darter, S. H. Carpenter, A. M. Ioannides		8. Performing Organization Report No.
9. Performing Organization Name and Address Department of Civil Engineering University of Illinois at Urbana-Champaign Urbana, Illinois 61801-2397		10. Work Unit No. (TRAIS) NCP #3C1b-1012
		11. Contract or Grant No. DTFH-61-85-C-00103
12. Sponsoring Agency Name and Address Federal Highway Administration 6300 Georgetown Pike McLean, Virginia 22101-2296		13. Type of Report and Period Covered Final Report-Phase I June 1985 to August 1988
		14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contracting Officer Technical Representative was Dr. Stephen W. Forster.

16. Abstract

This study was conducted to characterize and compare currently available rigid pavement analysis models and design methods and to develop new rigid pavement designs to be evaluated in full-scale experimental projects in an actual highway environment. The analysis models considered include: ILLI-SLAB, JSLAB, WESLIQID, WESLAYER, JCS-1, H51, CRCP-2, and RISC. The design methods include: 1985 AASHTO Guide, Zero-Maintenance JCP-1, California DOT, PCA, RPS-3 Texas SDHPT, ARBP-CRSI and Illinois DOT. The capabilities, limitations, and assumptions of each of these models and methods are discussed in detail to assess their adequacy to assist in the design and analysis of rigid pavements. Based upon the results obtained, several models and methods are recommended for use in the development of new rigid pavement designs.

A set of rigid pavement designs, featuring trapezoidal cross sections, widened truck lanes, permeable drainage layer, longitudinal drainage pipe, precoated dowels, shorter joint spacing (for JRCP) and tied PCC shoulders was developed for field testing through experimental projects in the various climatic zones of the United States. A project description form (PDF) is provided for the interested State agencies.

Guidelines were developed for joint load transfer design and joint spacing. Predictive models for doweled and nondoweled joints were developed.

17. Key Words Rigid pavement, design, performance distress, joints, evaluation, concrete finite element, structural analysis		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 375	22. Price

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha

MASS (weight)

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

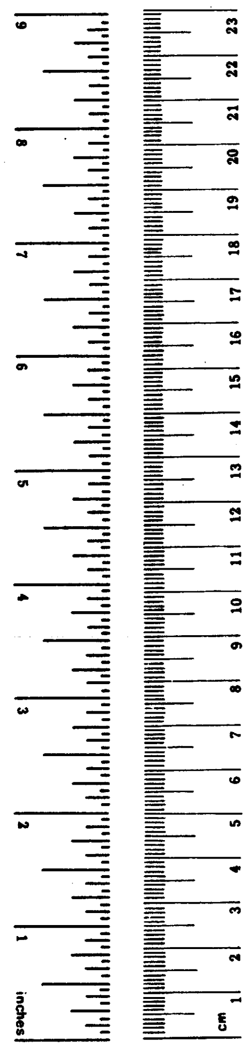
VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac

MASS (weight)

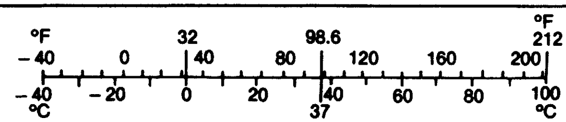
Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

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1.0 INTRODUCTION

This final report presents a comprehensive evaluation of several rigid pavement structural analysis models and design methods, a set of new rigid pavement designs for field experimental projects, and guidelines for joint designs. The purpose of the evaluations is to provide an assessment of the adequacy of these techniques to assist in the analysis and design of rigid pavements. Based upon the results obtained, models and methods are recommended for use in the new rigid pavement designs for field testing.

Rigid pavement analysis/design models include computer programs that mostly evaluate the structural aspects of a rigid pavement system. Models evaluated include the following:

- ILLISLAB.
- JSLAB.
- WESLIQID.
- WESLAYER.
- H51.
- CRCP-2.
- JCS-1.
- RISC.
- CMS Model.
- BERM.

The rigid pavement design methods are procedures that generally provide a complete rigid pavement design. Those evaluated include the following:

- AASHTO Design Guide--1986.
- Zero-Maintenance Design.
- California DOT JPCP Design.
- PCA Design.
- RPS-3 Texas SDHPT Design.
- ARBP-CRSI CRCP Design.
- Illinois DOT CRCP Design.

The evaluation involved the following tasks for most of the models or methods:

1. The fundamental concepts upon which the model/method was based were documented.

2. Any associated computer programs were obtained and put on-line, runs were made using a range of inputs and the results evaluated and documented.

3. Actual rigid pavement performance data were obtained from the NCHRP Project 1-19 (COPES) database and the Illinois CRCP database. These field data were compared with predicted performance data from each design method.

4. The capabilities of each design model and method were assessed on the basis of their ability to handle many aspects of rigid pavement design (e.g., slab thickness, length and width; loss of support beneath slab; joint load transfer and the slab/base interface).

5. Recommendations were made regarding the models and methods in rigid pavement design.

The information from these comprehensive evaluations was used to select the most adequate and promising models and methods for developing new rigid pavement designs.

A set of unique rigid pavement designs using a full factorial design approach was developed for the nine climatic zones. Several unique features were considered, including:

- Trapezoidal cross sections.
- Widened truck lanes.
- Permeable base layer.
- Longitudinal drainage pipe.
- Precoated dowels.
- Shorter joint spacings (for JRCP).
- Tied PCC shoulders.

This experiment required many sections and a long project having a two-factor, two-level factorial design. The trapozoidal cross sections and widened truck lanes were selected as the main factors with all other variables fixed. Four test sections plus a control section and some replication of sections are needed for each experimental project. These experimental projects will ideally be built in each climatic zone with long-term performance data collected for analysis. The experiment is recommended for consideration in the SHRP Specific Pavement Study program.

A project description form (PDF) was prepared for State agencies interested in participating in the program of rigid pavement experimental projects. This PDF will be used as an application form and an evaluation form for each interested State agency.

Two faulting predictive equations were developed for both doweled and undoweled jointed concrete pavements using the NCHRP Project 1-19 (COPES) database. A computer program "PFAULT" written by the staff in FORTRAN and incorporating these equations, is available in an IBM PC compatible version for joint faulting prediction. A joint load transfer guideline for jointed rigid pavements was developed to prevent faulting in transverse joints. This guideline provides a comprehensive system of evaluating joint design and the use of dowels for jointed concrete pavements.

2.0 CRITIQUE OF RIGID PAVEMENT MODELS

This section presents a critique of several rigid pavement structural models. The following material is presented for each model:

- Basic assumptions.
- Outline of capabilities.
- Inputs and outputs.
- Ability of model to assist in design of rigid pavement.
- Limitations.
- Detailed description of capabilities.
- Computer runs for sample problems.

Every effort was made to evaluate the models fairly and to determine their capabilities to assist the engineer in the design of rigid pavements.

2.1 ILLI-SLAB Finite Element Model

2.1.1 INTRODUCTION

ILLI-SLAB was originally developed in 1977 for the Federal Highway Administration (FHWA) and Federal Aviation Administration (FAA) for structural analysis of one- or two-layer concrete pavements, with or without mechanical load transfer systems at joints and cracks.[1] The original finite element ILLI-SLAB model is based on the classical theory of a medium-thick plate on a Winkler foundation, and is capable of evaluating the structural response of a concrete pavement system with joints and/or cracks.[2] It employs the 4-noded, 12-degree of freedom (dof) plate bending element (ACM or RPB 12).[3] The Winkler type subgrade is modeled as a uniform, distributed subgrade through an equivalent mass formulation.[4] The program uses a work equivalent load vector.[3]

ILLI-SLAB has been continually revised and expanded over 10 years through several research studies to improve the program's accuracy and ease of application; to facilitate meaningful interpretation of its results; and to incorporate new foundation models, partial slab-subgrade contact and thermal gradient modeling techniques.[8] The current version (October 10, 1986) is applicable to the structural evaluation of jointed plain concrete pavements. Continuously reinforced and jointed reinforced concrete pavements, however, may also be modeled indirectly.

2.1.2 Basic Assumptions

Assumptions regarding the concrete slab, stabilized base, overlay, dowel bars, and aggregate interlock are briefly summarized as follows:

- Small deformation theory of an elastic, homogeneous medium-thick plate is employed for the concrete slab, stabilized base and overlay. Such a plate is thick enough to carry transverse load by flexure, rather than in-plane force (as would be the case for a thin member), yet is not so thick that transverse shear deformation becomes important. In this theory, it is assumed that lines normal to the middle surface in the deformed plate remain straight, unstretched, and normal to the middle surface of the deformed plate. Each lamina parallel to the middle surface is in a state of plane stress, and no axial or in-plane shear stress develops due to loading.

- In the case of a bonded stabilized base or overlay, full strain compatibility is assumed at the interface. For the unbonded case, shear stresses at the interface are neglected.

- Dowel bars at joints are linearly elastic, and are located at the neutral axis of the slab.

- When aggregate interlock is specified for load transfer, load is transferred from one slab to an adjacent slab by shear. However, with dowel bars some moment as well as shear may be transferred across the joints. The "aggregate interlock factor" can range from 0 to more than 1.00E+08 for associated deflection load transfer efficiencies of 0 to 100 percent. This relationship is quite complex and markedly nonlinear.

2.1.3 Capabilities

Various types of load transfer systems, such as dowel bars, aggregate interlock or a combination of these can be considered at slab joints and cracks. The model can also accommodate the effect of another layer such as a stabilized base or an overlay, either with perfect bond or no bond. Thus, ILLI-SLAB provides several options that can be used in analyzing the following design and rehabilitation problems:

- Multiple wheel and axle loads in any configuration, located anywhere on the slab.

- A combination of slab arrangements such as multiple traffic lanes, traffic lanes and shoulders, or a series of transverse cracks such as in continuously reinforced concrete pavements.

- Jointed concrete pavements with longitudinal and transverse cracks with various load transfer systems.

- Variable subgrade support, including complete loss of support over any specified portion of the slab.

- Concrete shoulders with or without tie bars.

- Pavement slabs with a stabilized or lean concrete base, or asphalt or concrete overlay, assuming either perfect bond or no bond between the two layers.

- Concrete slabs of varying thicknesses and moduli of elasticity, and subgrades with varying moduli of support.

- A linear temperature gradient in uniformly thick slabs. (Currently the subgrade must be modeled as a Winkler type and only a one-layer, one-slab system is permitted.)

- Partial contact of the slab with the subgrade with or without initial gaps using an iterative scheme. (Currently only a one-slab system is recommended.)

2.1.4 Input and Output

The program input includes:

- Geometry of the slab or slabs and mesh configuration.
- Load transfer system at the joints and cracks.
- Elastic properties and thickness of concrete, stabilized base or overlay.
- Subgrade type and stiffness.
- Applied loads, tire pressure, etc.
- Difference in temperature between bottom and top of slab (if desired).
- Density of the slab (if needed).
- Initial subgrade contact condition and amount of gap at each individual node (if desired).

The output produced by ILLI-SLAB includes:

- Nodal deflections and rotations.
- Nodal vertical reactions at the subgrade surface.
- Nodal normal, shear and principal stresses in the slab, and stabilized base or overlay at the top and bottom of each layer.
- Reactions on the dowel bars (if dowels are specified).
- Shear stresses at the joints for aggregate interlock and keyed joint systems.
- Summary of maximum deflections and stresses and their location.

The ILLI-SLAB model has been extensively verified by comparison with the available theoretical solutions and the results from experimental studies.[5,6,7,8] This type of thorough verification is unmatched by other programs considered in this report. The program results have been shown to be highly dependent on element size and aspect ratio (ratio of long to short side of element). Detailed sensitivity analyses have been performed on these parameters to determine the range and limitations of the program. Recommended parameter limits are discussed in section 2.1.8.

2.1.5 Design Optimization

ILLI-SLAB is capable of assisting the design engineer to optimize pavement design, performance, and costs. A small sample of design situations where ILLI-SLAB can be utilized as a tool in decision making are listed below:

- For a given loading condition the user can vary slab thickness for a maximum allowable stress in the slab. This type of analysis will assist the user to specify required slab thicknesses for various subbase or base types with all other parameters remaining constant.

- Various joint design alternatives can be compared including doweled versus undoweled joints, in combination with varying joint spacings, slab thicknesses, and subbase or overlay characteristics.

- Various mechanical load transfer devices can be compared by varying dowel diameter and spacing with or without the effect of aggregate interlock.

- Various designs of lane widening, tied portland cement shoulders, and monolithic curb/shoulder construction may be analyzed.

- Overlay thickness design considerations are possible, such as asphalt concrete versus bonded and unbonded concrete overlays.

- Various subgrade models may be utilized and the results compared to determine the effects of each model's assumptions. However, the user must have a thorough knowledge of each of these subgrade models.[8]

- For a given subgrade support, it becomes possible to determine the significance of day and night time temperature gradients through various slab thicknesses and strengths.

- The response of pavements subjected to partial contact with or without gaps may be considered.

2.1.6 Limitations

ILLI-SLAB also has limitations. It does not have the ability to consider all types of pavement or all factors that affect a pavement. The more significant limitations are listed below:

- Analyzes jointed reinforced concrete pavements and continuously reinforced concrete pavements only in an indirect way.

- Considers a maximum of two slab layers in addition to the subgrade (for example, slab + subbase or overlay + slab).

- Considers only a single slab, layer, and subgrade model (Winkler) when considering temperature gradients through the slab and gaps between slab and subgrade.

- Does not consider the effects of drainability of the pavement section exists.

- Does not consider volume of vehicle traffic.

- Considers only transverse joints and/or cracks with identical connection and load transfer mechanisms.

- Considers only longitudinal joints and/or cracks with identical connection and load transfer mechanisms.

2.1.7 Detailed Description of Capabilities

The user must be aware of these capabilities and limitations to optimize the use of the program. A detailed explanation of ILLI-SLAB's ability to consider many of the factors in rigid pavement performance is presented.

P.C.C. SLAB

Thickness

The thickness of the slab or slabs is directly input into the program. A constant thickness may be input or thickness may vary within a slab, or slab to slab, by specifying the desired thickness at each particular node. Variation in thickness is limited by the number of nodes which in turn is related to the computer memory available.

Length and Width

ILLI-SLAB allows up to 10 slabs in each of the X and Y directions separated by cracks or joints. More slabs can be accommodated with a minor modification of a program parameter. All joints and cracks must run parallel to, and along the entire length of the X and Y directions. There is no limit to the dimensions of the slabs, although the number of nodes used depends on the computer memory available. Layout of the nodes in each of the slabs considered should follow the suggested mesh fineness, aspect ratio, and other requirements to ensure valid results.[8] Optimization of computer memory can be accomplished by taking advantage of any symmetry that exists in the configuration of the slabs and loaded areas.

Stiffness and Strength

The Poisson's ratio is a direct input to the program and is assumed constant for all slabs considered. The modulus of elasticity may be assumed constant for all slabs or may vary from node to node.

Fatigue Properties

Fatigue properties are not considered in ILLI-SLAB.

Durability

Durability of the concrete slab is not considered in ILLI-SLAB.

BASE/SUBBASE

General

ILLI-SLAB accepts a maximum of two layers in a pavement system on a chosen subgrade. This allows a surface course and one base or subbase layer. In the future, ILLI-SLAB may be expanded to accept a greater number of layers. However, this will also be limited by computer memory available.

Any type of base/subbase, stabilized or unstabilized, for which a modulus of elasticity and Poisson's ratio can be defined may be modeled by ILLI-SLAB. Open-graded bases/subbases can be modeled in this way but the user must realize the modulus of elasticity of untreated open graded layers is a function of its confinement provided by the surrounding layers (i.e., surface layer, subgrade strength). For example, an open-graded layer will respond with a higher modulus of elasticity when confined by a concrete slab or a firm subgrade.

ILLI-SLAB models the slab and base/subbase interface as fully bonded or completely unbonded. In the bonded case, full strain compatibility exists at the interface therefore there is no slip between the two layers. For the unbonded case, shear stresses at the interface are completely neglected. Therefore, the layers are allowed to slip freely.

Stiffness

The Poisson's ratio is a direct input to the program. It is assumed the Poisson's ratio is constant throughout the base or subbase. The modulus of elasticity may be assumed constant throughout the base or subbase or may vary at any node desired. It should be noted that ILLI-SLAB assumes that joints and cracks exist in the base or subbase at each and every joint or crack in the slab above. The assumed joint in the base or subbase has the identical connection and load transfer characteristics that are specified in the slab joint or crack above it.

Durability

Durability of the base or subbase is not considered in ILLI-SLAB. However, the modulus of elasticity can be reduced to reflect loss of stiffness.

Erodibility/Loss of Pavement Support

The October 10, 1986 version of ILLI-SLAB and previous versions can only indirectly model pavements with loss of support by reducing the subgrade modulus k at the node where a void is assumed. The user must be aware that a void under an unloaded pavement may be closed by slab deflection when the pavement is loaded and thus create some amount of slab support at that location. Therefore, assumed depth of void, slab deflection, and subgrade modulus must be carefully monitored to produce reliable results. Knowledge of the subgrade model used with a trial and error solution is required.

The current version of ILLI-SLAB, October 10, 1986, allows the node and amount of initial gap to be input and uses an iterative type technique (identical to WESLIQID) to determine the final subgrade contact condition and slab response. However, this new technique assumes either full support or no support at a given node. The Winkler type subgrade model is used in this analysis.

Drainability

Drainability of the base or subbase is not considered in ILLI-SLAB.

Layer Thickness

The base or subbase thickness is directly input into the program. A constant thickness may be input or the thickness may vary by inputting the desired thickness at each particular node.

SUBGRADE

General

ILLI-SLAB allows the user a choice of five different models of subgrade support: the linear discontinuous type (Winkler, Springs, Resilient), and the support systems that do not behave in the linear discontinuous fashion (elastic solid and Vlasov foundations).

The Winkler and Springs models use the modulus of subgrade reaction "k" obtained from the static plate load test to characterize the subgrade. However, most subgrade support systems display a stress level dependent load-deflection response. The resilient, Boussinesq, and Vlasov subgrade models in ILLI-SLAB attempt to represent the subgrade in these more realistic responses.[7,9]

The resilient subgrade model was developed to account for subgrade behavior in response to rapidly moving, repeated loads.[7] This model is based on the concept of the resilient stress dependent modulus of subgrade soils.[10] K_R is analogous to the subgrade reaction, "k", from the plate load test, but is defined by:

K_R = rapidly applied plate pressure/resilient recoverable plate deflection.

Algorithms were developed relating K_R and the level of resilient deflection using ILLI-PAVE, a finite element program in which subgrade material stress-strain relations from repeated, impulse-type tests are employed.[7,11,10] These algorithms form a part of an iterative scheme in ILLI-SLAB according to which a selected initial value of K_R (depending on subgrade type) is corrected after each iteration to correspond to the deflection level obtained.

After two or three iterations, the values are negligibly different. The following broad subgrade types within the resilient subgrade type are a part of ILLI-SLAB:

1. Very soft (K_R = 300 psi/in) (8.3 kg/cm^3).
2. Soft (K_R = 425 psi/in) (11.8 kg/cm^3).
3. Medium (K_R = 725 psi/in) (20.0 kg/cm^3).
4. Stiff (K_R = 1000 psi/in) (27.7 kg/cm^3).
5. Other: The user specifies individual regression parameters to obtain a different K_R versus deflection relation.

The Boussinesq elastic solid idealization in ILLI-SLAB is based upon a procedure described by Cheung and Zienkiewicz.[12] They assume a piecewise uniform approximation to the subgrade reaction, and form the subgrade stiffness matrix by inverting the flexibility matrix obtained by Boussinesq's theory. Cheung and Zienkiewicz claim that this is an adequate approximation but that this is only true for square areas of influence (aspect ratio = 1).[12] Significant deterioration of the approximation occurs for other values of the aspect ratio (length to width ratio of element)[8]. The Vlasov two-parameter subgrade model uses the concept of strain energy to derive the necessary stiffness matrices. For such a foundation, subgrade reaction is related to surface deflection and a foundation constant describing the shear interaction between adjacent springs. Once again, an aspect ratio = 1 should be used to avoid error.[8]

The Winkler, Springs, Boussinesq, and Vlasov subgrade models are listed in ILLI-SLAB as follows:

Winkler:	This is a stress independent, uniform Winkler subgrade modeled as a uniform, distributed subgrade through an equivalent mass formulation.
Springs:	Support is provided by a spring at each of the four corners of the elements (stress independent).
Boussinesq:	An elastic solid foundation (some restrictions apply).
Vlasov:	A two parameter foundation.

A detailed explanation and investigation of each of these foundation models and their correlation with one another and with the Westergaard analysis is available in reference 8.

Stiffness

The user must specify the type of subgrade model desired in the analysis. Depending on this choice the necessary parameters must be input. For example, when the Winkler subgrade model is chosen the user may enter a constant subgrade modulus, k, or may vary the subgrade modulus by specifying the desired k at each particular node. If the elastic solid subgrade is chosen Poisson's ratio and the modulus of elasticity must be directly input and is assumed constant for the entire subgrade.

Several numerical analyses comparing the previously listed types of subgrade models have been performed. For each subgrade model detailed recommendations for input data have been developed. When following these recommendations more reliable results from ILLI-SLAB can be expected.

Drainability

Drainability of the subgrade is not considered in ILLI-SLAB.

Moisture Sensitivity

Moisture sensitivity of the subgrade is not directly considered. However, if a relationship between the subgrade modulus and moisture content can be established, this factor may be considered by running ILLI-SLAB and varying the subgrade modulus.

Volume Change Potential

Volume change potential of the subgrade is not considered in ILLI-SLAB.

Shoulders

General

Shoulders can be directly considered by placing the shoulder slab adjacent to the traffic lane and using the various load transfer options available.

Materials

The modulus of elasticity may be assumed constant or may vary at any node desired. Poisson's ratio of the different slabs must be assumed equal to that of the traffic lane. The base material for the shoulders may have a constant or variable modulus of elasticity but must have a Poisson's ratio equal to the Poisson's ratio of the base material under all slabs.

Thickness and Geometry

The thicknesses of the shoulder, surface and subbase courses may be input as constants or may vary by specifying the desired thicknesses at each particular node. There are only two limitations to the geometry of the shoulder:

- Only rectangular slabs may be considered.
- The computer memory available.

Reinforcement

Reinforcement in the shoulders may be assumed as explained in the Reinforcing Steel Section.

Tying with Mainline and Jointing System

The shoulder may be tied to the mainline with tie bars.

REINFORCING STEEL

General

ILLI-SLAB can indirectly consider continuously reinforced and jointed reinforced concrete pavements by assuming that transverse and or longitudinal cracks exist at short spacings. Load transfer can then be assumed by tie bars (or welded wire mesh) and/or aggregate interlock to any load transfer efficiency desired. With this assumption, the user must have a thorough knowledge of medium-thick plate theory since extremely short spacings violate this theory and erroneous results may occur. It should be noted that ILLI-SLAB neglects any additional bending stiffness in the slab that would be provided by the reinforcement.

LOAD TRANSFER AT JOINTS

Aggregate Interlock and Mechanical Devices

The type of load transfer must be specified in ILLI-SLAB. Aggregate interlock, dowels, or a combination of the two are available. When specifying aggregate interlock load transfer, any efficiency can be obtained in an indirect fashion. The user must assume an "aggregate interlock factor" and then determine deflection load transfer efficiency (defined as the ratio of deflection of the unloaded slab to the loaded slab) from the deflection output. The "aggregate interlock factor" may then be increased or decreased to adjust the associated deflection load transfer efficiency. This trial-and-error procedure may be continued until the desired efficiency is achieved.

When dowels or reinforcement bars are specified. The following parameters must be directly input into ILLI-SLAB:

- Joint width.
- Modulus of elasticity of dowel bars.
- Dowel bar inside diameter (if hollow).
- Dowel bar outside diameter.
- Dowel concrete interaction.
- Dowel spacing.

Dowel concrete interaction must be calculated by either a three dimensional analysis or Friberg's analysis. The appropriate equations are listed in the ILLI-SLAB input guide. If the Friberg's analysis is used an additional parameter, the modulus of dowel support, is required for the calculation.

When a combination of aggregate interlock and dowels are specified, both aggregate interlock factor, and dowel parameters are required. Once again the trial-and-error procedure of varying the aggregate interlock factor can be used to achieve any deflection load transfer efficiency desired, bounded by the minimum provided by the dowels to a maximum of 100 percent.

JOINT DESIGN

Longitudinal and Transverse Spacing of Joints

Any joint spacing desired may be specified in either the longitudinal or transverse directions. However, ILLI-SLAB assumes a joint to run across the full length of all slabs in the analysis. Thus, staggering of joints in adjacent slabs or skewing of joints is not permitted.

Sealant

Joint sealant reservoir design and sealant properties are not considered in ILLI-SLAB.

SHRINKAGE, CURLING and WARPING

Curling

ILLI-SLAB is capable of considering a linear temperature gradient in uniformly thick slabs. Currently the temperature gradient option is limited to the Winkler type subgrade model and only a one-layer, one-slab system is permitted. The iterative type method of computation of these thermal effects is identical to that used in the WESLIQID program and is explained in reference 24.

Shrinkage and Warping

The physical effects of shrinkage and moisture warping and their associated stresses are not considered in ILLI-SLAB.

DRAINAGE SYSTEM EFFECTIVENESS

ILLI-SLAB does not directly consider any drainage effects. However, if a relationship between subgrade support modulus, k, and the moisture content of the subgrade can be established, the effects of drainage may be considered.

CLIMATE

ILLI-SLAB does not consider any climatic effects except thermal gradients through the slab.

TRAFFIC LOADINGS

Truck Volume

Truck volume is not directly considered in ILLI-SLAB. It may be considered by using the tensile stress in the slab calculated by ILLI SLAB and the modulus of rupture in a fatigue cracking prediction equation.

Axle Load Distribution

ILLI-SLAB has no restrictions on tire contact area, tire pressure, or quantity and location of loaded wheels. This wide range of choice allows a pavement analysis for any type of vehicle with any type of loading. However, only rectangular loaded areas are permitted.

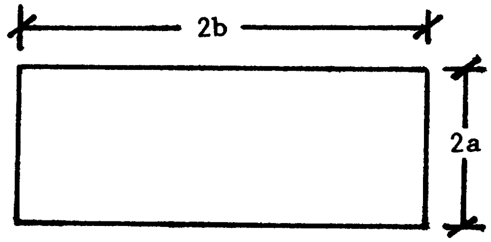
Truck Lane Distribution and Lateral Wander

ILLI-SLAB is capable of considering only one loading condition for each computer run. Therefore no direct statistical analysis on truck wander is computed. However, the vehicle considered may be positioned at any location desired. This allows the user to conduct an analysis by generating ILLI-SLAB results for all possible vehicle positions on a given pavement and correlate these results with the statistical distribution of the vehicle position in the lane.

2.1.8 Calculations

To use the ILLI-SLAB program in analyzing pavements, the designer must determine the quantity, orientation, and dimensions of the slab or slabs he wishes to model and the total computer memory available. The pavement considered must then be subdivided into elements, each element having four nodes. This is subdivision referred to as a mesh. The following general criteria should be used to optimize computer memory available and accuracy of results. Detailed limitations and justifications of these criteria are discussed in reference 8.

Typical element:



2a - short side of element
2b - long side of element

Mesh recommendations:

- Aspect ratio (2b/2a), should be less than 3.0 .
- In vicinity of 2 x loaded area, 2a/h should be approximately equal to 0.8.

2a - short side of element
2b - long side of element
h - slab thickness.

Memory use:

Computer storage capacity > $26 * Nx * Ny^2$

Nx - No. of nodes in X direction
Ny - No. of nodes in Y direction

Note: This equation was developed when using the Winkler subgrade model. A significantly greater storage capacity is required by the Boussinesq elastic solid model.

Figures 1, 2, 3, and 4 present the corner, transverse edge, longitudinal edge, and interior loading condition meshes used in calculations, respectively. Figure 5 shows the cross section of the pavement.

Joints are incorporated by specifying the axis coordinate, or node, along the desired axis, twice, thus generating a set of nodes for each side of the joint.

ILLI-SLAB allows the designer to consider all possible loading conditions to determine the critical fatigue location, which may not necessarily occur at the same location as the critical stress or critical deflection. Four loading conditions with load transfer by dowels, aggregate interlock, and free edges were analyzed by ILLI-SLAB. These results are summarized in tables 1 and 2. The input data for the one layered pavement and a two-layered pavement are listed in tables 3 and 4.

In general, the two layered pavement increased maximum deflection, reduced maximum subgrade stress and held constant or slightly increased maximum tensile stress in the slab when comparing the results to that of the one-layered pavement analysis.

When doweled transverse joints and longitudinal tied shoulders were compared to no load transfer (aggregate interlock = 0), the maximum tensile stress in the slab was reduced in all loading conditions except the interior. The maximum subgrade stress and maximum deflection was reduced significantly except in the interior loading condition, where no change was achieved. The corner loading condition was most affected and the longitudinal edge loading condition being the least affected. The aggregate interlock load transfer system had the same general effects as when dowels were used except that it was slightly more effective in reducing stresses and deflection with the given minimum load transfer assumed. This is just a small sample of the comparisons that the design engineer can make to create an optimum design using ILLI-SLAB as a tool.

Day and night time thermal gradients were analyzed using the ILLI-SLAB program. Due to program limitations a single slab, single layer system, with 12-by 12-in (30.48 cm by 30.48 cm) square mesh was used for all thermal gradient analyses. Slab size and thickness, and subgrade stiffness were varied for a typical day and night time temperature gradient of 3.0 and 1.5 (-16.1 and -16.9 degrees celsius) degrees Fahrenheit per inch of slab thickness respectively. Maximum principal tension stresses and additional input required is listed in table 5. The results illustrate the strong significance of slab size, slab thickness and subgrade strength on stresses induced by thermal gradients. The results also correlate very well with the identical thermal gradient analyses documented in the Zero-Maintenance investigation.[46,47]

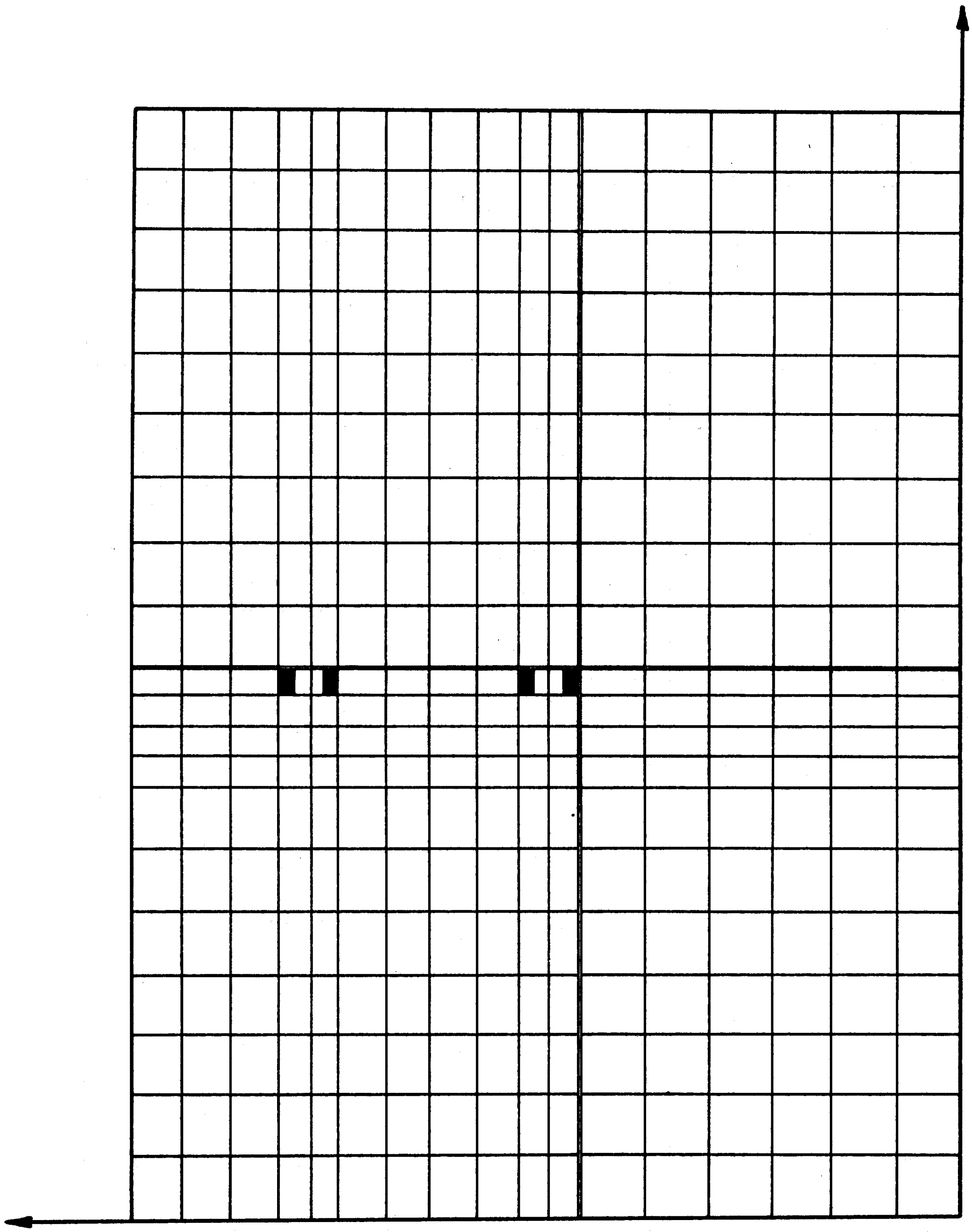


Figure 1. Corner loading mesh for finite element analysis.

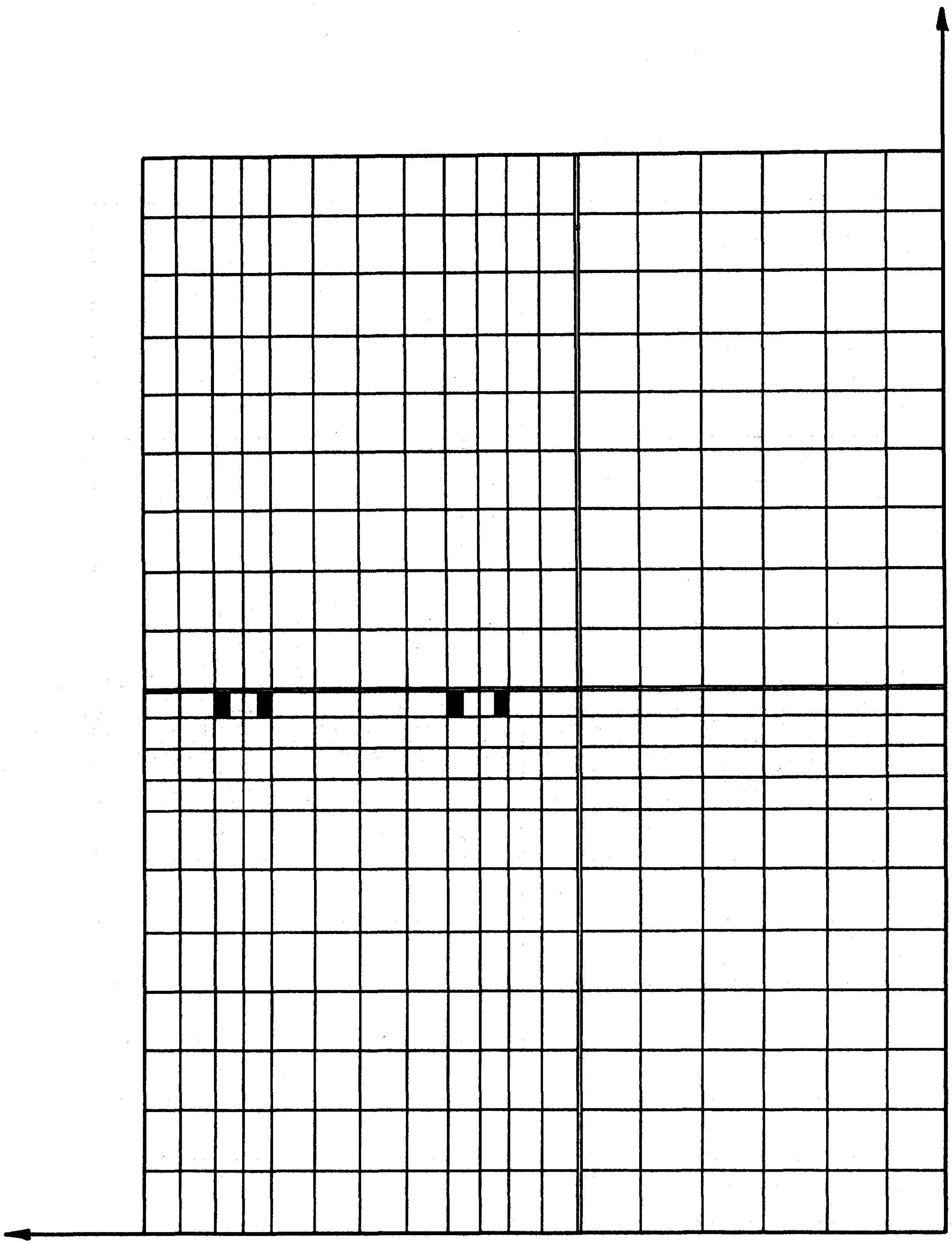


Figure 2. Transverse edge loading mesh for finite element analysis.

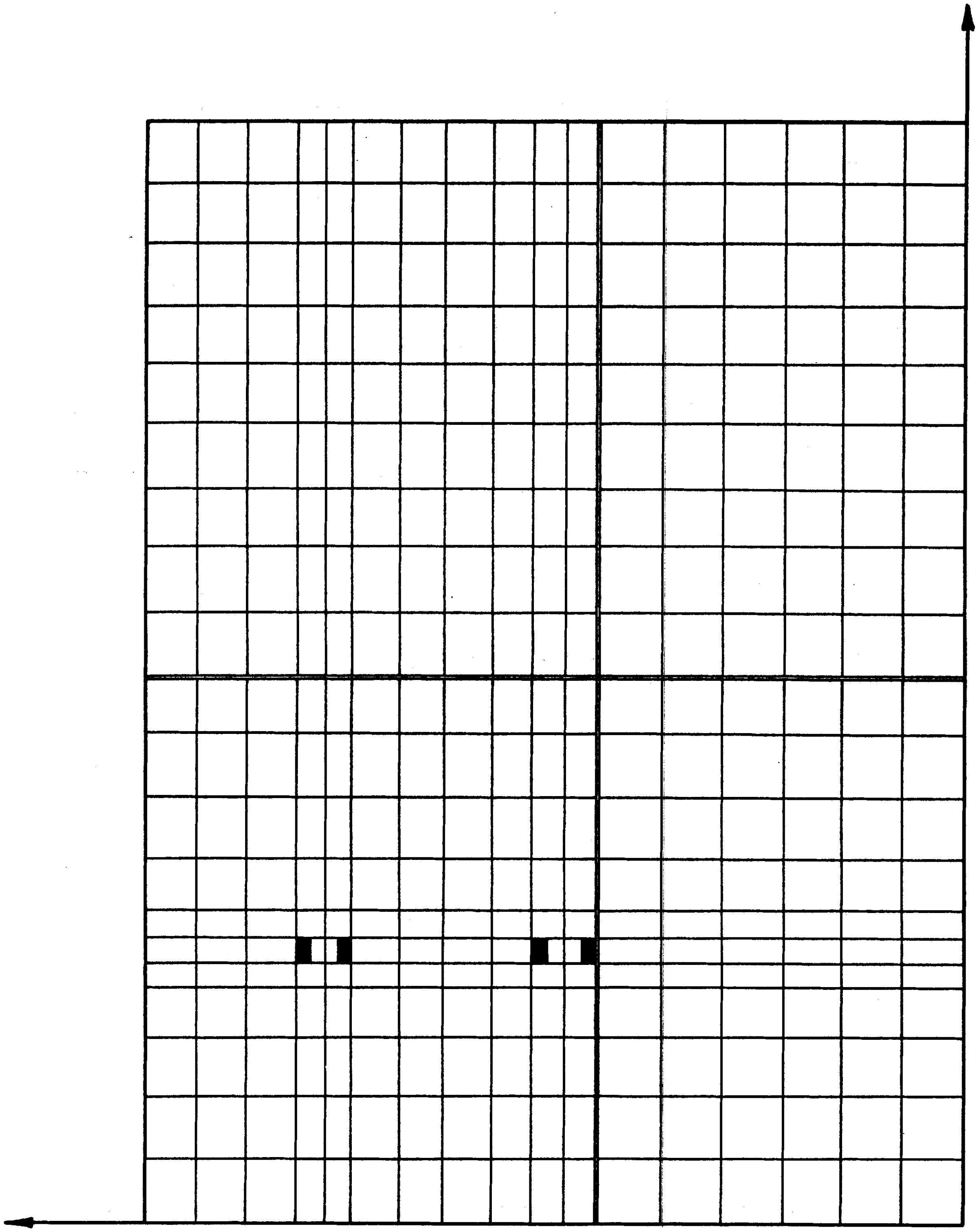


Figure 3. Longitudinal edge loading mesh for finite element analysis.

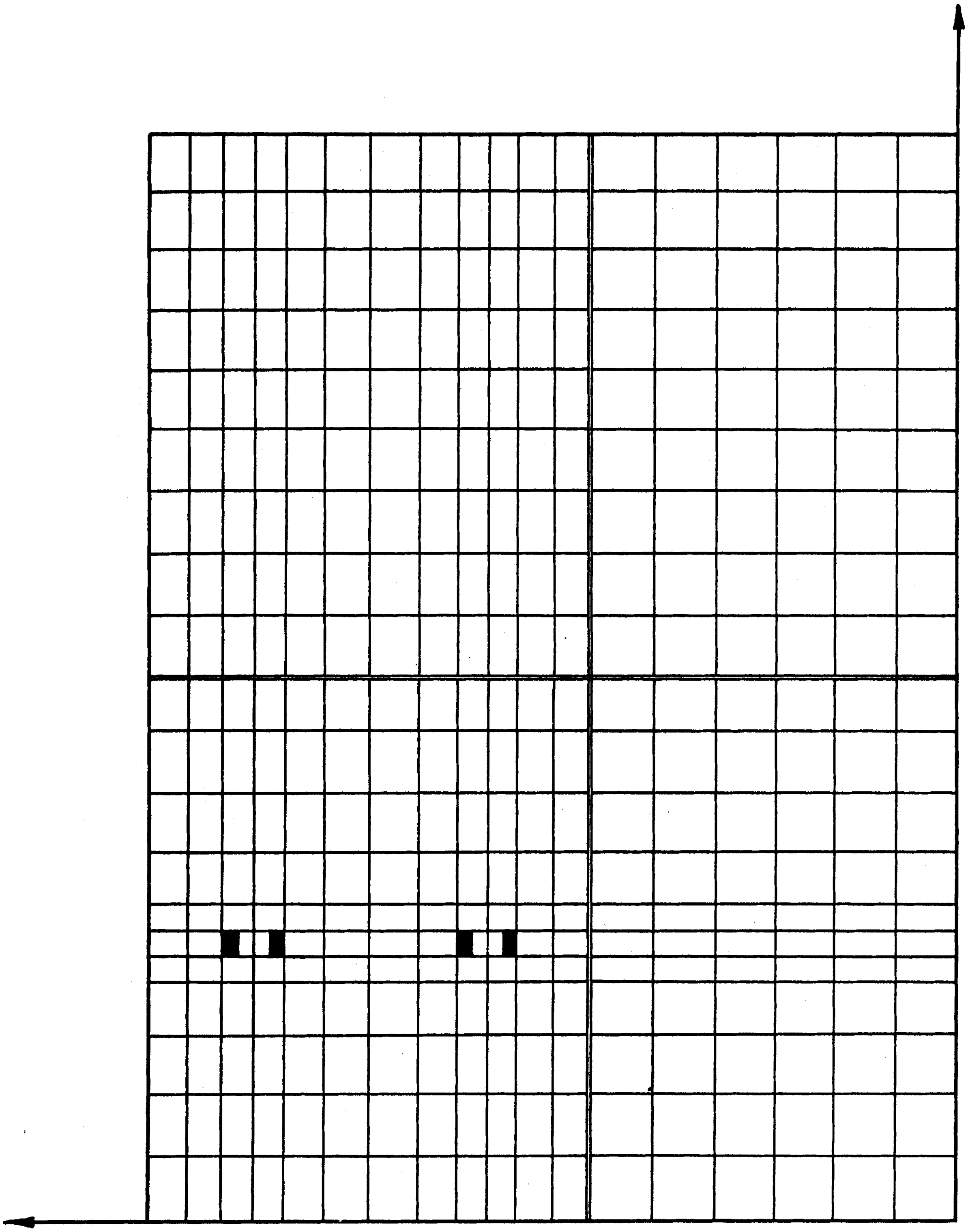
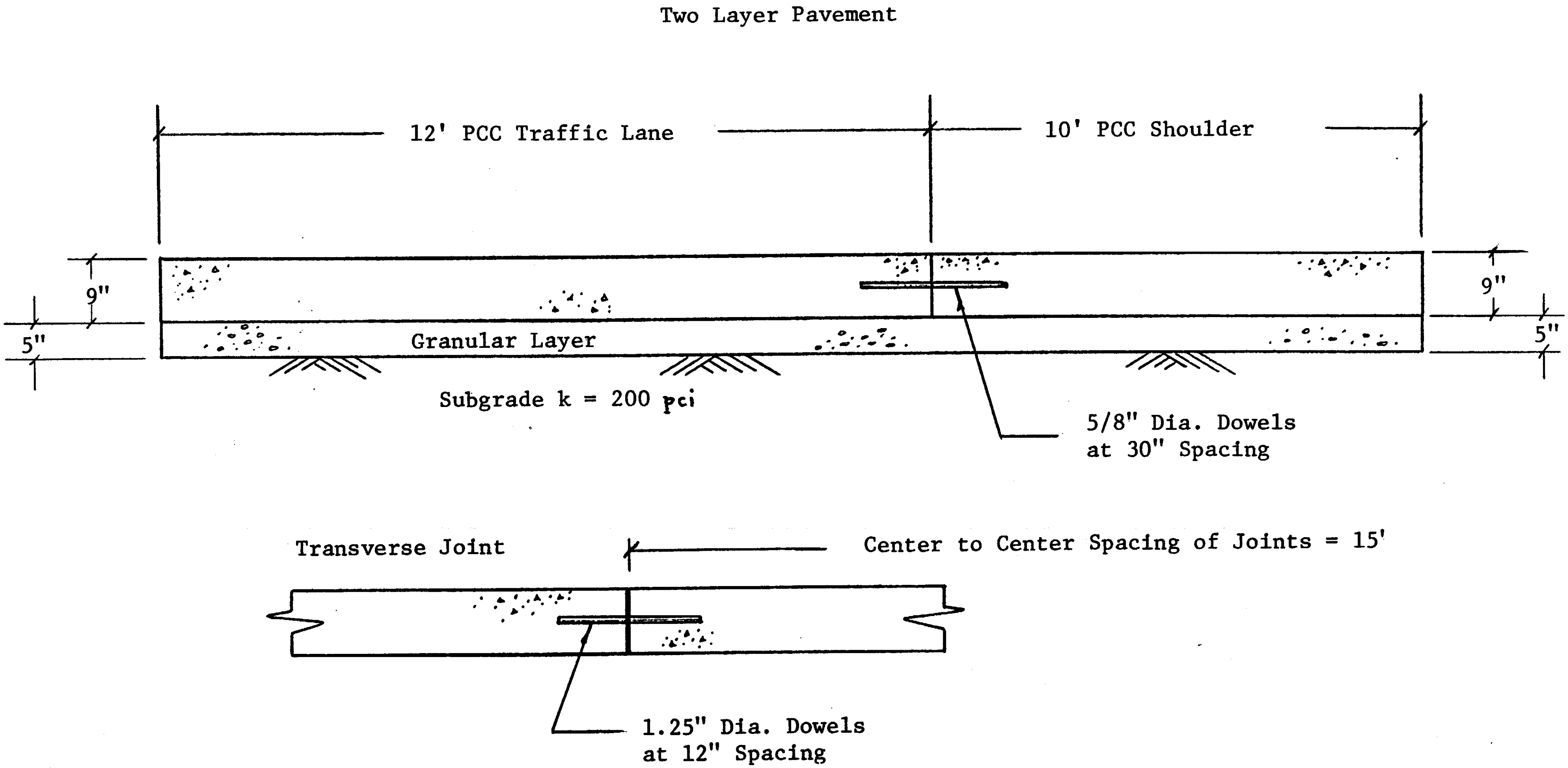


Figure 4. Interior loading mesh for finite element analysis.



1 inch = 2.54 cm
1 foot = 0.3048 m
1 psi/inch = 0.2768 kg/cm^3

Figure 5. Cross section of example concrete pavement.

Table 1. ILLI-SLAB output results for corner and transverse edge loading.

PROGRAM: ILLI-SLAB
RESULTS FOR 1-AND 2-LAYER PAVEMENT SYSTEMS
CORNER AND TRANSVERSE EDGE LOADING CONDITIONS (SEE FIGURES 1 AND 2)

LOADING CONDITION	NO. OF LAYERS	DOWELS	AGGREGATE INTERLOCK FACTOR TRANS	AGGREGATE INTERLOCK FACTOR LONG	MINIMUM LOAD TRANSFER TRANS [%]	MINIMUM LOAD TRANSFER LONG [%]	MAXIMUM PRINCIPAL TENSION STRESS BOT/SLAB [PSI]	MAXIMUM PRINCIPAL TENSION STRESS BOT/BASE [PSI]	MAXIMUM SUBGRADE STRESS [PSI]	MAXIMUM DEFLECTION [MILS]
CORNER	1	NO	100000	31570	86%	65%	103.3		2.5	12.3
	1	YES			88%	51%	98.8		2.7	13.3
	1	NO	0	0	0%	0%	* 195.1		7.6	38.2
UNBONDED	2	NO	100000	31570	86%	65%	97.9	21.8	2.4	12.1
UNBONDED	2	YES			88%	50%	93.6	20.8	2.6	13.1
UNBONDED	2	NO	0	0	0%	0%	* 184.7	* 41.0	7.5	37.4
BONDED	2	NO	100000	31570	85%	61%	* 51.9	37.1	1.9	9.7
BONDED	2	YES			87%	47%	* 54.2	35.2	2.1	10.5
BONDED	2	NO	0	0	0%	0%	* 119.4	37.6	5.8	28.9
TRANSVERSE	1	NO	100000	31570	91%	65%	112.9		2.4	12.1
EDGE	1	YES			93%	50%	107.6		2.4	12.1
	1	NO	0	0	0%	0%	126.5		4.7	23.7
UNBONDED	2	NO	100000	31570	91%	65%	106.8	23.7	2.4	11.9
UNBONDED	2	YES			93%	50%	101.7	22.6	2.4	11.9
UNBONDED	2	NO	0	0	0%	0%	118.8	26.4	4.7	23.3
BONDED	2	NO	100000	31570	90%	61%	38.4	39.2	1.9	9.7
BONDED	2	YES			92%	46%	* 40.2	37.3	2.0	9.8
BONDED	2	NO	0	0	0%	0%	* 83.8	* 40.3	3.9	19.3

* CRITICAL TENSION STRESS SHOWN IS IN TOP OF LAYER

1 psi = 0.07031 kg/cm^2
1 mil = 0.0254 mm

Table 2. ILLI-SLAB output results for longitudinal edge and interior loading.

PROGRAM: ILLI-SLAB
RESULTS FOR 1-AND 2-LAYER PAVEMENT SYSTEMS
LONGITUDINAL EDGE AND INTERIOR LOADING CONDITIONS (SEE FIGURES 3 AND 4)

LOADING CONDITION	NO. OF LAYERS	DOWELS	AGGREGATE INTERLOCK FACTOR TRANS	AGGREGATE INTERLOCK FACTOR LONG	MINIMUM LOAD TRANSFER TRANS [%]	MINIMUM LOAD TRANSFER LONG [%]	MAXIMUM PRINCIPAL TENSION STRESS BOT/SLAB [PSI]	MAXIMUM PRINCIPAL TENSION STRESS BOT/BASE [PSI]	MAXIMUM SUBGRADE STRESS [PSI]	MAXIMUM DEFLECTION [MILS]
LONG. EDGE	1	NO	100000	31570	66%	74%	181.1		1.6	8.0
	1	YES			68%	60%	195.3		1.7	8.7
	1	NO	0	0	0%	0%	240.1		2.8	14.0
UNBONDED	2	NO	100000	31570	98%	74%	172.3	38.3	1.6	7.9
UNBONDED	2	YES			83%	60%	185.7	41.3	1.7	8.6
UNBONDED	2	NO	0	0	0%	0%	227.7	50.6	2.8	13.7
BONDED	2	NO	100000	31570	72%	72%	66.8	68.1	1.3	6.3
BONDED	2	YES			80%	57%	71.9	73.4	1.4	6.8
BONDED	2	NO	0	0	0%	0%	83.8	85.5	2.2	10.8
INTERIOR	1	NO	100000	31570	70%	76%	136.6		1.4	7.0
	1	YES			77%	61%	136.9		1.4	7.0
	1	NO	0	0	0%	0%	139.0		1.4	7.0
UNBONDED	2	NO	100000	31570	68%	76%	130.0	28.9	1.4	6.9
UNBONDED	2	YES			75%	61%	130.2	28.9	1.4	6.8
UNBONDED	2	NO	0	0	0%	0%	132.3	29.4	1.4	6.8
BONDED	2	NO	100000	31570	91%	72%	50.4	51.5	1.2	5.8
BONDED	2	YES			92%	56%	50.8	51.8	1.2	5.8
BONDED	2	NO	0	0	0%	0%	51.0	52.0	1.1	5.5

* CRITICAL TENSION STRESS SHOWN IS IN TOP OF LAYER

1 psi = 0.07031 kg/cm^2
1 mil = 0.0254 mm

Table 3

Parameters assumed in analysis for one-layered pavement ILLI-SLAB model

TYPE OF PAVEMENT	JPCP
SURFACE LAYER	
PCC SLAB THICKNESS	9 IN
POISSON'S RATIO	0.20
MODULUS OF ELASTICITY	5000000 PSI
SUBGRADE	
SUBGRADE MODEL	WINKLER
SUBGRADE MODULUS	200 PSI/IN
DOWEL AND JOINT PARAMETERS	
JOINT WIDTH	0.25 IN
MODULUS OF DOWEL SUPPORT	1500000 PSI/IN
MODULUS OF ELASTICITY OF DOWEL BARS	29000000 PSI
POISSON'S RATIO OF DOWEL BARS	0.30
LONGITUDINAL JOINT	
DOWEL BAR DIAMETER	0.625 IN
DOWEL BAR SPACING	30.0 IN
DOWEL CONCRETE INTERACTION BY FRIBERG'S ANALYSIS	444279
TRANSVERSE JOINT	
DOWEL BAR DIAMETER	1.25 IN
DOWEL BAR SPACING	12.0 IN
DOWEL CONCRETE INTERACTION BY FRIBERG'S ANALYSIS	1514419
LOADING	
TYPE OF AXLE	DUAL WHEEL
GROSS WEIGHT OF AXLE	18.0 KIPS
TIRE PRESSURE	100.0 PSI

1 in = 2.54 cm
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3
1 kip = 454 kg

Table 4

Paramerter assumed in analysis for two-layer pavement ILLI-SLAB model.

TYPE OF PAVEMENT	JPCP	
SURFACE LAYER		
PCC SLAB THICKNESS	9	IN
POISSON'S RATIO	0.20	
MODULUS OF ELASTICITY	5000000	PSI
BASE		
BONDING CONDITION	BONDED/UNBONDED	
THICKNESS	5	IN
POISSON'S RATIO	0.20	
MODULUS OF ELASTICITY	2000000	PSI
SUBGRADE		
SUBGRADE MODEL	WINKLER	
SUBGRADE MODULUS	200	PSI/IN
DOWEL AND JOINT PARAMETERS		
JOINT WIDTH	0.25	IN
MODULUS OF DOWEL SUPPORT	1500000	PSI/IN
MODULUS OF ELASTICITY OF DOWEL BARS	29000000	PSI
POISSON'S RATIO OF DOWEL BARS	0.30	
LONGITUDINAL JOINT		
DOWEL BAR DIAMETER	0.625	IN
DOWEL BAR SPACING	30.0	IN
DOWEL CONCRETE INTERACTION BY FRIBERG'S ANALYSIS	444279	
TRANSVERSE JOINT		
DOWEL BAR DIAMETER	1.25	IN
DOWEL BAR SPACING	12.0	IN
DOWEL CONCRETE INTERACTION BY FRIBERG'S ANALYSIS	1514419	
LOADING		
TYPE OF AXLE	DUAL WHEEL	
GROSS WEIGHT OF AXLE	18	KIPS
TIRE PRESSURE	100	PSI

1 in = 2.54 cm
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3
1 kip = 454 kg

Table 5. ILLI-SLAB curling analysis results.

PROGRAM: ILLI-SLAB
RESULTS FOR CURLING ANALYSIS WITHOUT TRAFFIC LOAD
MAXIMUM PRINCIPAL SLAB STRESS

SLAB THICKNESS (INCH)	SLAB SIZE (FT.)	SUBGRADE "K" VALUE (PCI) 50	50	200	200	500	500
		TEMPERATURE GRADIENT (DEGREES F/INCH) 1.5	-3.0	1.5	-3.0	1.5	-3.0
8.0	12 × 15	67.0	117.3	117.6	180.5	145.2	218.9
8.0	12 × 20	120.5	222.7	165.3	283.2	182.1	301.5
10.0	12 × 15	51.1	91.7	98.9	152.5	126.5	183.4
10.0	12 × 20	107.2	198.7	164.5	276.6	191.9	315.2
10.0	12 × 25	157.8	302.7	203.7	369.5	218.8	387.5
10.0	12 × 30	190.7	373.1	217.1	408.5	221.1	417.9
14.0	12 × 15	30.4	58.2	69.1	108.2	91.7	131.2
14.0	12 × 20	76.0	143.2	133.2	216.2	163.9	240.1

Note: A positive temperature gradient is defined as slab bottom temperature higher than slab top temperature.

Unit Weight of Concrete = 0.0868 lb/in^3
Concrete Coefficient of Expansion = 5.0x10^-6 in/in/F

1 inch = 2.54 cm
1 foot = 0.3048 m
1 psi/inch = 0.02768 kg/km^3
°F = (°C x 1.8) + 32

2.2 JSLAB Finite Element Model

2.2.1 Introduction

JSLAB was developed to model jointed concrete pavements using the finite element technique.[22] The basic assumptions and derivation are very similar to ILLI-SLAB. The subgrade is modeled as a Winkler-type dense liquid, through an equivalent mass formulation as in ILLI-SLAB.[22]

2.2.2 Basic Assumptions

Assumptions in the development of JSLAB are identical to ILLI-SLAB (see 2.1.2) with the following exceptions:

- The subgrade is assumed to be a series of uniformly distributed springs under each element. The stiffness matrix in both JSLAB and ILLI-SLAB for the subgrade considered is based on the work of Tabatabaie.[1] The modified stiffness matrix currently used in ILLI-SLAB was verified by rederiving the subgrade stiffness matrix using the concept of strain energy as opposed to the principle of virtual work.[1] As expected, both approaches yield identical results. However, an error was found in the stiffness matrix that can lead to 3 to 5 percent error in the results.[23] This error is documented in reference 8 in which the error and the modifications to ILLI-SLAB are explained in detail. The stiffness matrix was corrected by following documentation concerning the changes made to ILLI-SLAB and changing the algebraic signs of the same elements of the stiffness matrix that had been changed in ILLI-SLAB. These changes are documented and have produced symmetrical output for all symmetrical loading problems tested to date. These include 1, 4, and 9 slab systems.[8]

2.2.3 Capabilities

JSLAB has identical capabilities as ILLI-SLAB (see 2.1.3) with the exception of the partial contact with initial gap option. In addition, JSLAB has the following capabilities:

- Ability to consider nonuniformly spaced dowels across the longitudinal or transverse joints.

- Consideration of noncircular load transfer devices.

2.2.4 Input and output

The program input includes:

- Geometry of the slab or slabs, and mesh configuration.
- Load transfer system at the joints.

- Elastic properties and thickness of PCC slab, stabilized base or overlay.
- Subgrade stiffness.
- Unit weight of concrete and temperature gradient through slab.
- Initial slab displacements (if not zero).
- Applied loads.

The output produced by JSLAB includes:

- Dowel shear and moment at each node along joint.
- Nodal deflections and rotations.
- Nodal bending stresses along X and Y axes.
- Nodal shear stress in XY plane
- Nodal vertical component of applied load.
- Nodal moment component of applied load along X and Y axes.

The user must manually calculate the principal stresses to obtain the maximum bending stress in the slab, overlay or base, for each loading condition. In some symmetric loading conditions the principal axes will be coincident with the X and Y axes.

2.2.5 Design Optimization

JSLAB may be used in all of the design situations listed in the ILLI-SLAB discussion (see 2.1.5) with the exception that only one subgrade model, the Winkler dense liquid, is employed by JSLAB. The additional design situations in which JSLAB may assist the engineer are listed:

- Various configurations of nonuniformly spaced dowels across joints can be compared with or without the effect of aggregate interlock.

- Consideration of warping effects due to top and bottom variation in pavement moisture content by an equivalent temperature gradient.

2.2.6 Limitations

The significant limitations in JSLAB include those listed in the ILLI-SLAB discussion (see section 2.1.6) with the exception of the limitation regarding moisture gradients through the slab. Other significant limitations to JSLAB are listed below:

- The principal bending stresses are not calculated.

- The vertical stress on the subgrade is not calculated.

- Only a one-layer pavement system with a uniform thickness can be analyzed when a moisture gradient through the slab is considered.

- Two program runs are required for thermal gradient analyses, once taking slab weight into consideration and once with slab weight equal to zero, and subtracting the resulting stresses at each particular node by hand to obtain the final stress caused by thermal gradients through the slab. This type of analysis requires a significant increase in computer and interpretation time for the user.

- When vertical slab displacements are specified, applied loads cannot be located at that particular node or over any element adjacent to that node.

2.2.7 Detailed Description of Capabilities

JSLAB's ability to consider the factors that affect the performance of rigid pavements are often identical to ILLI-SLAB. Therefore references to section 2.1.7 will be made when appropriate. A detailed explanation of the additional capabilities is presented.

P.C.C SLAB

JSLAB's capabilities in regard to rigid pavement thickness, length and width, stiffness and strength, are identical to ILLI-SLAB (see 2.1.7). As in ILLI-SLAB, fatigue properties and durability of the Portland cement concrete are not considered.

BASE/SUBBASE

JSLAB accepts a maximum of two layers in a pavement system on a Winkler dense liquid subgrade. This allows a surface course and one base or subbase layer. The capabilities in regard to slab interface friction, stiffness, durability, drainability, and layer thickness of the base or subbase is identical to ILLI-SLAB (see 2.1.7). Erodability or loss of pavement support can be indirectly modeled as in ILLI-SLAB (see 2.1.7) with the additional option of specifying a vertical slab displacement at any given node. However, if the user chooses to specify vertical displacements of the slab then applied loads cannot be specified at that node or any element adjacent to that node.

SUBGRADE

JSLAB employs the Winkler type dense liquid subgrade, modeled as a uniform, distributed subgrade through an equivalent mass formulation as in ILLI-SLAB subgrade type 6 (see 2.1.7).[4] Moisture sensitivity of the subgrade may be indirectly considered as in ILLI-SLAB. Drainability and volume change potential of the subgrade is not considered in JSLAB.

SHOULDERS

JSLAB's capabilities in regard to materials, thickness, geometry, and reinforcement in the shoulder are identical to ILLI-SLAB (see 2.1.7). Tying the shoulder to mainline and other jointing systems involved may be modeled as discussed in ILLI-SLAB (see 2.1.7), with the additional options of noncircular and or nonuniformly spacing the dowels.

REINFORCING STEEL

JSLAB can indirectly consider continuously reinforced and jointed reinforced concrete pavements with the identical limited approach as discussed in ILLI-SLAB (see section 2.1.7).

LOAD TRANSFER AT JOINTS

Aggregate interlock, dowels or a combination of the two are available in JSLAB. The modeling and input required is identical to ILLI-SLAB (see section 2.1.7) with the addition of noncircular dowels and of unequal dowel spacing capabilities. When noncircular dowels are specified the user must include the cross-sectional area and moment of inertia of the noncircular dowel in addition to the input discussed in section 2.1.7.

JOINT DESIGN

The longitudinal and transverse joint spacings, sealant, and load transfer design is identical to ILLI-SLAB (section 2.1.7).

SHRINKAGE, CURLING, AND WARPING

Shrinkage

The physical effects of material shrinkage with time are not considered in JSLAB.

Curling

JSLAB is capable of considering a linear temperature distribution in a single layer pavement system of uniform thickness. This is modeled by application of a calibrated moment along the slab edges due to the temperature variation.[22] The temperature gradient is considered positive when the higher temperature is at the top surface. This is the opposite of ILLI-SLAB, WESLIQID, and WESLAYER programs.

Warping

JSLAB can also consider a linear moisture distribution in a single layer pavement system of uniform thickness. These warping effects are modeled by inducing an equivalent temperature gradient in the slab.[22] A negative equivalent temperature gradient is assumed to represent a slab top drier than slab bottom.

DRAINAGE SYSTEM EFFECTIVENESS

JSLAB does not directly consider any drainage effects.

CLIMATE

JSLAB is only capable of considering temperature and moisture linear distributions through the PCC slab for one layered uniformly thick pavement systems. Uniform temperature or moisture changes and other climatic changes are not considered in JSLAB.

TRAFFIC LOADINGS

JSLAB's capabilities regarding truck volume, axle load distribution and truck lane distribution and lateral wander in lane are identical to ILLI-SLAB section 2.1.7. However, the input required by JSLAB describing the location of the loaded areas is not as convenient for the user.

2.2.8 Calculations

For comparative reasons, the identical parameters used by the ILLI-SLAB analysis were input into JSLAB. The critical longitudinal edgeloading condition was chosen and input into JSLAB. Figure 3 shows the mesh configuration. The results of the program runs are listed in Table 6. When ties (modeled as dowel bars) were inserted across the longitudinal joint the calculated load transfer (defined as the ratio of deflection of the unloaded slab to the loaded slab) was considerably less than the results from ILLI-SLAB. Thus, the effect of dowels across joints is more conservative in JSLAB for a given set of parameters. The layer stresses and slab deflections produced by JSLAB differed by less than 5 percent from ILLI-SLAB results. In general, as the load transfer efficiencies approached the same value these differences decreased.

A curling analysis (the effect of a temperature gradient through the slab) was performed for the exact parameters considered in the ILLI-SLAB analysis. Results are listed in table 7. The JSLAB program generated smaller stresses than ILLI-SLAB and WESLIQID for all given conditions considered. Thus the JSLAB calibrated moments used to model temperature gradients are not conservative when compared to the ILLI-SLAB and WESLIQID results.

Table 6. JSLAB results for standard loading condition specified in figure 3.

PROGRAM: JSLAB
RESULTS FOR 1-AND 2-LAYER PAVEMENT SYSTEMS
LONGITUDINAL EDGE LOADING CONDITION (SEE FIGURES 3)

LOADING CONDITION	NO. OF LAYERS	DOWELS	AGGREGATE INTERLOCK FACTOR TRANS	LONG	MINIMUM LOAD TRANSFER TRANS [%]	LONG [%]	MAXIMUM TENSION STRESS BOT/SLAB [PSI]	BOT/BASE [PSI]	MAXIMUM DEFLECTION [MILS]
LONG. EDGE	1	NO	100000	31570		75%	175.1		8.0
	1	YES				44%	203.2		9.8
	1	NO	0	0	ERATIC	0%	232.3		14.0
UNBONDED	2	NO	100000	31570	BECAUSE	74%	166.6	37.0	7.9
UNBONDED	2	YES			OF	44%	193.2	42.9	9.6
UNBONDED	2	NO	0	0	DISTANCE	0%	220.2	48.9	13.8
BONDED	2	NO	100000	31570	FROM	72%	66.9	68.3	6.3
BONDED	2	YES			LOAD	42%	77.1	78.7	7.7
BONDED	2	NO	0	0		0%	84.1	85.8	10.9

1 psi = 0.07031 kg/cm^2
1 mil = 0.0254 mm

Table 7. JSLAB curling analysis results.

PROGRAM: JSLAB
RESULTS FOR CURLING ANALYSIS WITHOUT TRAFFIC LOAD
MAXIMUM PRINCIPAL SLAB STRESS

SLAB THICKNESS (INCH)	SLAB SIZE (FT)	SUBGRADE "K" VALUE (PCI) 50	50	200	200	500	500
		TEMPERATURE GRADIENT (DEGREES F/INCH) 1.5	-3.0	1.5	-3.0	1.5	-3.0
8.0	12 x 15	52.3	98.6	97.7	141.3	116.7	162.4
8.0	12 x 20	94.0	180.2	127.4	225.0	133.5	233.8
10.0	12 x 15	40.1	77.1	83.5	120.4	109.4	141.8
10.0	12 x 20	84.7	160.4	133.8	221.3	151.2	249.4
10.0	12 x 25	124.1	241.5	160.6	292.5	159.8	300.1
10.0	12 x 30	148.5	249.9	168.1	321.5	166.5	322.4
14.0	12 x 15	23.8	47.8	59.9	88.2	81.6	105.9
14.0	12 x 20	60.1	116.0	114.6	184.2	141.7	204.9

Note: A positive temperature gradient is defined as slab bottom temperature higher than slab top temperature.

Unit Weight of Concrete = 0.0868 lb/in^3
Concrete Coefficient of Expansion = 5.0x10^-6 in/in/F

1 inch = 2.54 cm
1 foot = 0.3048 m
1 psi/inch = 0.02768 kg/cm^3
°F = (°C x 1.8) + 32

2.3 WESLIQID Finite Element Model

2.3.1 Introduction

The WESLIQID finite element computer program was developed for the analysis of concrete pavements subjected to multiple-wheel loads and temperature gradients. The subgrade is modeled as a Winkler (dense liquid) foundation. Thus, only forces and deformations in the vertical directions are considered and each force is proportional to the corresponding vertical deflection. The program can accommodate any number of rectangular shaped slabs, arranged in any arbitrary pattern, connected by dowel bars or other load transfer devices at the joints. The program can also handle cracks perpendicular or parallel to the joints.

2.3.2 Basic Assumptions

Assumptions involved with the development of WESLIQID are briefly summarized as follows:

- The two dimensional finite element method employed is based on the classical theory of medium-thick plates, also employed in ILLI-SLAB. This theory includes the assumption that a plane section before bending remains plane after bending, and there is no variation in vertical deflection along the thickness of the slab; i.e., the deflection at the top of the slab is the same as that at the bottom.

- Assumptions regarding bonding of layers and dowel bars are identical to ILLI-SLAB (see section 2.1.2).

- The weight of the slab may be considered directly.

- The externally applied loads are converted to a system of statically equivalent nodal loads, which often are not work equivalent to the applied loads.[8]

2.3.3 Capabilities

WESLIQID has identical capabilities as ILLI-SLAB (see section 2.1.3) with the addition of the following:

- The ability to consider a linear temperature gradient in slabs of uniform thickness.

- The ability to consider directly partial contact of the slab with the subgrade with or without initial gaps using an iterative scheme.

- The ability to model various load transfer devices across transverse or longitudinal joints (can specify different jointing characteristics for shoulder and traffic lane transverse joints).

- Allows the user a choice to specify: (1) shear and moment efficiencies, (2) a spring constant, or (3) data on dowel bars to model each joint in the slab layout.

- Once subgrade reactive forces at each node are determined, stresses and strains in the subgrade soil may be calculated using Boussinesq's or Burmister's equations, if the user supplies values of subgrade modulus and Poisson's ratio. Superposition of individual nodal forces is employed in this computation.

2.3.4 Input and Output

The program input includes:

- Dimensioning of the matrices.

- Geometry of the slab or slabs.

- Load transfer systems at the joints.

- Elastic properties and thickness of Portland cement concrete slab, stabilized base or overlay including bonding condition (two layers maximum).

- Convergence criteria, i.e., relaxation factor, convergence limits, maximum number of iterations, etc.

- Linear thermal gradient through pavement layers.

- Subgrade stiffness and initial contact condition with slab.

- Slab nodes in which stresses are to be printed.

- Initial thermal stresses and deflections in slab.

- Locations and depths to calculate subgrade stress and deflections.

The output produced by WESLIQID includes:

- Data describing convergence of solution.

- Nodal deflections and rotations desired.

- The maximum nodal X, Y, and principal stresses at specified locations in the slab and or stabilized base or overlay.

- Shear and moment developing at the joints.

WESLIQID results compared well with those of available solutions, such as the Westergaard solution, Pickett and Ray's influence charts (H51 program), and the discrete element method. Comparisons between the percent load transfer across the joint computed by the WESLIQID program and those measured in a series of full-scale test sites were also good. A detailed explanation of the program's verification is available in reference 24.

2.3.5 Design Optimization

WESLIQID may be utilized in all of the design situations listed in the ILLI-SLAB discussion (see section 2.1.5) with the exception that only one subgrade model, the Winkler (dense liquid), is employed by WESLIQID. The additional effects of day and night time temperature gradients through the slab at various seasons of the year can also be considered.

2.3.6 Limitations

WESLIQID has limitations with respect to the types of pavement that can be analyzed and its ability to consider the factors that affect the pavement. The more significant limitations are listed below:

- Input data must be carefully chosen to ensure a converging solution.
- Ability to analyze jointed reinforced concrete pavements and continuously reinforced concrete pavements only in an indirect way.
- Limited to a maximum of two pavement layers in addition to the subgrade.
- Limited to a maximum of 9 slabs and 11 joints or cracks with 200 nodes and 130 elements.
- Temperature gradients may only be considered for slab configurations of uniform thickness.

2.3.7 Detailed Description of Capabilities

A detailed explanation of WESLIQID's ability to consider many of the factors in rigid pavement performance is presented.

PCC SLAB

Thickness

The thickness of the slab or slabs is directly input into the program. A constant thickness may be input or thickness may vary within a slab, or from slab to slab, by specifying the desired thickness at each particular node. Variation in thickness is limited by the number of nodes.

Length and Width

WESLIQID accepts all joints and cracks that run parallel to, and along the entire length, of the X and Y directions. Limits on slab size include computer memory and a 15 node maximum in the X or Y directions of any slab. WESLIQID also allows a maximum of 9 slabs separated by cracks or joints represented with up to 200 nodes and 130 elements. Optimization of computer memory using symmetry is available. These limitations and others are illustrated and discussed in the input guide.[24]

Stiffness and Strength

The modulus of elasticity and Poisson's ratio are directly input into the program and assumed constant for all slabs considered.

Fatigue and durability of the pavement section are not considered by WESLIQID.

BASE/SUBBASE

General

WESLIQID accepts a maximum of two layers in a pavement system. This allows a surface course and one base or subbase layer either perfectly bonded or unbonded to the pavement slab.

Stiffness

The modulus of elasticity and Poisson's ratio of the base or subbase is directly input into the program and is assumed constant for the entire pavement considered.

Layer Thickness

The base or subbase thickness is directly input into the program. A constant thickness may be input or the thickness may vary by inputting the desired thickness at each particular node.

Erodability/Loss of Pavement Support

WESLIQID can directly consider the effects of loss of support by inputting the nodal number at which the subgrade reactive pressure is initially assumed to be zero. The user may also specify the nodal number and amount of gap between slab and subgrade. An iterative process is then performed to determine the loss of support areas after loads are applied.

Durability and drainability of the base or subbase is not considered in WESLIQID.

SUBGRADE

General

WESLIQID models the subgrade as a dense liquid foundation by attaching four springs at the corners of each plate bending element. The reactive force between the subgrade and slab at each node equals the product of the modulus of the subgrade reaction, k, and the deflection at the node. For reactive forces at nodes along the joint that are induced by the deflections of adjacent slabs, the forces are computed through the stiffness matrix of the elements adjacent to the joint.

Stiffness

The subgrade modulus, k, may be entered as a constant or may vary by specifying the desired, k, at each particular node. Once subgrade reactive forces at each node are determined, stresses and strains in the soil may be calculated using Boussinesq's or Burmister's equations, if the user supplies values of subgrade elastic modulus and Poisson's ratio. Superposition of individual forces is employed in this computation.

Drainability and volume change potential of the subgrade is not considered in WESLIQID. Moisture sensitivity may be indirectly considered as described in Section 2.1.7.

SHOULDERS

General

Shoulders can be directly considered by placing the shoulder slab adjacent to the traffic-lane. Several load transfer systems may be modeled.

Materials

The modulus of elasticity and Poisson's ratio of the shoulder section must be assumed identical to the traffic-lane section.

Thickness

The thickness of the shoulder, surface and subbase courses may be input as constants or may vary by specifying the desired thicknesses at each particular node. The geometry of the shoulder has the same limits as the traffic-lane slabs (see discussion for P.C.C. Slab, above).

Reinforcement

Reinforcement in the shoulders may be assumed as explained in Reinforcing Steel paragraph, below.

Tying with Mainline Jointing Systems

The shoulder may be tied to the mainline with dowels and aggregate interlock or the user may specify moment or shear efficiencies across each joint in the slab configuration (see Load Transfer at Joints below).

REINFORCING STEEL

General

WESLIQID can indirectly consider continuously reinforced and jointed reinforced concrete pavements as described in ILLI-SLAB, section 2.1.7. However, extremely short spacings violate the medium-thick plate theory employed by WESLIQID. The user must have a thorough knowledge of this theory to avoid erroneous results.

LOAD TRANSFER AT JOINTS

Four options are provided for specific load transfer at the joints. Three involve shear transfer only, while the fourth involves moment transfer.

- Efficiency of shear transfer: Load transfer efficiency is specified as a ratio of vertical deflections at two adjacent nodes on either side of the joints.

- Spring constant: A spring constant, defined as a force causing unit deflection, is specified by the user. This representation takes into consideration shear at the joint.

- Diameter and spacing of dowels: This option only applies to cases where dowels are the only load transfer device. Dowel diameter, spacing, and modulus of dowel support are specified by the user. Selection of the latter is a design decision depending upon the tightness with which dowels are held in the concrete, type of dowels, strength of concrete, and method of construction.

- Efficiency of moment transfer: This is defined as a fraction of the full moment, which is determined by assuming that rotations on both sides are the same, rather than as the ratio of rotations at two adjacent nodes on either side of the joint. A moment transfer efficiency of 100 percent implies equal rotations on both sides of the joint. A zero moment transfer efficiency requires that the moment of all nodal points along the joint is zero, although rotations may not be zero. Unless the efficiency is 0.0 or 1.0 at all joints, it is necessary to analyze the problem twice. First, an efficiency of 1.0 is assumed for all joints where real efficiency is not equal to zero, to determine the full moments. These moments are multiplied by the real moment transfer efficiency and are applied as external moments during a second analysis.

JOINT DESIGN

Longitudinal and Transverse spacing of Joints

Although the slabs can be arranged in any manner, there are rules to be followed. Along a joint between two slabs, the rules are:

- The number of nodes along the joint must be equal.

- For a node on one side of the joint, there is one and only one corresponding node on the other side and the distance between the two nodes is the joint width.

Allowable slab configurations are illustrated in detail and explained in reference 24.

Sealant

Joint sealant reservoir design and sealant properties are not considered in WESLIQID.

SHRINKAGE, CURLING, and WARPING

Curling

WESLIQID is capable of considering a linear temperature gradient in uniformly thick slabs. Detailed explanation of this calculation is available in reference 24.

Shrinkage and Warping

The physical effects of material shrinkage with time and warping due to moisture distributions through the slab are not considered in WESLIQID.

DRAINAGE SYSTEM EFFECTIVENESS

WESLIQID does not directly consider any drainage effects. However, if a relationship between subgrade support modulus k and the moisture content of the subgrade can be established, the effects of drainage may be considered.

CLIMATE

WESLIQID is only capable of considering temperature gradients through the pavement section for uniformly thick slab configurations. Other climatic effects are not considered in WESLIQID.

TRAFFIC LOADINGS

WESLIQID's capabilities regarding truck volume, axle load distribution, truck lane distribution and lateral wander in lane are identical to ILLI-SLAB section 2.1.7. In addition WESLIQID can accept concentrated nodal loads and moments.

2.3.8 CALCULATIONS

Effort was spent to run the identical longitudinal edge loading mesh and material properties considered in the ILLI-SLAB analysis (see section 2.1.8). To accomplish this the original program nodal limits had to be expanded considerably. Documented instruction was followed without success.[24] It was concluded that the memory required to run this particular analysis was so large that the program has problems with internal addressing.

A curling analysis (the effect of a temperature gradient through the slab) was performed on a single slab using various sizes, thicknesses, and subgrade stiffnesses. The maximum principal tensile stresses in the slab are listed in table 8. The stresses resulting from the positive and negative temperature gradients correlated very well with the documented results in the Zero-Maintenance investigation.[46,47] For the curling analysis, the expanded nodal limit version of WESLIQID appeared to run normally in spite of the problems occurring in other portions of the program.

Table 8. WESLIQID curling analysis results.

PROGRAM: WESLIQID
RESULTS FOR CURLING ANALYSIS WITHOUT TRAFFIC LOAD
MAXIMUM PRINCIPAL SLAB STRESS

SLAB THICKNESS (INCH)	SLAB SIZE (FT)	SUBGRADE "K" VALUE (PCI) 50	50	200	200	500	500
		TEMPERATURE GRADIENT (DEGREES F/INCH) 1.5	-3.0	1.5	-3.0	1.5	-3.0
8.0	12 x 15	69.2	121.5	120.1	184.4	147.9	227.6
8.0	12 x 20	121.3	223.9	166.1	278.5	182.4	302.6
10.0	12 x 15	52.9	95.1	102.0	157.9	129.2	191.2
10.0	12 x 20	108.1	201.3	165.1	279.3	192.4	320.8
10.0	12 x 25	159.3	303.4	205.6	364.0	219.9	388.3
10.0	12 x 30	191.2	373.5	217.4	410.6	221.0	413.9
14.0	12 x 15	31.7	60.7	71.0	111.9	94.0	136.5
14.0	12 x 20	76.8	145.6	133.6	221.1	164.0	246.3

Note: A positive temperature gradient is defined as slab bottom temperature higher than slab top temperature.

Unit Weight of Concrete = 0.0868 lb/in^3
Concrete Coefficient of Expansion = 5.0x10^-6 in/in/F

1 inch = 2.54 cm
1 foot = 0.3048 m
1 psi/inch = 0.02768 kg/cm^3
°F = (°C x 1.8) + 32

2.4 WESLAYER Finite Element Model

2.4.1 Introduction

The WESLAYER finite element computer program was developed to compute the state of stress in a rigid pavement supported on an elastic solid or layered elastic foundation.[24] WESLAYER's method of solution and general input and output is very similar to WESLIQID. Therefore, reference will be made to section 2.3 when appropriate. The version of WESLAYER received for this study was a later version than the one referred to in the published documentation. Significant differences were discovered between input required by the published input guide and the updated version of WESLAYER. Considerable effort would have been required to rewrite the input guide. Therefore, it was decided that the program should be evaluated based on published documentation alone.

2.4.2 Basic Assumptions

Assumptions involved with the development of WESLAYER are identical to the WESLIQID program discussed in section 2.3.2, with the exception of the subgrade which is modeled as an elastic solid.[24/1] Because of the assumption of an elastic solid, a vertical force at one nodal point in the subgrade causes vertical movements at surrounding nodes and vice versa. In computing the subgrade reactive forces at each nodal point, Boussinesq's homogeneous half-space analysis and the layered elastic theory are used to formulate the flexibility matrix for single-layer and multilayer subgrades respectively. In the multilayer case, the layered elastic theory is not used directly at every nodal point in order to save computer time; rather, the theory is used to compute the deflections at 21 different offset points, and interpolation subroutines are used to determine the remaining deflections.

2.4.3 Capabilities

WESLAYER has identical capabilities as WESLIQID with the exception that only two slabs, uniformly thick may be modeled with one joint between them. This is due to the large computer storage capacity required by the elastic solid representation of the subgrade. Also, at the joint the program considers only the shear transfer and assumes the moment transfer to be zero.

2.4.4 Input and Output

The program input and output is identical to WESLIQID (see section 2.3.4) with the additional input of layer data, modulus of elasticity and Poisson's ratio, for the layered elastic representation of the subgrade.[24/3]

2.4.5 Design Optimization

WESLAYER is capable of assisting the design engineer to optimize materials in pavement performance and costs. A sample of design situations where WESLAYER can be utilized are listed below:

- For a given loading condition the user can vary the uniform slab thickness for a maximum allowable stress in the slab. This type of analysis will assist the user in determining required slab thickness for various subbase or base types with all other parameters remaining constant.

- Various joint design alternatives can be compared, including doweled versus undoweled joints, in combination with various joint spacings, slab thicknesses, and subbase or subgrade layer thicknesses and strengths.

- Various load transfer systems can be compared by varying dowel diameter and spacing or varying shear transfer efficiency directly.

- The effects of day and night time temperature gradients through the slab at various seasons of the year can be investigated.

- The effects of various subgrade layer thickness and stiffness can be determined.

2.4.6 Limitations

WESLAYER has limitations as to the type of pavement and the ability to consider the factors that affect a pavement. The more significant limitations are listed below:

- Input data must be carefully chosen to assure a converging solution.

- Analyzes jointed reinforced and continuously reinforced concrete pavements only in an indirect away.

- Limited to a maximum of 2 slabs, 1 joint with 70 nodes and 60 elements.

- Limited to a maximum of five layers in the subgrade.

- Temperature gradients may only be considered for uniformly thick slab configurations.

2.4.7 Detailed Description of Capabilities

A detailed explanation of WESLAYER's ability to consider many of the factors in rigid pavement performance is presented.

PCC SLAB

Thickness

The thickness of the slab or slabs is directly input into the program. A uniform thickness must be assumed for the slab or slabs considered.

Length and Width

WESLAYER accepts a maximum of two rectangular slabs. Slab size is only limited by computer memory. Optimization of the computer memory using symmetry is available.

Stiffness and Strength

The modulus of elasticity and Poisson's ratio are directly input into the program and assumed constant for all slabs considered.

Fatigue and durability of the pavement section are not considered by WESLAYER.

BASE/SUBBASE

General

WESLAYER accepts a maximum of five layers under the surface P.C.C. slab. This allows the user to model the slab foundation with various combinations of base, subbase and subgrade layers.

Stiffness

The modulus of elasticity and Poisson's ratio of the base or subbase are directly input into the program and assumed constant for the entire pavement considered. It should be noted that WESLAYER models these lower layers as continuous media under the joint in the overlying slab.

Layer Thickness

The base and or subbase thickness is directly input into the program. A constant thickness for each layer is assumed under all slabs considered.

Erodability/Loss of Support

WESLAYER considers erodability as discussed in WESLIQID (see section 2.3.7).

Durability and Drainability of the base or subbase is not considered in WESLAYER.

SUBGRADE

General

The subgrade is assumed to be a Boussinesq elastic solid for single-layer subgrade soils or a Burmister layered elastic system for multilayer subgrade soils. A maximum of five layers below the P.C.C. slab is allowed.

Stiffness

The modulus of elasticity and Poisson's ratio are directly input into the program for each of the layers. These parameters must be constant under all slabs considered. Once subgrade reactive forces are determined, stresses and strains in the subgrade soil are calculated using Boussinesq's or Burmister's equations at specified locations.[24/3]

Drainability, volume change potential, and moisture sensitivity of the subgrade soils is not considered in WESLAYER.

SHOULDERS

General

WESLAYER considers a maximum of two slabs. Therefore, a travel lane slab and a shoulder slab may be modeled with the various load transfer devices available.

Materials

The modulus of elasticity and Poisson's ratio of the shoulder slab must be assumed identical to the travel lane slab.

Thickness

The thickness of the shoulder slab, base, subbase, and other subgrade layers must be assumed identical to that of the travel lane.

Reinforcement

Reinforcement in the shoulders may be assumed as explained in section 2.3.7 for WESLIQID.

Tying with Mainline and Jointing System

The shoulder slab may be tied to the mainline with dowels or by specifying a shear transfer efficiency across the joint (see Load Transfer at Joints).

REINFORCING STEEL

General

WESLAYER considers reinforced pavements as discussed in WESLIQID (see section 2.3.7).

LOAD TRANSFER AT JOINTS

Aggregate Interlock and Mechanical Devices

Uniformly spaced dowels or specified shear efficiency are the available load transfer mechanisms in WESLAYER. The inputs and assumptions for the two options are as discussed in WESLIQID section 2.3.7.

Joint design, curling, climate, and traffic loading considerations are as discussed in WESLIQID (see section 2.3.7). Warping and drainage system effectiveness are not considered in WESLAYER.

2.5 H51ES Computerized Influence Charts

2.5.1 Introduction

The H51ES program incorporates an analytical method for calculating the bending stress at the free edge of a loaded semi-infinite slab resting on a dense liquid or elastic solid foundation.[16] This computerized procedure was first developed for the dense liquid subgrade and was later expanded to include the elastic solid idealization.[17,8] It is essentially a computerized version of the corresponding Pickett and Ray influence charts.[18]

2.5.2 Basic Assumptions

To find the maximum stress produced in a concrete slab by a loaded wheel configuration the following assumptions are made:

- The concrete slab is semi-infinite and of constant thickness.
- The loading on the slab is uniform over the contact area.
- The critical stress occurs on the underside of the slab parallel to the edge.
- The subgrade supporting the slab behaves as a dense liquid or elastic solid.
- All other assumptions involved in the development of the Pickett and Ray charts apply.[18]

2.5.3 Capabilities and Design Optimization

H51ES can be utilized to generate curves relating pavement thickness and subgrade modulus to maximum stresses at various locations along the slab edge. The program allows the user to orient the wheel configuration in any position desired with respect to the slab edge. Multiple wheel positions and slab thickness solutions can be generated by a single run of the H51ES program to produce the desired data quickly and easily. The design engineer can use this data to specify the required PCC pavement thickness based on allowable stresses and subgrade support.

2.5.4 Input and Output

The general program input includes:

- Subgrade support stiffness.
- Elastic properties (elastic modulus and Poisson's ratio) and thickness of PCC slab.
- Geometric layout and magnitude of loading.

H51ES has an optional as well as a normal method of input. The normal method of input requires the information above and generates a maximum edge stress in the pavement. The optional method requires a maximum allowable stress be input and back-calculates the required slab thickness and subgrade support. Other optional input methods involve the manner in which the applied load is oriented on the slab.

The output generated by H51ES includes:

- Listing of the input parameters to verify the input file.
- Total block count for the entire wheel configuration according to the Pickett and Ray charts. In addition, an option is available in which the block count is listed for each wheel in the configuration.
- The resulting pavement edge stress or required pavement thickness depending on method of input.

2.5.5 Limitations

The Pickett and Ray charts which are employed in H51ES were developed for determining stresses in a semi-infinite plain PCC slab. The more significant limitations to the program are listed below:

- Jointing (including slab size) and load transfer systems are not considered.
- The effects of thermal and moisture gradients are not considered.
- Base and subbases are not directly considered.
- Only edge stresses can be calculated.
- Loss of support beneath the slab is not considered.
- Wheel configurations must be symmetrical about two perpendicular axes.

2.5.6 Detailed Description of Capabilities

A detailed explanation of H51ES's ability to consider many of the factors in rigid pavement performance is presented below.

PCC SLAB

Thickness
The thickness of the slab is directly input into the program. A constant thickness must be assumed. In an optional method of input the program can solve also for the required thickness.

Length and Width
A semi-infinite slab is assumed in H51ES. Therefore the effect of slab size cannot be considered.

Stiffness and Strength

The Poisson's ratio and Modulus of Elasticity of the PCC slab are directly input into the program. Both of these parameters are assumed constant throughout the slab.

Fatigue and Durability

Fatigue properties and durability of the PCC slab are not considered in H51ES.

BASE/SUBBASE

The H51ES program does not directly consider the effects of a base or subbase layer. The increase in slab support may be indirectly approximated by employing the subgrade support value that is based upon a standard plate bearing test on the top of the base or subbase layer. For a stiff base, however, this approach is not valid. The additional effects of durability, drainability, and erodability of a base or subbase layer can not be considered by H51.

SUBGRADE

H51ES allows the user a choice between the dense liquid or the elastic solid subgrade models. Depending on this choice the necessary parameters must be input. If the dense liquid subgrade model is chosen the user must enter a constant subgrade modulus (k-value). If the elastic solid subgrade model is chosen, the subgrade Young's modulus and Poisson's ratio must be input instead and these are assumed constant throughout the subgrade. Drainability, volume change potential, and moisture sensitivity of the subgrade are not considered in H51ES.

SHOULDERS

Since H51ES considers only one slab, the effect of shoulders of any type on the traffic lane cannot be analyzed by the program.

The following factors that affect pavement performance are not considered in H51ES:

- Reinforcing steel.
- Load transfer at joints.
- Joint design.
- Shrinkage, curling, and warping.
- Drainage system effectiveness.
- Climate.

TRAFFIC LOADINGS

Axle Load Distribution

Wheel configurations must have a double center-line of symmetry, but may be oriented in any position with respect to the slab edge. The axle load is assumed to act uniformly over the elliptical shaped tire-prints. This allows the user to apply many wheel and loading conditions to determine the critical situation, that creates the maximum stress.

Truck Volume, Lane Distribution and Lateral Wander

H51ES does not consider truck volume, lane distribution and lateral wander.

2.5.7 Calculations

Stresses were calculated using the H51ES program for the transverse and longitudinal edge loading conditions. An 18-kip dual wheel axle was employed in the analysis (identical to the ILLI-SLAB analysis) with the additional parameters listed in table 9. In H51ES the area of the tire-prints are approximated by a summation of strips. The user has control over the accuracy of this approximation by inputting the number of strips to be used. This can be increased until the results satisfy a desired tolerance. A value of 200 strips (B = 200.0) was selected for the analysis. Figure 6 illustrates the layout of the two loading conditions. The calculated stresses and the corresponding locations are listed in table 10. The longitudinal edge loading condition generated the largest tensile stresses in the slab, which is consistent with the ILLI-SLAB analysis.

Table 9.

Parameters assumed in analysis one-layer pavement
H51ES program.

TYPE OF PAVEMENT	JPCP	
SLAB PROPERTIES		
SLAB THICKNESS	9.0	INCHES
POISSON'S RATIO	0.20	
MODULUS OF ELASTICITY	5000000	PSI
SUBGRADE		
SUBGRADE MODEL	WINKLER	
SUBGRADE MODULUS (K VALUE)	200	PCI
LOADING		
TYPE OF AXLE	SINGLE-AXLE,DUAL WHEEL	
GROSS WEIGHT OF AXLE	18	KIPS
TIRE PRESSURE	100	PSI

Table 10.

H51ES influence chart results.

TRANSVERSE EDGE LOADING CONDITION

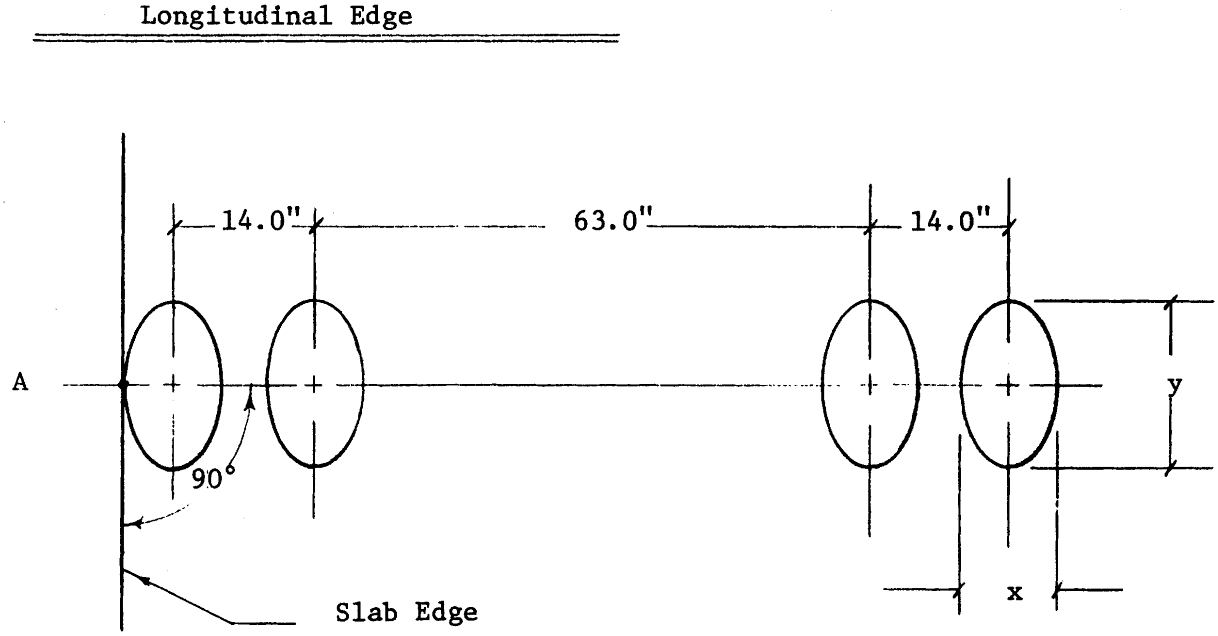
LOCATION	STRESS *
A	-27.7
B	169.8
C	158.0
D	171.8

LONGITUDINAL EDGE LOADING CONDITION

LOCATION	STRESS *
A	254.2

* = STRESS IN PSI (TENSION +) BOTTOM OF SLAB

1 in = 2.54 cm
1 psi = 0.07031 kg/cm^2
1 kip = 454 kg



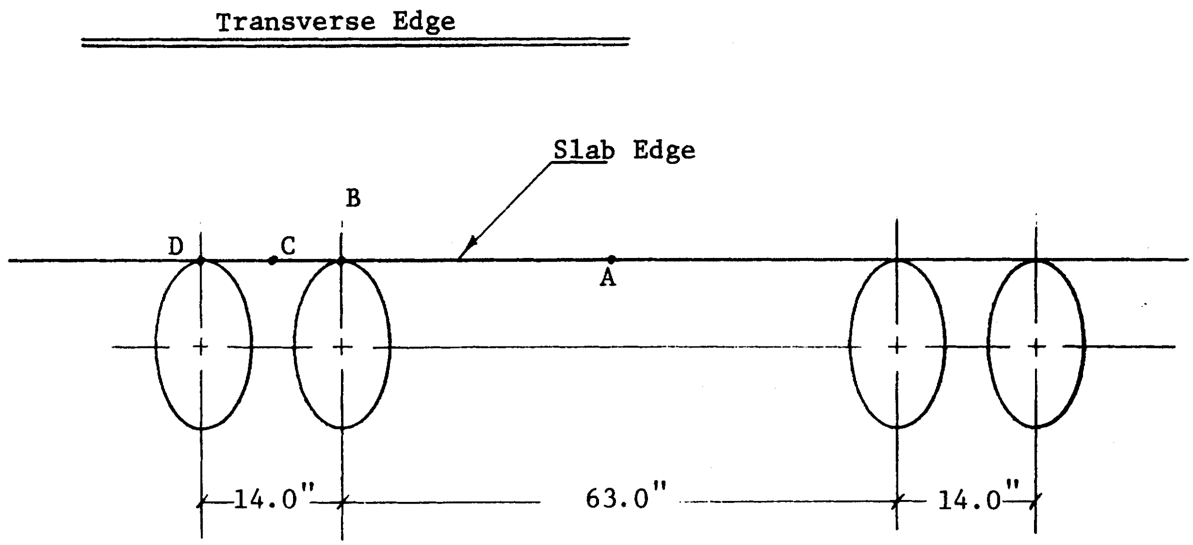


Figure 6. Layout of truck axle load at transverse joint and at longitudinal edge joint.

1 in = 2.54 cm

2.6 CRCP-2 Computer Program

2.6.1 Introduction

The computer program CRCP-2 was developed by Abou-Ayyash and McCullough at the Center for Transportation Research, University of Texas at Austin.[21] The program attempts to predict the behavior of continuously reinforced concrete pavements subjected to drying shrinkage, uniform temperature drop, and interior wheel load stress.

2.6.2 Basic Assumptions

In the development of the model the following assumptions were made:

- Concrete and steel properties are linearly elastic.
- Material properties are independent of space.
- Temperature variations and shrinkage due to drying are uniformly distributed throughout the slab and hence, a one-dimensional axial structure model is adopted for the analysis of the problem.
- The effect of creep of concrete and slab warping are neglected.
- In the fully bonded sections of the concrete slab there is no relative movement between steel and the concrete.
- The force displacement curve which characterizes the frictional resistance between the concrete slab and the underlying base is elastic.
- An average bond strength value was assumed to predict the rate of load transfer from steel bars to concrete.[19]
- A crack occurs when the concrete stress exceeds the concrete strength, and after cracking the concrete stress at the location of the crack is zero.

Other assumptions are involved in the mathematical derivation of the model and are described in detail in reference 21.

2.6.3 Capabilities

The CRCP-2 computer program can be used to determine the combined effect of external loads and internal loads on a continuously reinforced concrete pavement. In 1977 the program was expanded to include extreme values of variable combinations such as:

- The analysis of a continuously reinforced concrete pavement placed over a treated base with high frictional resistance.
- The analysis of a high percentage of steel reinforcement.
- The analysis of high temperature drops.

2.6.4 Input and Output

The program input includes:

- Steel properties.
- Concrete properties including the age-tensile strength relationship.
- Environmental conditions during and after curing.
- External load or calculated stress due to load and when it is applied.
- Slab-base friction relationship.

The output produced by CRCP-2 includes:

- The final crack spacing and width.
- The maximum concrete and steel stresses.
- The changes in steel and concrete stresses, friction forces plotted along the horizontal stations of the slab. Station 1 is at midslab and station 101 is at the crack.
- Variations of concrete strength, concrete stress, steel stress, drying shrinkage, crack width and the changes of crack spacing with time are listed.

The derivation of the mathematical models used by CRCP-2 uses the principles of mechanics of materials and the structural equilibrium of the system to predict the behavior of the pavement.

2.6.5 Design Optimization

CRCP-2 is capable of analyzing various materials and environmental conditions to predict their effects on continuously reinforced concrete pavements. Comparisons that may be analyzed include:

- High strength versus low strength steel.
- Deformed bars versus deformed wire fabric.
- Various curing temperature and minimum temperature combinations.
- Various reinforcement ratios.
- Various age and tensile strength of concrete relationships.
- Various slab-base friction relationships.
- Combinations of the above and various wheel load stresses.

2.6.6 Limitations

The significant limitations to CRCP-2 are listed below:

- Fatigue due to repetition of load is not considered.
- Curling stress due to temperature differential is not considered.
- Only interior single wheel loading conditions can be analyzed (the Westergaard interior loading condition equation is employed to calculate the concrete stress due to wheel load which assumes uncracked slabs). For other loading conditions, the user must input the concrete stress due to the wheel load. Typical edge punchouts are caused by an entirely different stress at the top of the slab, transverse to the longitudinal joint when the wheel load is at the outer edge of the slab.
- Only continuously reinforced concrete pavements can be analyzed.
- Slab longitudinal jointing systems are not considered.
- Shoulder type and relationship with the mainline are not considered.

2.6.7 Detailed Description of Capabilities

A detailed explanation of CRCP-2's ability to consider the factors that affect continuously reinforced concrete pavements is presented.

PCC SLAB

Thickness

A constant thickness must be assumed throughout the slab and is directly input into the program.

Length and Width

The development of CRCP-2 involves a one-dimensional analysis that is derived using a unit width, i.e., width is not involved in the computation. The length of the slabs is determined in the analysis of the program.

Stiffness and Strength

CRCP-2 allows the user to specify an age-strength relationship in the concrete by inputting the concrete tensile strength and the respective age in days to generate a curve. A maximum of 20 points to describe the curve is permitted. The program then analyzes the pavement considering the elapsed time before loads are applied to obtain a more accurate prediction of the concrete strength and how the pavement will behave. The program also contains a default age-strength relationship in which the 28 day compressive strength is required in the input and the age-strength relationship is generated by CRCP-2 which is based on the age-strength curve from reference 20. This type of input allows the user to consider the concrete strength increase beyond the 28 day strength.

Fatigue and Durability

Fatigue and durability of the PCC slab is not considered in CRCP-4.

BASE, SUBBASE AND SUBGRADE

The support material is only considered in its friction relationship with the slab. This relationship may be created by inputting one point assuming either a straight line or parabolic relationship between friction force and the magnitude of the movement of the slab. The user can also define the curve by inputting the movement of the slab and its corresponding friction force per unit length at several points.[21] A maximum of 12 points is permitted.

CRCP-2 requires the modulus of subgrade reaction be input if the user chooses to have the program calculate the PCC slab tensile stress due to an external load. The program utilizes the Westergaard equation for interior loading condition for this task.

The following characteristics of base, subbase, and subgrade that affect rigid pavement performance are not considered in CRCP-4, and will not be discussed further:

- Durability.
- Erodability, loss of pavement support.
- Drainability.
- Layer thickness of base or subbase.
- Moisture sensitivity of subgrade.
- Volume change potential of subgrade.

SHOULDERS

The relationship between the shoulder and the mainline pavement and their jointing system is not considered in CRCP-4. However, the shoulder can be analyzed independently if the shoulder is continuously reinforced concrete.

REINFORCING STEEL

CRCP-2 is capable of analyzing concrete pavements with two types of continuous reinforcing, deformed bars and deformed wire fabric. The following input is required to describe the reinforcing:

- Percent steel reinforcing.
- Yield stress of steel.
- Elastic modulus of steel.
- Thermal coefficient of steel.
- Reinforcing bar diameter (if bars are used).
- Transverse wire spacing (if deformed wire fabric is used).

JOINT DESIGN AND LOAD TRANSFER

The program is limited to a one-dimensional analysis of continuously reinforced concrete pavements. Joint design and load transfer across joints is not considered.

SHRINKAGE, CURLING AND WARPING

The dimensional changes in a continuously reinforced pavement caused by drying shrinkage of the concrete and temperature variation after curing were investigated and the CRCP-2 design method was developed in the study described in reference 19. The theoretical model was based on the material properties; stress; strain interaction between steel, concrete, subgrade; and the internal forces caused by the temperature drop and shrinkage of the slab. The model determines the crack spacing by comparing concrete stress with concrete strength at each time interval to obtain a final crack spacing. The additional stresses caused by external loads can also be included in this analysis. However, the effects of curling stresses caused by the temperature differential across the depth of the slab are not considered.

DRAINAGE SYSTEM EFFECTIVENESS

The effect of drainage on rigid pavements is not considered by CRCP-4.

CLIMATE

The program considers the effects of a uniform temperature decrease or increase after curing on continuously reinforced concrete pavements (see Shrinking, Curling and Warping). The additional effects of corrosion, moisture, and freeze-thaw susceptibility are not considered by CRCP-4.

TRAFFIC LOADINGS

The effects of external wheel loadings on continuously reinforced concrete pavements can be considered by CRCP-4. The program is capable of calculating the PCC slab tension stresses for the interior single wheel loading condition only, using the Westergaard equation. The user also has the option to prescribe the maximum tension stress at the slab interior. The calculation for this stress is performed by the user. Due to the mathematical derivation of the CRCP-2 model the corner loading condition cannot be considered.[19]

The volume of trucks or lateral wander in the traffic lane cannot be considered by CRCP-4.

2.6.8 Calculations

Typical material properties were chosen for input into CRCP-2 as shown in table 11. Reinforcement ratios of 0.59 and 0.70 percent with various pavement thicknesses were analyzed. Environmental conditions included, constant and incrementally decreasing minimum daily air temperature until the concrete gained full strength.

The results listed in table 12 showed calculated parameters (crack spacing, maximum steel stress, etc.) unchanged for all slab thicknesses considered when post curing temperatures were held constant. In general the results were most sensitive to the assumed post curing temperatures. Also, when the reinforcement ratio was increased the crack spacing shortened. The user who recognizes the assumptions of the program could use CRCP-2 to investigate the cracking behavior of continuously reinforced concrete pavements by experimenting with many other parameters.

Table 11.

Parameters assumed in analysis for
continuously reinforced concrete pavements
CRCP-2 model.

STEEL PROPERTIES		
TYPE OF REINFORCEMENT	DEFORMED BARS	
PERCENT STEEL REINFORCEMENT	0.59, 0.70	PERCENT
REINFORCING BAR DIAMETER	0.625, 0.750	INCHES
YIELD STRESS	60000	PSI
ELASTIC MODULUS	29000000	PSI
THERMAL COEFFICIENT OF EXPANSION	0.0000065	IN/IN/°F
CONCRETE PROPERTIES		
SLAB THICKNESS	7, 8, 9	IN
THERMAL COEFFICIENT OF EXPANSION	0.0000055	IN/IN/°F
DRYING SHRINKAGE STRAIN	0.0002	IN/IN
UNIT WEIGHT OF CONCRETE	150	LB/FT^3
28 DAY COMPRESSIVE STRENGTH	4000	PSI
TENSILE STRENGTH	RELATIONSHIP ASSUMED BY BUREAU OF RECLAMATION	
SLAB BASE FRICTION CHARACTERISTICS		
TYPE OF FRICTION CURVE	STRAIGHT LINE	
FRICTION FORCE PER UNIT LENGTH	1.0	PSI
SLAB MOVEMENT	-0.1	IN
ENVIRONMENTAL INPUT		
CURING TEMPERATURE	75	°F
NO. OF DAYS BEFORE CONC. GAINS FULL STRENGTH	28	DAYS
MIN. TEMP. AFTER CONC. GAINS FULL STRENGTH	70, 0	°F
NO. OF DAYS BEFORE MIN. TEMP. OCCURS	90	DAYS

1 in = 2.54 cm
1 psi = 0.07031 kg/cm^2
$^oF = (^oc \times 1.8) + 32$

Table 11 (cont)

Parameters assumed in analysis for
continuously reinforced concrete pavements
CRCP-2 model.

MINIMUM DAILY TEMPERATURE

WHEN CONSTANT

DAY 1-28 MIN. TEMP. - 70 (°F)

WHEN VARIES

DAY #	MIN. TEMP. (°F)	DAY #	MIN. TEMP. (°F)
1	70.0	15	35.0
2	68.5	16	32.5
3	65.0	17	30.0
4	62.5	18	27.5
5	60.0	19	25.0
6	57.5	20	22.5
7	55.0	21	20.0
8	52.5	22	17.5
9	50.0	23	15.0
10	47.5	24	12.5
11	45.0	25	10.0
12	42.5	26	7.5
13	40.0	27	5.0
14	37.5	28	2.5

Table 12. CRCP-2 continuously reinforced concrete pavement analysis results.

PROGRAM: CRCP-2
ANALYSIS OF CONTINUOUSLY REINFORCED PAVEMENTS
VARIOUS SLAB THICKNESSES WITHOUT WHEEL LOADS

SLAB THICKNESS [INCHES]	REINFORCE. RATIO [%]	BAR SIZE [INCHES]	CONSTANT TEMP.	CONC. COMP. STRENGTH [PSI]	CRACK SPACING [FEET]	CRACK WIDTH [INCHES]	MAX. CONC. STRESS [PSI]	MAX. STEEL STRESS [PSI]	CONC. TENS. STRENGTH [PSI]
7.0	0.59	0.625	YES	4000	33.90	0.0445	472	77400	474
7.0	0.59	0.625	NO	4000	8.20	0.0501	499	84400	497
8.0	0.59	0.625	NO	3000	6.15	0.0378	432	73100	431
8.0	0.59	0.625	YES	4000	33.90	0.0445	472	77400	474
8.0	0.59	0.625	NO	4000	8.20	0.0501	499	84400	497
8.0	0.59	0.625	NO	5000	10.20	0.0622	588	94300	556
9.0	0.59	0.625	YES	4000	33.90	0.0445	472	72400	474
9.0	0.59	0.625	NO	5000	8.20	0.0501	499	84400	497
7.0	0.70	0.750	NO	4000	6.95	0.0428	498	71000	497
8.0	0.70	0.750	NO	4000	6.95	0.0428	498	71000	497
9.0	0.70	0.750	NO	4000	6.95	0.0428	497	71000	497
3.0	0.59	0.625	YES	4000	33.90	0.0443	475	77200	474
8.0	0.59	0.625	* YES	4000	7.54	0.0461	478	80900	474

* FINAL ENVIRONMENT TEMPERATURE SET TO 0 DEGREES FAHRENHEIT

1 inch = 2.54 cm
1 foot = 0.3048 mm
1 psi = 0.07031 kg/cm^2

2.7 Jointed Concrete Shoulder Design Procedure Program (JCS-1)

2.7.1 Introduction

The computer program JCS-1 was developed in 1978 for the FHWA. This program is a part of a comprehensive design procedure based on fatigue for plain jointed concrete shoulders with an objective of controlling shoulder slab cracking.[13] JCS-1 was developed to compute the accumulated fatigue damage over the design life of the PCC shoulder. Field and lab data and a finite element analysis were correlated to develop the necessary relationships between fatigue damage and slab cracking that are employed by JCS-1.[13]

The traffic lane pavement design, whether jointed plain, jointed reinforced, or continuously reinforced, is only considered through the load transfer of the longitudinal joint.

2.7.2 Basic Assumptions

There are many assumptions in the development of JCS-1 of which the user must be aware to ensure valid results for a particular application.[13] The finite element method utilized by JCS-1 was originally developed by Huang and Wang [14] at the department of Civil Engineering University of Kentucky. This method was then modified slightly at the University of Illinois. The basic assumptions in this finite element method include:

- Classical theory of medium-thick plates.
- A plane section before bending remains plane after bending.
- The slabs are homogeneous, isotropic, and elastic.
- The subgrade acts as a Winkler foundation, i.e., the reactive pressure between subgrade and slab at any given point is proportional to the deflection at that point.

The finite element procedure follows essentially that of reference 12.

The comprehensive PCC shoulder fatigue damage analysis was developed based on the following:

- The two critical fatigue damage locations in the shoulder are both at the shoulder slab longitudinal edge midway between the transverse joints, either: (1) the lane/shoulder joint or (2) the outer shoulder edge (these locations were determined using both field and slab fatigue analysis results).[13]
- Critical edge stresses caused by traffic loads are considered to prevent transverse cracking.

- Load stresses were computed using the finite element program over a range of design variables. Stress prediction models were derived using multiple stepwise regression techniques and individual equations were derived for traffic load stress at each of the shoulder edges due to loading condition at the edge under consideration.[13] A fatigue curve relating the ratio of repeated flexural stress to modulus of rupture and the number of stress applications to failure was utilized and represents a reliability of 76 percent. This relationship was generated as part of a research project, "Design of Zero-Maintenance Pavements" [15], that was conducted at the University of Illinois.

- The proportion of mainline traffic encroaching on the shoulder inner edge and parking on the shoulder outer edge are used in the fatigue analysis.

- Fatigue damage is computed according to Miner's hypothesis.

- A correlation between computed fatigue damage and measured field cracking was determined. A limiting damage for PCC shoulder design is selected to control cracking.

Each of these assumptions and their basis is explained by Sawan et al.[13]

2.7.3 Capabilities

The designer must specify trial structural designs, determine the required inputs, run the JCS-1 computer program, and analyze the output fatigue data at the two PCC shoulder critical locations. The program is written to analyze any one or a combination of facts, including shoulder thickness, mainline thickness, shoulder width and load transfer efficiency across the longitudinal joint; and to provide output for each combination, while holding all other inputs constant. The designer can, therefore, examine a range of combinations of the above four factors for a given traffic and foundation support, with only one run of the program.

2.7.4 Input and Output

The program input includes:

- Traffic lane slab thickness and load transfer across longitudinal shoulder joint.

- Concrete strength properties.

- Shoulder design width and thickness.

- Traffic data at beginning and end of design period.

- Foundation support data (k-value).

The output produced by JCS-1 includes:

- The fatigue damage accumulated during each year of the shoulder design life for two different locations in the shoulder slab: at the outer edge due to parked traffic and at the inner edge due to encroached traffic on the shoulder.

- The total fatigue damage during the entire design period due to parked traffic and due to encroached traffic.

Complete verification of the design procedure requires construction of the recommended designs in various climatic regions and observation of their performance over the structural design life. In lieu of this costly and time consuming procedure, a reasonable verification was obtained by comparing the design of two experimental plain jointed concrete shoulder projects. These limited results indicated that the new design procedure produces designs that are compatible with the existing design practices.[13]

2.7.5 Design Optimization

JCS-1 is capable of assisting the design engineer to optimize materials in PCC shoulder performance and costs considering both type and quantity of traffic. Shoulder design situations where JCS-1 can be employed are listed below:

- Determine required shoulder thickness given traffic data, longitudinal load transfer efficiency, subgrade support, and PCC slab properties.

- Determine the effects of higher strength versus normal strength PCC.

- Trade-offs between foundation support and slab thickness.

- Trade-offs between longitudinal load transfer efficiency and PCC shoulder slab thickness.

2.7.6 Limitations

JCS-1 was strictly developed to design PCC shoulders. Therefore, the traffic lane itself is only considered in its effect on the shoulder. The significant limitations to JCS-1 are listed below:

- The traffic lane design is not considered, except for thickness and load transfer.

- No climatic considerations exist.

- No consideration exists for the effects of drainability.

- No reinforcement considerations exist.

- Design is based upon the fatigue analysis of only two locations on the shoulder.

- The design procedure has not been thoroughly tested.

- The program was developed based on a finite element analysis over a limited range of variables (subgrade modulus, strength of concrete, Poisson's ratio, slab thickness).

2.7.7 Detailed Description of Capabilities

A detailed explanation of JCS-1's ability to consider the factors in rigid pavement performance is presented.

PCC SLAB (Traffic Lane)

Thickness
The thickness of the traffic lane is directly input into the program. A constant thickness throughout the slab is assumed.

Length and Width
The dimensions of the slab were assumed to be 15 ft in length and 12 ft in width. The fatigue analysis in JCS-1 assumes the critical fatigue locations to exist at the longitudinal inner and outer edges of the shoulder at midpoint between transverse joints. This assumption for practical applications is independent of length and width. Therefore, the application of JCS-1 to pavements with different length and width should be reasonable.

Stiffness and Strength
The material used in and under the PCC traffic lane slab must be identical to the shoulder materials.

Fatigue Properties
Fatigue properties of the PCC traffic lane are not considered in JCS-1.

Durability
Durability of the PCC traffic or shoulder lane is not considered in JCS-1.

BASE/SUBBASE

The effects of base or subbase is not directly considered in JCS-1. However, it is suggested if a base or subbase is used, the subgrade modulus measured at the top of the base or subbase be used in the analysis.[13] The base or subbase layer thickness and its stiffness is represented by the magnitude of the increase in the adjusted modulus of subgrade support. However, a very stiff subbase should not be assigned a k-value greater than 500 pci (13.8 kg/cm^3).

Erodability/loss of pavement support
The amount of erosion of the base, subbase or subgrade at any time is expressed as the width in inches of a rectangular strip parallel to the PCC shoulder inner edge (longitudinal joint) that has no contact with the pavement slab when loaded or

unloaded. The strip width specified refers to the loss of support at the end of the design period and has a maximum width of 12 in. The erodability at the beginning of the design period is assumed to be zero in the program. The amount of erodability at any time after the pavement is opened to traffic is linearly interpolated between initial and final erodability factors.

Durability

Durability of the base or subbase is not directly considered in JCS-1. However, the concept is taken into account in the erodability analysis.

Drainability

Drainability of the base or subbase is not considered in JCS-1.

SUBGRADE

Stiffness

A constant subgrade modulus of reaction, k, is directly entered into the program. This modulus is obtained by the 30 in (76.2 cm) plate bearing test is described by ASTM, or is estimated from soil properties.

Drainability, Moisture Sensitivity, and Volume Change Potential of the subgrade are not considered in JCS-1.

SHOULDERS

Materials

The mean modulus of rupture at 28 days as determined by the test procedure specified in AASTHO designation T-97, using third point loading, is the basis for determining concrete flexural strength. The 28 day modulus of rupture is adjusted for concrete variability to represent a confidence level of 85 percent after the coefficient of variation is input. The finite element program used to develop JCS-1 assumed a constant modulus of elasticity of 6.25E06 psi (4.39 x10^5 kg/cm^2) and a Poisson's ratio of 0.28 for the Portland cement concrete.

Thickness and Geometry

The thickness of the PCC shoulder slab is directly input into JCS-1 and may differ from the traffic lane. Shoulder width in feet is directly input into the program. JCS-1 was developed for a shoulder width range of 3 to 10 ft (0.9 to 3.05 m). It is recommended that this range be adhered to as deviations may lead to erroneous results. Tapered shoulders showed no advantages in the development of this program, therefore they are not included in JCS-1. A length of 15 ft (4.6 m) between transverse joints for both traffic lane and shoulder were used in the analysis.

Reinforcement

The effects of reinforcement in the shoulder is neglected in JCS-1.

Tying with Mainline and Jointing System

The effect of tying the shoulder to the mainline is represented by the specified deflection load transfer efficiency. The type of jointing system, aggregate interlock or dowels, is disregarded and only the resulting deflection load transfer efficiency is considered.

REINFORCING STEEL

The program JCS-1 was developed for plain jointed concrete shoulders, therefore the effects of reinforcing steel are not considered.

LOAD TRANSFER AT JOINTS

The fatigue analysis in JCS-1 assumes the longitudinal edges midway between the transverse joints of the shoulder slab to be the critical fatigue locations. Therefore, the transverse joints and their load transfer efficiencies are not considered. However, the longitudinal joint deflection load transfer efficiency between the traffic lane slab and shoulder slab is directly entered into the program.

JOINT DESIGN

Longitudinal and Transverse Spacing of Joints

JCS-1 only considers one joint in its analysis, the longitudinal joint between the traffic lane slab and the shoulder slab.

Sealant

Joint sealant reservoir design and sealant properties are not considered in JCS-1.

SHRINKAGE CURLING and WARPING

These physical effects and their associated stresses are not considered in JCS-1. Thus, shoulder joint spacing greater than 15 ft is not recommended.

DRAINAGE SYSTEM EFFECTIVENESS

JCS-1 does not directly consider any drainage effects. However, if a relationship between subgrade support modulus (k) and the moisture content of the subgrade can be established, the effects of drainage may be considered.

CLIMATE

JCS-1 does not consider any climatic effects.

TRAFFIC LOADINGS

A detailed input of traffic data is required in JCS-1 as follows:

- PCC shoulder design life.
- Average daily traffic at beginning and end of design period, both directions.
- Percent trucks in average daily traffic.
- Percent trucks in heaviest traveled or design lane.
- Percent truck directional distribution.
- Mean axles per truck.
- Length of surveyed shoulder stretch in miles.
- Average length of total encroachments per truck in the surveyed shoulder stretch in miles.
- Percent of trucks that park on the surveyed shoulder stretch relative to the design lane truck traffic.
- Number of single axle load distribution groups.
- The percent axle loads in each of the single axle load groups.
- Number of tandem axle load distribution groups.
- The percent axle loads in each of the tandem axle load groups.

With this information JCS-1 calculates the actual quantity and magnitude of loadings that effect the PCC shoulder slab for a given design life. However, the user must have a thorough knowledge of the definitions and assumptions involved with the traffic data to obtain reliable results.[13]

Truck Volume

A linear relationship is assumed between the initial and final ADT to obtain the ADT at intermediate years. The appropriate distribution percentages are then employed to obtain the truck volume in each load group that effects the PCC shoulder slab.

Axle Load Distribution

The average percent of total load applications occurring within a specified load group (usually 2000 pound range) must be estimated for the entire design analysis period and directly input into JCS-1. A maximum of 40 axle load distribution groups, single plus tandem, is allowed.

Truck Lane Distribution

The lane distribution of trucks varies with many factors including; number of lanes, urban/rural location, traffic volume, and percent trucks. In the development of JCS-1 it is suggested this parameter can be best estimated through manual vehicle counts on the existing or similar highways in the area. This estimated lane distribution value is directly input into JCS-1.

Truck Lateral Wander in Lane

JCS-1 considers three types of traffic that use PCC shoulders: encroached traffic, parked traffic, and normal lane traffic. Encroached traffic is the part of the mainline traffic that encroaches on the shoulder occasionally and then merges back to the mainline. Parked traffic is the part of the mainline traffic that parks on the shoulder for emergency reasons or otherwise. If it is anticipated that the PCC shoulder would be used by regular traffic at any stage of its design life, then this extra amount of traffic should be counted for as a part of the shoulder design traffic. A detailed explanation of JCS-1's ability to consider these various types of traffic was documented in reference 13.

2.7.8 Calculations

A sample run of JCS-1 was performed. Shoulder design life, slab properties and traffic data were assumed and are listed in table 13. Shoulder thickness of 5, 6, 7, 8, and 9 inches were analyzed to determine a range of predicted fatigue damage for the given conditions. JCS-1 allows this procedure to be executed in a single program run. The results are as follows:

PCC Shoulder Thickness	Total Fatigue Damage for Design Period Parked Traffic	Encroached Traffic
5 in (13 cm)	0.418 E25	0.353 E04
6 in (15 cm)	0.574 E12	0.695 E00
7 in (18 cm)	0.334 E05	0.652 E-02
8 in (20 cm)	0.106 E01	0.316 E-03
9 in (27 cm)	0.104 E-02	0.351 E-04

(1 in = 2.54 cm)

These fatigue damage values were calculated using Miner's Hypothesis, according to which a material should fracture when the accumulated damage equals 1.0. However, even if Miner's Hypothesis were exact, variability of material strength, loads, and other properties would cause a variation in accumulated damage from slab to slab ranging from much less than one to much greater. In the development of JCS-1 a curve was generated relating Miner's fatigue damage to a cracking index.[13] The designer, with the use of this curve, can select a limiting design fatigue damage value to limit the cracking of the shoulder slabs. Once the fatigue damage value is computed for a given design, the cracking index over the design period can be estimated.

Table 13.

Parameters assumed in analysis for JCS-1 program.

SHOULDER DESIGN LIFE	20.0	YEARS
SLAB PROPERTIES		
SHOULDER THICKNESS	5,6,7,8,9	IN
TRAFFIC LANE THICKNESS	8.0	IN
SHOULDER WIDTH	10.0	FT
MEAN PCC MODULUS OF RUPTURE (28 DAYS)	750	PSI
COEFFICIENT OF VARIATION OF PCC MODULUS	10	%
LOAD TRANSFER EFFICIENCY BETWEEN SHOULDER AND TRAFFIC LANE	50	%
FOUNDATION SUPPORT		
DESIGN MODULUS OF FOUNDATION SUPPORT (K)	200	PCI
ERODABILITY OF FOUNDATION SUPPORT AT END OF DESIGN PERIOD	8.0	IN
TRAFFIC		
ADT AT BEGINNING OF DESIGN PERIOD	17100	EACH
ADT AT END OF DESIGN PERIOD	39100	EACH
PERCENT TRUCKS OF ADT	21	%
PERCENT TRUCKS IN DESIGN TRAVELED LANE	85.15	%
PERCENT DIRECTIONAL DISTRIBUTION	50	%
MEAN AXLES PER TRUCK	2.60	EACH
LENGTH OF SURVEYED STRETCH	10.0	MILES
AVERAGE LENGTH OF TOTAL ENCROACHMENTS PER TRUCK IN THE SHOULDER STRETCH	0.24	MILES
PERCENT TRUCKS THAT PARK ON THE SHOULDER	0.016	%
NUMBER OF SINGLE-AXLE LOAD INTERVALS	13	EACH
NUMBER OF TANDEM-AXLE LOAD INTERVALS	17	EACH

SINGLE-AXLE LOAD DISTRIBUTION TABLE

WEIGHT RANGE (POUNDS)	PERCENT IN RANGE
0 - 3000	5.75
3001 - 7000	10.33
7001 - 8000	7.76
8001 - 12000	20.54
12001 - 16000	4.37
16001 - 18000	1.77
18001 - 20000	1.02
20001 - 22000	0.54
22001 - 24000	0.34
24001 - 26000	0.14
26001 - 30000	0.04
30001 - 32000	0.01
32001 - 34000	0.01

(1 in = 2.54 cm, 1 ft = .3048 m, 1 mile = 1.609 km)
(1 lb = .454 kg)

Table 13 (cont).

Parameters assumed in analysis for JCS-1 program.

TANDEM AXLE LOAD DISTRIBUTION TABLE

WEIGHT RANGE (POUNDS)	PERCENT IN RANGE
0 - 6000	0.27
6001 - 12000	13.34
12001 - 18000	7.05
18001 - 24000	5.51
24001 - 30000	14.92
30001 - 32000	3.61
32001 - 34000	1.40
34001 - 36000	0.50
36001 - 38000	0.25
38001 - 40000	0.16
40001 - 42000	0.11
42001 - 44000	0.08
44001 - 46000	0.07
46001 - 50000	0.07
50001 - 52000	0.02
52001 - 54000	0.01
54001 - 56000	0.01

However, the designer must consider the assumptions involved with the development of this curve to obtain successful results.

JCS-1 is an easy to use tool that is tailored to the design of jointed plain concrete shoulders. The computer processing time for a design problem is about 9 seconds for analyzing a range of shoulder thicknesses. The storage requirement for the program is 40,000 bytes on a Cyber 175 mainframe computer. It could easily be converted to a microcomputer.

2.8 RISC Finite Element Model

2.8.1 Introduction

The September 1982 version of RISC is part of a mechanistic design procedure for rigid pavements and is based on the coupling of a finite element slab resting on a multilayer elastic solid foundation of up to three discrete layers. The program considers up to three slabs in a row with or without shoulders and such parameters as joint spacing, joint width, the effect of dowel bars and tie bars, load location, and thermal curling stresses.[25] However, the program requires a large amount of high-speed computer time (30-50 minutes of computer CPU time for a single run). Therefore, the program is quite expensive to use for certain types of investigations.

2.8.2 Basic Assumptions

The assumptions regarding the modeling of materials in the pavement section are briefly summarized as follows:

- The finite element method employed to represent the concrete slab is based on the theory of a flat thin elastic shell, in which the median plane is assumed to completely represent the shell element. Also all assumptions for a thin Kirchhoff plate apply.

- The slab foundation is represented as a Boussinesq elastic solid for one-layer foundations and a Burmister multilayered elastic solid for two and three layered foundations.

- The pavement materials are modeled as linearly elastic.

- The environmental effects are incorporated into the design through the AASHTO regional factor which is used to modify the traffic.

- Fatigue calculations are based on the RII distress function for a Terminal Serviceability Index of 2.0.[26]

- Faulting predictions are based on the PCA model for plain jointed pavements and the Darter model for doweled pavements.[27,28]

- Dowel concrete interaction is calculated by the Friberg analysis with the modulus of dowel support, K, assumed as 1,500,000 pci (41520 kg/cm^3).[29]

2.8.3 Capabilities

RISC is capable of analyzing a variety of rigid pavement cross sections as described:

- One, two, or three pavement slabs in a row, with or without concrete shoulders.
- Various base, subbase and subgrade, stiffness and uniform thickness combinations. Edge and/or corners may be specified as in contact or out of contact with the above slab.
- Concrete slabs of various uniform thickness and stiffness.
- A linear vertical temperature gradient in the slabs may be specified to compute thermal stresses.
- Traffic loadings may be considered by specifying the magnitude of the standard dual-wheel truck axle for edge, corner or midslab locations.
- Stresses may be computed as a result of traffic loading alone, temperature alone, or a combination of both.
- Transverse joints may be doweled or undoweled.

2.8.4 Input and Output

The program input includes:

- Geometry of the slabs and shoulders.
- Longitudinal and transverse joint data including dowel and tie information.
- Elastic properties and uniform thickness of slab, shoulder and foundation layers.
- Wheel load magnitude (total load) at edge, corner or midslab locations.
- Vertical temperature gradient through slabs if considered.
- Daily traffic number as described in reference 25.
- Contact condition of edges and corners of slabs and shoulders.
- Subgrade drainage condition and subbase type.

The output produced by RISC includes:

- Summary of input data.
- Maximum displacement of slab and its location.
- Critical stress in slab and its location.
- Predicted fatigue life both in 18-kip equivalent single axle-loads and in years.
- Predicted faulting during life.
- Bending and bearing stresses in dowel bars.
- Maximum pressure on support layer directly beneath slab.
- Minimum transverse joint load transfer efficiency (defined as the ratio of deflections of the unloaded to the loaded slab) and location.

The RISC structural analysis model was verified by comparison with multilayer theory, Pickett and Ray theory, and field performance. A detailed explanation of the verification of RISC is available in reference 25.

2.8.5 Design Optimization

RISC is capable of assisting the design engineer to optimize pavement performance and costs. A small sample of design situations where RISC can be used as a tool in decision making are listed below:

- For a given loading condition the user can investigate the effects of various uniform slab thicknesses.
- Various doweled transverse joint and longitudinal tied joint designs between traffic lane and shoulder can be compared in combination with varying joint spacings, slab thickness, and foundation characteristics.
- The effects of day and night time temperature differentials through the slab at various seasons of the year can be investigated.
- The effects of corner and/or edge voids beneath the traffic lane and/or shoulder slabs for edge, corner, and interior loading conditions may be analyzed.

2.8.6 Limitations

The significant limitations of RISC's structural model in considering types of pavement and factors that effect pavement performance are listed below:

- RISC analyzes jointed reinforced and continuously reinforced concrete pavements only in an indirect way.
- RISC is limited to a maximum of three slabs in a row with or without shoulders.
- RISC is limited to a standard dual wheel loading at a choice of three predetermined locations.[25]
- All transverse joints must have identical load transfer mechanisms.
- The traffic lane, shoulder longitudinal joint must have identical characteristics for all slabs considered.
- Load transfer across longitudinal shoulder joint is not calculated.
- RISC is limited to predetermined void (loss of support) locations and dimensions.
- Unit weight and coefficient of thermal expansion of concrete slab are assumed in program and are not direct inputs.
- Bonding condition between layers is assumed in the program and is not a direct input.
- Flexural stresses in base and/or other support layers are not calculated.
- Subgrade stress is not calculated when more than one-layered pavements are modeled.
- Critical tension stress location in slab (top or bottom of slab) is not indicated.
- Only maximum displacements and stresses are output.
- Extremely large computer run times are required.
- Fatigue and faulting models in RISC are questionable.

2.8.7 Detailed Description of Capabilities

A detailed explanation of RISC's ability to consider many of the factors in rigid pavement performance is presented.

PCC SLAB

Thickness

The thickness of the slab or slabs is directly input into the program. A uniform thickness must be assumed, however, the thickness may vary from slab to slab.

Length and Width

RISC allows up to three slabs in only one direction. Slab length has no limit and may vary from slab to slab. Slab width must be the same for all pavement slabs but may differ from shoulder slab width.

Stiffness and Strength

The Poisson's ratio, tangent modulus of elasticity, and 28-day flexural strength of the concrete must be directly input into the program. These parameters are assumed constant for all slabs and shoulders considered.

Fatigue

The RII fatigue model developed is employed in RISC.[26] This prediction model uses the AASHTO road test data and was based on plate theory resting on a multilayered elastic solid subgrade. The model incorporates such effects as actual load placement, slab geometry, effects of load transfer devices, variation of material properties by lane location. The resulting distress equation is:

$$Nf = 22209.0 * (Mr/S)^{4.29} \qquad (1)$$

Where: Nf = The number of equivalent 18-kip single axle loadings required to produce a Terminal Serviceability Index of 2.0.
S = Load induced stress in the PCC slab.
Mr = Modulus of rupture of the PCC.

The RISC program uses this equation to predict the fatigue life of the pavement in 18 kip equivalent axle loads and in years. Since there was a huge amount of pumping and subsequent cracking of the AASHTO thinner slabs (< 8 in), the validity of this model is questionable for thicker slabs.

Faulting

Faulting predictions are based on models for plain undoweled and doweled concrete pavements respectively.[7,28] Packard used multiple regression analyses on data from 404 sections of highways located in five States. Parameters considered in the Packard faulting prediction model include:

- Age of pavement.
- Pavement thickness.
- Joint spacing.
- Average daily truck traffic.
- Subbase and subgrade type.

A correlation coefficient of 0.878 and a standard error of 0.020 in were reported for this model.[27]

Multiple regression analyses were also used on the field data from 74 doweled pavement projects located in Illinois with several different designs, materials, climatic conditions, and traffic variables.[28] Parameters considered in the prediction model for doweled pavements include:

- Subbase thickness.
- Dowel diameter.
- Slab thickness.
- k-value of foundation.
- 18-kip ESAL over life.
- Average annual temperature.
- Age of concrete sections.

A coefficient of correlation of 0.76 and a standard error of 0.034 in were reported for this model.[28]

Durability of the PCC slab is not considered in RISC.

BASE/SUBBASE

General

In addition to the surface slab, RISC accepts a maximum of three other layers in a pavement section. This allows the user a choice to specify a base and or subbase course, or separate the subgrade layer to better represent the existing soil conditions. The bonding condition between these layers is not an input in RISC and the assumption written into the program (bonded or unbonded) was not documented.

Stiffness

If a base or subbase is specified, the modulus of elasticity and Poisson's ratio must be directly input for the corresponding layers. These parameters are assumed constant under all slabs and shoulders considered.

Durability

Durability of the base or subbase is not considered in RISC. However, the modulus of elasticity may be reduced to reflect loss of stiffness.

Erodability/Loss of Pavement Support

Slabs may be assumed in contact with support layers, or voids may be assumed. Any edge, corner or combination of the two may be specified to be out of contact. Edge voids are assumed to be 18 in (46 cm) wide and extend along the entire edge. Corner voids are assumed to be triangular in shape with 36-in (91 cm) legs. If shoulders are present where corner voids are specified the void is assumed to extend under the shoulder slabs.

Drainability

Drainability of the pavement section is input as either good or poor. Due to lack of published information, RISC's actual method of consideration of this parameter cannot be determined.

Layer Thickness

The base or subbase thickness is directly input into the program. A constant thickness must be assumed under all slabs and shoulders considered.

SUBGRADE

General

The foundation under the slab is assumed to be a layered elastic solid and may consist of one, two, or three distinct layers. The top layers have finite thickness and the bottom layer thickness is infinite. Thus, base, subbase, or subgrade layers can be specified to best represent the pavement foundation conditions.

Stiffness

The Poisson's ratio, modulus of elasticity, and thickness (excluding the lowest layer) of each foundation layer specified must be input.

Drainability

See Base/Subbase Section.

Moisture sensitivity and volume change potential of the subgrade is not considered in RISC.

SHOULDERS

General

Shoulders are directly considered and may be specified as tied or untied to the traffic lane slab.

Materials

The Poisson's ratio, modulus of elasticity and 28-day flexural strength are assumed as specified for the traffic lane. The shoulder foundation is also assumed identical to that of the traffic lane.

Thickness and Geometry

The thickness of the shoulder slabs must be uniform but may vary from slab to slab and differ from the traffic lane slab. The width of the shoulder slabs is directly input into the program. The shoulder width is assumed constant for all shoulder slabs considered. Traverse joints must be aligned, therefore, shoulder slab lengths are assumed identical to the traffic lane slabs.

Reinforcement

Reinforcement in the shoulders may be considered as explained in the Reinforcing Steel section following.

Tying with Mainline and Jointing System

The longitudinal joint, between pavement and shoulder, may be untied or tied with deformed bars with specified diameter and spacing.

REINFORCING STEEL

General

RISC can indirectly model continuously reinforced and jointed reinforced concrete pavements by assuming that cracks or joints exist at short spacings. Reinforcing can then be assumed by specifying uniformly spaced dowels at the joints or cracks. As in ILLI-SLAB, this assumption requires a thorough knowledge of thin plate theory since extremely short spacings violate this theory and erroneous results may occur.

LOAD TRANSFER AT JOINTS

Aggregate interlock and Mechanical Devices.

Aggregate interlock is not considered in RISC. Transverse joints may be plain (completely free, load transfer = 0) or doweled. When dowels are specified the following parameters must be directly input into RISC:

- Number of dowels per transverse joint.
- Dowel bar diameter.
- Dowel bar looseness.

Dowel concrete interaction is calculated by the Friberg analysis with the modulus of dowel support, K, assumed as 1,500,000 pci (41520 kg/cm^3). Dowel looseness can be specified from 0 to 8 mils (0 to 0.2 mm).[25]

The longitudinal joint between pavement and shoulder may be plain (completely free, load transfer = 0) or tied with tie bars. When tie bars are specified the diameter and number of tie bars per linear foot of joint must be directly input.

JOINT DESIGN

Longitudinal and Transverse Spacing of Joints

RISC accepts any desired transverse joint spacing. Traffic lane and shoulder slabs may be any specified width but only one longitudinal joint is considered. Skewing of joints is not permitted.

Sealant

Joint sealant design and sealant properties are not considered in RISC. However, joint width is directly input into the program and is considered in load transfer calculations.

SHRINKAGE, CURLING and WARPING

Shrinkage and Warping

The physical effects of material shrinkage with time and warping due to moisture distributions through the PCC slab are not considered in RISC.

Curling

A linear vertical temperature gradient in the slabs may be specified to compute thermal stresses. The temperature gradient is considered positive when the higher temperature is at the top surface (opposite to that of ILLI-SLAB, WESLIQID, WESLAYER). Unit weight and coefficient of thermal expansion of the concrete slab is assumed in the program and is not a direct input. The numerical values assumed in the program were not documented. Detailed explanation of the curling computation is documented in reference 25.

DRAINAGE SYSTEM EFFECTIVENESS

Drainability of the pavement section is input as either good or poor. Due to lack of information, RISC's actual method of consideration of this parameter cannot be determined.

CLIMATE

Environmental effects are incorporated into the design through the AASHTO regional factor which is used to modify the traffic, i.e. , the design traffic for a particular climate is the estimated equivalent 18-kip axle loads times the regional factor.[30] The applicability of this concept to rigid pavements is highly questionable.

TRAFFIC

Truck Volume

The daily design traffic number (DTN) in terms of equivalent 18-kip axle loads, as developed from the AASHTO load equivalency factors, is directly input the program.[31] Both the fatigue and faulting prediction models use this parameter. Further explanation of this calculation is available.[25]

Axle Load Distribution

Traffic loading is assumed to be the result of a standard dual wheel truck axle [25/1] with a specified load magnitude. Load position may be at edge or corner positions. Midslab position may also be specified, but in this case only one set of dual wheels (1/2 axle) is used.

Truck Lane Distribution and Lateral Wander

The load placement is assumed to be 18 in from the edge to the center-line of the dual tire which attempts to reflect actual mean load placement. This does not take into consideration the increased damage from loads that are closer to the edge.

2.8.8 COMPUTATIONS

The RISC program was run over a range of input values very similar to ILLI-SLAB. Since RISC models the subgrade as an elastic solid and the other programs (ILLI-SLAB, JSLAB, WESLIQID) considered model the subgrade as a dense liquid, the results cannot be directly compared. RISC also assumes the location of the outer wheel in the predetermined corner and edge loading conditions to be 18 in from the mainline-shoulder joint. In ILLI-SLAB, the wheel loads were located directly against the mainline-shoulder joint (for the example, see figures 1 and 3). The two slab system with shoulders considered was assumed to have the identical dimensions as in the ILLI-SLAB analysis, illustrated in figure 5. The exact input is listed in tables 14 and 15 for the one- and two-layer pavement systems respectively.

Results are listed in table 16 for the corner and edge loading condition. The subgrade model and wheel load location differences made a comparison of the RISC results to other programs impossible. In the corner loading condition, dowel looseness of 0 and 8 mils were assumed and showed to have a very significant effect on load transfer efficiency and related slab stresses. Also the benefit of dowels in the pavement systems was reflected in the drastically increased predicted fatigue life. This could only be interpreted to mean fatigue cracking near the transverse joint (i.e., corner breaks or longitudinal cracking, not transverse cracking). Computer run times for these anaylses ranged between 30 and 45 CPU minutes.

A day and night time curling analysis was performed on 12 by 15 ft (3.7 by 4.6 m) slabs. As in the ILLI-SLAB analysis the slab thickness and subgrade stiffness were varied. The results are listed in table 17. The unit weight and coefficient of thermal expansion of the concrete slab was assumed internally in the program and was not documented. The stresses determined were very high and increased for increasing slab thickness and were relatively constant for increasing subgrade stiffness. All of these listed trends seemed highly questionable.

Table 14.

Parameters assumed in analysis for one-layer pavement RISC model.

TYPE OF PAVEMENT	JPCP	
SURFACE LAYER		
PCC SLAB THICKNESS	9	IN
POISSON'S RATIO	0.20	
MODULUS OF ELASTICITY	5000000	PSI
28 DAY FLEXURAL STRENGTH	750	PSI
SUBGRADE		
SUBGRADE MODEL	ELASTIC SOLID	
MODULUS OF ELASTICITY	20000	PSI
POISSON'S RATIO	0.40	
DOWEL AND JOINT PARAMETERS		
LONGITUDINAL JOINT		
TIE BAR DIAMETER	0.625	IN
TIE BAR SPACING	30.0	IN
JOINT WIDTH	0.25	IN
TRANSVERSE JOINT		
DOWEL BAR DIAMETER	1.25	IN
DOWEL BAR SPACING	12.0	IN
DOWEL LOOSENESS	0.0	MILS
ACTUAL JOINT SPACING	15	FT
JOINT WIDTH	0.25	IN
LOADING		
WHEEL LOAD MAGNITUDE	9.0	KIPS
PREDETERMINED CORNER AND EDGE LOCATIONS		
TRAFFIC		
2-LANE, 2-WAY PAVEMENT		
DAILY 18-KIP ESAL	2740	EACH
DIRECTIONAL DIST. FACTOR	0.5	
LANE DISTRIBUTION FACTOR	1.0	
GROWTH FACTOR	1.0	
REGIONAL FACTOR	1.5	
DAILY TRAFFIC NUMBER	2055	EACH
DRAINAGE		
SUBGRADE DRAINAGE	GOOD	

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 kip = 454 kg

Table 15. Parameters assumed in analysis for two-layer pavement RISC model.

TYPE OF PAVEMENT	JPCP	
SURFACE LAYER		
PCC SLAB THICKNESS	9	IN
POISSON'S RATIO	0.20	
MODULUS OF ELASTICITY	5000000	PSI
28 DAY FLEXURAL STRENGTH	750	PSI
BASE		
THICKNESS	5	IN
POISSON'S RATIO	0.20	
MODULUS OF ELASTICITY	2000000	PSI
SUBGRADE		
SUBGRADE MODEL	ELASTIC SOLID	
MODULUS OF ELASTICITY	20000	PSI
POISSON'S RATIO	0.40	
DOWEL AND JOINT PARAMETERS		
LONGITUDINAL JOINT		
TIE BAR DIAMETER	0.625	IN
TIE BAR SPACING	30.0	IN
JOINT WIDTH	0.25	IN
TRANSVERSE JOINT		
DOWEL BAR DIAMETER	1.25	IN
DOWEL BAR SPACING	12.0	IN
DOWEL LOOSENESS	0.0	MILS
ACTUAL JOINT SPACING	15	FT
JOINT WIDTH	0.25	IN
LOADING		
WHEEL LOAD MAGNITUDE	9.0	KIPS
PREDETERMINED CORNER AND EDGE LOCATIONS		
TRAFFIC		
2-LANE, 2-WAY PAVEMENT		
DAILY 18 KIP ESAL	2740	EACH
DIRECTIONAL DIST. FACTOR	0.5	
LANE DISTRIBUTION FACTOR	1.0	
GROWTH FACTOR	1.0	
REGIONAL FACTOR	1.5	
DAILY TRAFFIC NUMBER	2055	EACH
DRAINAGE		
SUBGRADE DRAINAGE	GOOD	

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 kip = 454 kg

Table 16. RISC results for corner and longitudinal edge loading.

PROGRAM:RISC
RESULTS FOR 1-AND 2-LAYER PAVEMENT SYSTEMS
CORNER AND LONGITUDINAL EDGE LOADING CONDITIONS

LOADING CONDITION	NO. OF LAYERS	DOWELS	MINIMUM LOAD TRANSFER TRANS [%]	MAX. PRINCIPAL TENSION STRESS [PSI]	MAX. PRESSURE BENEATH SLAB [PSI]	MAXIMUM DEFLECTION [MILS]	PREDICTED FATIGUE LIFE [YEARS]	PREDICTED FAULTING [IN.]
CORNER								
	1	NO	0	227.6	14.5	12.2	4.9	0.09
	2	NO	0	186.9	25.4	9.5	11.5	0.14
	1	YES	98.5	138.5	9.1	9.8	41.5	0.34
	1	**YES	52.6	170.2	10.1	18.2	17.2	0.07
	2	YES	98.7	120.4	21.8	8.1	75.7	0.15
LONGITUDINAL EDGE								
	1	NO	0	160.7	6.3	7.7	21.9	0.24
	2	NO	0	146.9	4.9	7.1	32.2	0.30
	1	YES	99.5	144.9	2.7	7.2	34.3	0.25
	2	YES	99.2	133.8	4.9	6.7	48.3	0.28

** DOWEL LOOSENESS OF 8 MILS ASSUMED IN TRANSVERSE JOINT

1 in = 2.54 cm
1 psi = 0.07031 kg/cm^2
1 mil = 0.0254 mm

Table 17. RISC curling analysis results.

PROGRAM: RISC
RESULTS FOR CURLING ANALYSIS WITHOUT TRAFFIC LOAD
MAXIMUM PRINCIPAL SLAB STRESS

SLAB THICKNESS (IN)	SLAB SIZE (FT)	SUBGRADE "E" VALUE (PSI) 20000	20000	50000	50000
		TEMPERATURE GRADIENT (DEGREES F/INCH) 1.5	-3.0	1.5	-3.0
8.0	12 by 15	312.9	609.7	313.3	609.5
10.0	12 by 15	392.2	771.1	392.5	771.1

Note: A positive temperature gradient is defined as slab bottom temperature higher than slab top temperature.

Unit Weight and Coefficient of Thermal Expansion of Concrete are as assumed internally in program.

1 in = 2.54 cm
1 ft = 0.3048 m
°F = (°C x 1.8) + 32

2.9 The CMS Model

The Climatic-Material-Structural (CMS) model is a computer program designed to develop climatic variables of importance to the structural analysis of a pavement structure.[32,33] In its current version, it is set up for a flexible pavement. The program consists of two major subprograms, a heat transfer model and a moisture movement model. These two programs calculate the climatic input values required in pavement design of a flexible pavement. The heat transfer model calculates the temperature gradient in the pavement structure during the day. The moisture model calculates the moisture condition in the subgrade over time. The temperature gradient is critical for flexible pavements because of the stiffness-temperature relationship for the asphalt concrete surfacing. The stiffness of the asphalt concrete affects the fatigue life of the asphalt concrete, the rutting potential, and the thickness selection for the surface layer. The moisture condition of the subgrade interacts with the material properties of the subgrade to produce a variable resilient modulus throughout the year. The prediction of the seasonal variation in resilient modulus is a critical design value which must be accounted for in a mechanistic design procedure.

The integration of these two models into one package to calculate the two most important values for a flexible pavement represents a step forward in the inclusion of environmental variables in pavement design. The outputs of this program can be used as input for any structural model to evaluate the impact of environment on the load carrying capacity of the pavement. The model itself contains no structural model to calculate load response.

The input parameters required include the thermal characterization of the material layers, the environmental parameters of the geographical location for the pavement, the moisture parameters of the subgrade, and the resilient modulus-moisture relationship of the subgrade. If desired, asphalt cement properties can be input to calculate the stiffness of the surface using the van Der Poel nomographic procedure.

<u>Applicability to Rigid Pavements</u>

This model can be immediately applied to rigid pavement design without significant modification. Modifications required would be primarily in the temperature output desired for calculation of curling and warping stresses in the PCC slab. The program calculates temperature profiles at specified intervals throughout the day. Any structural model which accepts a temperature gradient could use the data from the CMS model. The moisture model provides the resilient modulus which is now the standard in the AASHTO design method. To be used in other design procedures, a conversion between resilient modulus and k value would have to be developed.

2.10 BERM Design Procedure Program

2.10.1 Introduction

The BERM computer program is the structural analysis portion of a procedure for the structural design of roadway shoulders.[25] An interactive microcomputer version and a batch-type mainframe version are available. The BERM program was developed using regression equations for critical stresses and strains and to predict the expected life of the shoulder design in terms of 18-kip equivalent single axle loads (ESAL).

As in JCS-1, the traffic lane pavement design is only considered by the structural support it provides to the shoulder through the longitudinal joint.

2.10.2 Capabilities

The RISC finite element program and a cracking distress model, developed from the AASHO Road Test data, were used to develop the regression equations employed by BERM. Both flexible and rigid shoulders can be designed with the procedure. The following pavement-shoulder combinations can be analyzed:

TRAFFIC LANE PAVEMENT	SHOULDER
Rigid (New Construction)	Tied, Keyed Rigid
	Tied, Rigid
	Flexible
Rigid (Existing Pavement)	Tied Rigid
	Flexible
Rigid (Widened Lane)	Flexible
Flexible (New Construction)	Flexible Monolithic
Flexible (Existing Pavement)	Flexible

The critical inner and outer edges are designed using fatigue distress functions and stress/strains resulting from encroaching and parked vehicles, similar to the JCS-1 procedures.

2.10.3 Limitations

The critical stress/strain regression models in the BERM program were developed from output generated by the RISC finite element program; therefore, they are subjected to the limits discuss in section 2.8.6 of the RISC discussion. In addition, the regression models employed by BERM were developed and limited to the range of variables listed in table 18. The results of the regression analyses on the data considered documented in the Berm manual and show very good correlation. A detailed analysis on the sensitivity of variations in the input parameters is also discussed in the Berm manual.

Table 18.

Range of variables used in model development.

VARIABLE	LOW	HIGH	UNITS	NO. OF LEVELS
RIGID MODEL				
Econc	3000.	6000.	KSI	4
Hconc	4.	12.	INCHES	5
Ebase	10.	40.	KSI	4
*Hbase	4.	12.	INCHES	3
Esubgrade	3.	30.	KSI	7
*Tie Bar. Dia.	0.5	1.0	INCHES	3
*Tie Spacing	24.	48.0	INCHES	3
FLEXIBLE MODEL, ENCROACHED TRAFFIC				
Easph.	150.	1000.	KSI	6
Hasph.	4.	12.	INCHES	5
Ebase	10.	40.	KSI	4
Hbase	4.	12.	INCHES	3
Esubgrade	3.	30.	KSI	7
FLEXIBLE MODEL, PARKED TRAFFIC				
Easph.	5.	150.	KSI	6
Hasph.	4.	12.	INCHES	5
Ebase	10.	40.	KSI	4
Hbase	4.	12.	INCHES	3
Esubgrade	3.	30.	KSI	7

Note: E = Modulus of Elasticty of Material.
H = Layer Thickness of Material
* = Did not enter regression model.

1 ksi = 70.31 kg/cm^2
1 in = 2.54 cm

2.10.4 Input and Output

The program input required is controlled by the type of mainline pavement and shoulder that is chosen (the possible combinations were listed previously). For example, the input required for a rigid mainline pavement with a tied rigid shoulder includes:

- Trial concrete slab thickness.
- Modulus of elasticity of conrete.
- Modulus of elasticity of base.
- Modulus of elasticity of subgrade.
- Concrete flexural strength.
- Design temperature.

The output produced by BERM includes:

- List of input data.

- Computed design lives in 18-kip equivalent single-axle loads (ESAL) at the inner and outer shoulder edge locations for the trial slab thickness and the trial slab thickness plus 1 in. These values are not the actual traffic in the outer lane, but the allowable loads on the shoulder pavement.

2.10.5 Calculations

Sample runs of the BERM program were performed. A rigid mainline pavement with a tied rigid shoulder was selected. Design lives, in 18-kip ESAL, were computed for shoulder slab thicknesses of 6, 7, 8, and 9 in to determine a range of predicted lives as in JCS-1. The program input and output is listed in tables 19 and 20. The design temperature (not listed) was assumed at 70 ^{o}F (21 ^{o}C). The program predicted equal design lives for inner and outer edges of the shoulder. This is a result of a conservative assumption in the program which neglects any load transfer across the longitudinal, mainline-shoulder joint that could be developed by tie bars. The effect of this tie is considered in the JCS-1 program and it has a major effect on the life of the inner shoulder joint. A detailed traffic analysis and other procedures are necessary to determine the required shoulder thickness.

Table 19.

Rigid tied shoulder design.

PAVEMENT/SHOULDER JOINT IS TIED	
CONCRETE MODULUS:	5000. ksi (34.5 GPa)
CONCRETE FLEX STRENGTH	750. psi (5.17 MPa)
SUBBASE MODULUS:	20.0 ksi (138. MPa)
SUBGRADE MODULUS:	5.0 ksi (34. MPa)

THE COMPUTED DESIGN LIVES, IN MILLIONS EQUIVALENT SINGLE AXLE LOADS

SURFACE THICKNESS (in.)	(mm)	OUTER EDGE	INNER EDGE
6.0	(152)	0.547*	0.547*
7.0	(177)	1.204	1.204
8.0	(203)	2.433	2.433
9.0	(228)	4.606	4.606

* Note that when there is a tied shoulder, there should be a difference in computed design lines with the outer "free edge" of the shoulder.

1 psi = 0.07031 kg/cm^2
1 in = 2.54 cm

Table 20.

Rigid tied and keyed shoulder design.

PAVEMENT/SHOULDER JOINT IS TIED, KEYED

CONCRETE MODULUS:	5000. ksi (34.5 GPa)
CONCRETE FLEX STRENGTH:	750. psi (5.17 MPa)
SUBBASE MODULUS:	20.0 ksi (138. MPa)
SUBGRADE MODULUS:	5.0 ksi (34. MPa)

THE COMPUTED DESIGN LIVES, IN MILLIONS EQUIVALENT SINGLE-AXLE LOADS

SURFACE THICKNESS (in) (mm)	OUTER EDGE	INNER EDGE
6.0 (152.4)	.547	.897
7.0 (177.8)	1.204	2.156
8.0 (203.2)	2.433	4.629
9.0 (228.6)	4.606	9.144

1 psi = 0.07031 kg/cm^2
1 in = 2.54 cm

3.0 CRITIQUE OF CURRENT DESIGN METHODS

The objective of this section is to evaluate several available rigid pavement design methods utilizing selected data from existing databanks. Comparisons and evaluations were conducted to determine whether or not the design procedures consider those factors that are known to cause pavement distress, requiring correct maintenance or eventual rehabilitation.

The design procedures evaluated in this study are listed in table 21. The database used for jointed concrete pavements was developed under NCHRP Project 1-19.[42] The database used for CRCP included 132 sections from Illinois collected in 1977 plus another set of data collected in 1985.[60,63]

The evaluation of commonly used design procedures consists of two complementary parts: conceptual and analytical.

Conceptual Evaluation

This involves an analysis of the fundamental basis for the development of each procedure, an assessment of theory and data, and a critical review of the assumptions used in development of each method. The probable limitations of each procedure in designing against important distress types existing in rigid pavements are also discussed.

Analytical Evaluation

The analytical evaluation consistsof two appraches: comparison of actual ESALS and specific design evaluation.

(a) The actual number of 18-kip equivalent single axle loads (ESALs) is compared to the predicted ESAL due to the measured loss in present serviceability index using the existing performance model in the design procedure. This is done for each section of pavement of JPCP and JRCP in the database for different climatic zones. Some procedures do not use the serviceability concept, however, so this analysis cannot be accomplished for all procedures.

(b) Specific design evaluation: A number of typical design situations are developed for the four major climatic zones. The design factors vary, including types of subgrade soil, types of load transfer (JPCP only) and joint spacing (JRCP only). These designs are then evaluated using the COPES database-derived predictive models for cracking, faulting, joint deterioration, pumping and serviceability rating. An Illinois CRCP database-derived predictive model was developed to evaluate the CRCP designs for failures per mile (steel ruptures, edge punchouts, and existing full-depth repairs).

This dual approach to design procedure evaluation provides an overall picture of the capabilities of each design method by combining the theoretical evaluation with actual field performance evaluation.

Table 21.

Summary of rigid pavement design methods that were evaluated.

Design Method	Year of Version	Type of Pavement
1. 1986 AASHTO Design Guide [39]	1985	JPCP, JRCP, CRCP
2. Zero-Maintenance Design [46,47,48]	1977	JPCP
3. California DOT Procedure [49]	1985	JPCP
4. Portland Cement Association [53]	1984	JPCP, JRCP
5. RPS-3 Texas SHDPT Design [59,60]	1975	JRCP, CRCP
6. Associated ReBar Producers-CRSI [66]	1981	CRCP
7. Illinois DOT Procedure [69]	1982	CRCP

Number in parentheses refer to the reference number of the procedure.

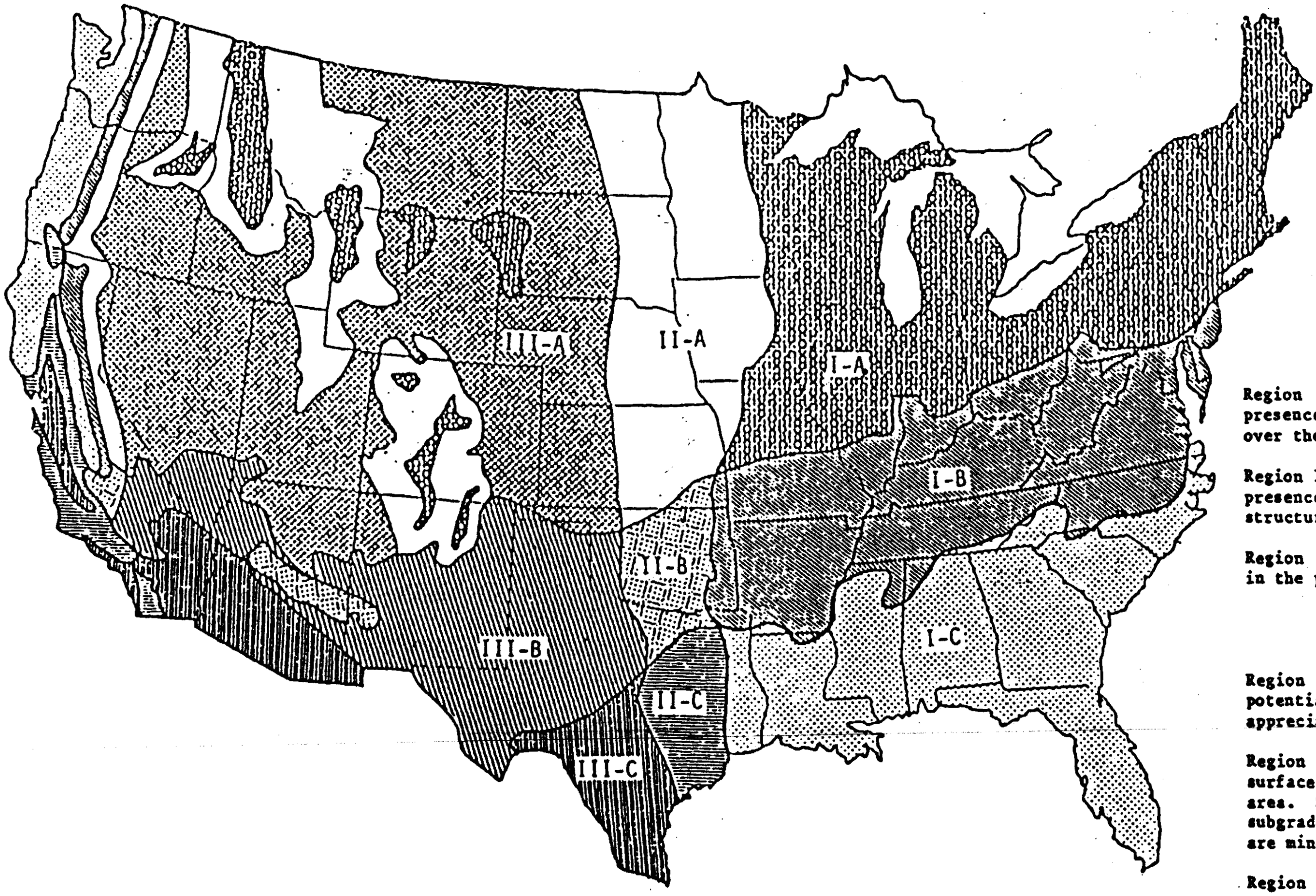
3.1 Description of National Climatic Zones.

A substantial amount of the distress in a pavement system can be related to the effects of moisture and temperature. The two basic quantities of moisture and temperature, acting singularly or in conjunction, have been shown to strongly contribute to distress development in rigid pavements.[1,4] The combined effects of the interaction of moisture and temperature on pavement performance were studied and nine climatic zones in the United States were identified.[34] The calculations to determine which zone a particular pavement falls into is primarily based on Thornthwaite's four climatic criteria: moisture adequacy, thermal efficiency, summer concentration of thermal efficiency, and seasonal distribution of moisture adequacy. The Thornthwaite classification system provides an excellent base for examining the climatic influences on pavement systems. The tentative boundaries for the nine climatic zones are shown in figure 7. These nine zones differentiate different moisture and temperature effects. Similar pavement designs should give similar behavior within each zone. The nine zones are described as follows: [34]

1. I-A, Low temperature - high moisture.
2. I-B, Freeze-thaw temperature - high moisture.
3. I-C, High temperature - high moisture.
4. II-A, Low temperature - variable moisture.
5. II-B, Freeze-thaw temperature - variable moisture.
6. II-C, High temperature - variable moisture.
7. III-A, Low temperature - low moisture.
8. III-B, Freeze-thaw temperature - low moisture.
9. III-C, High temperature - low moisture.

All nine zones were considered in the initial evaluation work as documented in appendix A. The results showed that the major four zones (I-A, I-C, III-A, III-C) had a significant effect on performance. The middle zones (II-A, IB, II-B, III-B, II-C) usually had an intermediate effect between that of the major four zones. One exception to this was zone I-B and II-B for JRCP, where somewhat more deterioration occurred than in adjacent zones. Therefore, in order to keep the analysis to a reasonable level, only the four major zones were utilized in the evaluations.

For the purpose of this study, four specific climatic variables were selected and their mean values for each of the nine climatic zones is shown in table 22.



Region I - High potential for moisture presence in the entire pavement structure over the entire year.

Region II - Seasonal variability in the presence of moisture in the pavement structure.

Region III - Very little moisture present in the pavement structure during the year.

Region A - Severe winters with a high potential for frost penetration to appreciable depths into the subgrade.

Region B - Freeze-thaw cycles in the surface and base course will dominate this area. Severe winters may produce frozen subgrades but long term freezing problems are minor.

Region C - Low temperatures are not a problem, high - temperature stability problems should be considered.

Figure 7. United States climatic zones.

Table 22.

Specific variables for the nine climatic zones.

Thermal Climates	Moist Climates		
	Dry (III)	Seasonally Wet (II)	Wet (I)
Freeze (A)	III-A *Prec : 30 Frze : 650 Temp : 8.3 T.R. : 41	II-A Prec : 61 Frze : 650 Temp : 9.4 T.R. : 44	I-A Prec : 89 Frze : 650 Temp : 8.9 T.R. : 40
Freeze-Thaw (B)	III-B Prec : 46 Frze : 75 Temp : 16.7 T.R. : 33	II-B Prec : 88 Frze : 50 Temp : 15 T.R. : 36	I-B Prec : 122 Frze : 75 Temp : 15 T.R. : 34
No Freeze Activity (C)	III-C Prec : 61 Frze : 0 Temp : 21.7 T.R. : 29	II-C Prec : 102 Frze : 0 Temp : 21.1 T.R. : 30	I-C Prec : 142 Frze : 0 Temp : 18.9 T.R. : 26

*NOTES: Prec = Mean annual precipitation, cms
Frze = Corps of Engineers Freezing Index, degree-days
Temp = Mean annual temprature, ^{o}C
T.R. = Temperature range (difference between ave. daily max temp. in July and ave. daily min. temp. in January), ^{o}C

3.2. 1986 AASHTO Design Guide for JPCP and JRCP

The "AASHO Interim Guide for Design of Pavement Structures" was developed in 1962 by the AASHO Design Committee through its subcommittee on Pavement Design Practices.[36] The Guide was evaluated and partly revised in 1972 and 1981.[37,38] In 1986, the subcommittee on Pavement Design revised the old guide under NCHRP Project 20-7/24 and issued the new version of the "AASHTO Guide for Design of Pavement Structures".[39] Some modifications were made in the design procedures of the previous version.

<u>Conceptual Evaluation</u>.

The rigid pavement design procedure is an extension of the pavement performance models developed from the results of the AASHO Road Test conducted near Ottawa, Illinois, from 1958 to 1960. The results relating load magnitude and repetition with thickness were modified and extended using Spangler's corner stress formula, which was developed from theoretical considerations of slab behavior, field observation, and laboratory investigations.[40] The nature of the design equation is basically empirical. A complete description of the development of the structural design model is given in the Appendix of the 1981 Interim Guide and in the AASHO Road Test report.[38,41] The following information summarizes the fundamental bases for development of the design procedure and its conceptual evaluation.

- The Road Test data provided empirical relationships between PCC slab thickness, load magnitude, axle type, number of load applications, and serviceability index of the pavement for Road Test conditions (i.e., specific environment and materials).

$$\log_{10} W = \log_{10} r + G/B \tag{2}$$

where W = axle load applications, for load magnitude L1 and axle type L2, to a serviceability index of P2

$$\log_{10} r = 5.85 + 7.35 \log (D + 1) + 4.62 \log (L1 + L2) + 3.28 \log (L2)$$

$$B = 1.0 + [3.63(L1+L2)^{5.20}] / [(D+1)^{8.46} L2^{3.52}]$$

$$G = \log [(P1-P2)/(P1-1.5)]$$

D = PCC slab thickness, inches
L1 = load on a single or a tandem axle, kips
L2 = axle code, 1 for single axles and 2 for tandem axles
P1 = initial serviceability index
P2 = terminal serviceability index

- This empirical model (equation 2) was modified and extended using the Spangler equation to include material properties including PCC flexural strength (F), PCC modulus of elasticity (E), and foundation support (k). The following basic assumptions were made in this extension:

a. There will be no variation in W for different load magnitudes if the level of the ratio of tensile stress/strength of the PCC slab is kept constant and such W will be accounted for by the AASHO Road Test equation 2.

b. Any change in the ratio tensile stress/strength resulting from changes in the values of E, k, and F (modulus of rupture) will have the same effect on W as an equivalent change in slab thickness (calculated by Spangler's equation) will have on W as per equation 2.

- Reliability concepts are introduced into the design process in order to decrease the risk of premature structural deterioration below acceptable levels of serviceability. The reliability design factor (F_R), accounts for chance variations in both the traffic prediction and the pavement performance prediction for a given W_{18}. This factor provides a predetermined level of reliability (R%) that pavement sections will survive the traffic for which they were designed.

- A so called drainage coefficient (C_d) based on the quality of drainage and the percent of the time the pavement structure is exposed to moisture levels approaching saturation is added to the design equation to provide an approximate way to consider the effect of drainage. This was done by modifying the load transfer coefficient, J. The C_d provides a relative basis of comparison as the value for C_d for condition at the AASHO Road Test is 1.0.

- The potential effect of subgrade swelling and frost heave are considered on the rate of loss in serviceability ($\triangle$ $PSI_{SW,FH}$).

- A Loss of Support factor (LS) is included in the design to account for the potential loss of support arising from subbase erosion and/or differential vertical soil movement. This LS factor diminishing the overall effective k-value based on the size of the void that may develop beneath the slab. Some suggested ranges of LS, depending on the type of subbase material, are provided.

The resulting final structural design model is given as follows:

$$W_{18} = W_{18}/F_R = W_{18}/10^{-Z_R S_o} \quad (3)$$

Where:

$$\log W_{18} = 7.35 \log(D+1) - 0.06 + Gt / [\ 1 + 1.624x10^7/(D+1)^{8.46} + (\ 4.22-0.32P2)*\log [\ (Sc*Cd\ (\ D^{.75} - 1.132\ )\ / (\ 215.63\ J)*(\ D^{.75} - 18.42/(Ec/k)^{.25}\ )\ ] \quad (4)$$

$$Gt = \log [(P1-P2)/(P1-1.5)]$$

where W_{18} = predicted number of 18-kip single-axle load applications
F_R = reliability design factor
Z_R = standard normal deviate corresponding to selected level of reliability
S_o = overall standard deviation for rigid pavement
D = thickness of pavement slab, inches
P_i = initial serviceability index
P_t = terminal serviceability
Sc = modulus of rupture for PCC used on specific project
J = load transfer coefficient used to adjust for load transfer characteristics of specific design
Cd = drainage coefficient
Ec = modulus of elasticity for PCC, psi
k = modulus of subgrade reaction, pci

Specific Conditions of the AASHO Road Test - The general conditions under which the basic structural design equation was developed from the field performance results are as follows:

1. Construction control - Construction was of extremely high quality, therefore, variations in concrete, aggregates, moisture, density, subgrade soil properties, etc., were much lower than can be expected in most normal highway construction.

2. Length of pavements - The length of the test section was 120 ft for the JPCP and 240 ft for the JRCP. The slab lengths are discussed under item 6.

3. Subbase - Subbase was an untreated densely graded sand-gravel with significant fines. This material pumped extensively on many sections which was a major reason for the failure of these sections.

4. Subgrade - Subgrade is a fine-grained A-6 soil with CBR ranging from 2 to 4, and a modulus of subgrade reaction of 45 pci is measured in the spring after the initial thaw.

5. Climate - Climate in northern Illinois has about 30 in annual precipitation and +4 in more annual precipitation than evaporation, thus a positive Thornthwaite Index of 30 exists in this area. The average depth of frost penetration is about 30 in, and the number of freeze-thaw cycles is 12 per year at the subbase level in the pavement.

6. Joints and Reinforcement - All joints were contraction type joints with dowel bars. Reinforcement with wire mesh was placed in slabs with 40 ft joint spacing. No reinforcement was used in the JPCP slabs with 15 ft joint spacing.

7. Length of Test - The test was conducted over a 2-year period, too short for effective evaluation of corrosion of mesh on dowels and deterioration of concrete.

8. Number of Load Applications - The total number of load applications applied to each loop was 1,114,000.

Accuracy of Structural Design Model - The empirical equation 2 was derived from results from the Road Test data, and relates specifically to the conditions listed above. Within these conditions, the ability of equation 2 to predict the exact number of load applications to any given level of serviceability index for a pavement was as shown in figure 8.[41] The shaded band indicates the range in load applications that includes approximately 90 percent of all the performance data. Referring to the top curve in figure 8, for example, if the slab thicknesses were 8 in (20.3 cm), the resulting number of 30-kip single-axle applications to a terminal serviceability index of 2.0 ranged between 400,000 to 1,910,000 for controlled AASHO Road Test conditions. If equation 2 is used for conditions different than those for which it was developed, its range of accuracy or associated error of prediction will be greater. This may be particularly true for different climatic conditions. The modified expression, equation 4, allows for changes in k, Ec, and Sc, but the accuracy of these adjustments is unknown.

Joint Design - The new Guide recommends three types of joints: contraction, construction, and expansion. It is recommended that the local service records are the best guide for the joint spacing design. A rough guideline for the joint spacing of JPCP is that the joint spacing (in feet) should not greatly exceed twice the slab thickness (in inches). Also, for JPCP the ratio of slab width to length should not exeed 1.25 as a general guideline. Skewed joints and randomized spacing are recommended for JPCP to minimize the effect of joint roughness. Joint dimensions are suggusted as 1/4 of the slab thickness for transverse contraction joints, and 1/3 of the depth thickness for longitudinal joints. A general guideline provided that the dowel diameter in inches should be equal to the slab thickness multiplied by 1/8, and the dowel spacing and length is 12-in and 18-in, respectively. The value of J recommended for a JPCP or JRCP with some type of load transfer device (LTD, such as dowel bars) at joints but without tied concrete shoulders is 3.2. For jointed pavements without load transfer at the joints, a value from 3.8 to 4.4 is recommended.

Drainage Design - In the new Guide, drainage effects are considered in terms of the effect of moisture on subgrade strength and on base erodability. For new rigid pavement design, the effect of drainage is considered by modifying the load transfer coefficient, J. Rationale as to how the subdrainage of a rigid pavement relates to joint load transfer is not provided. The recommended design of subsurface drainage systems is provided in the appendix of the Guide.

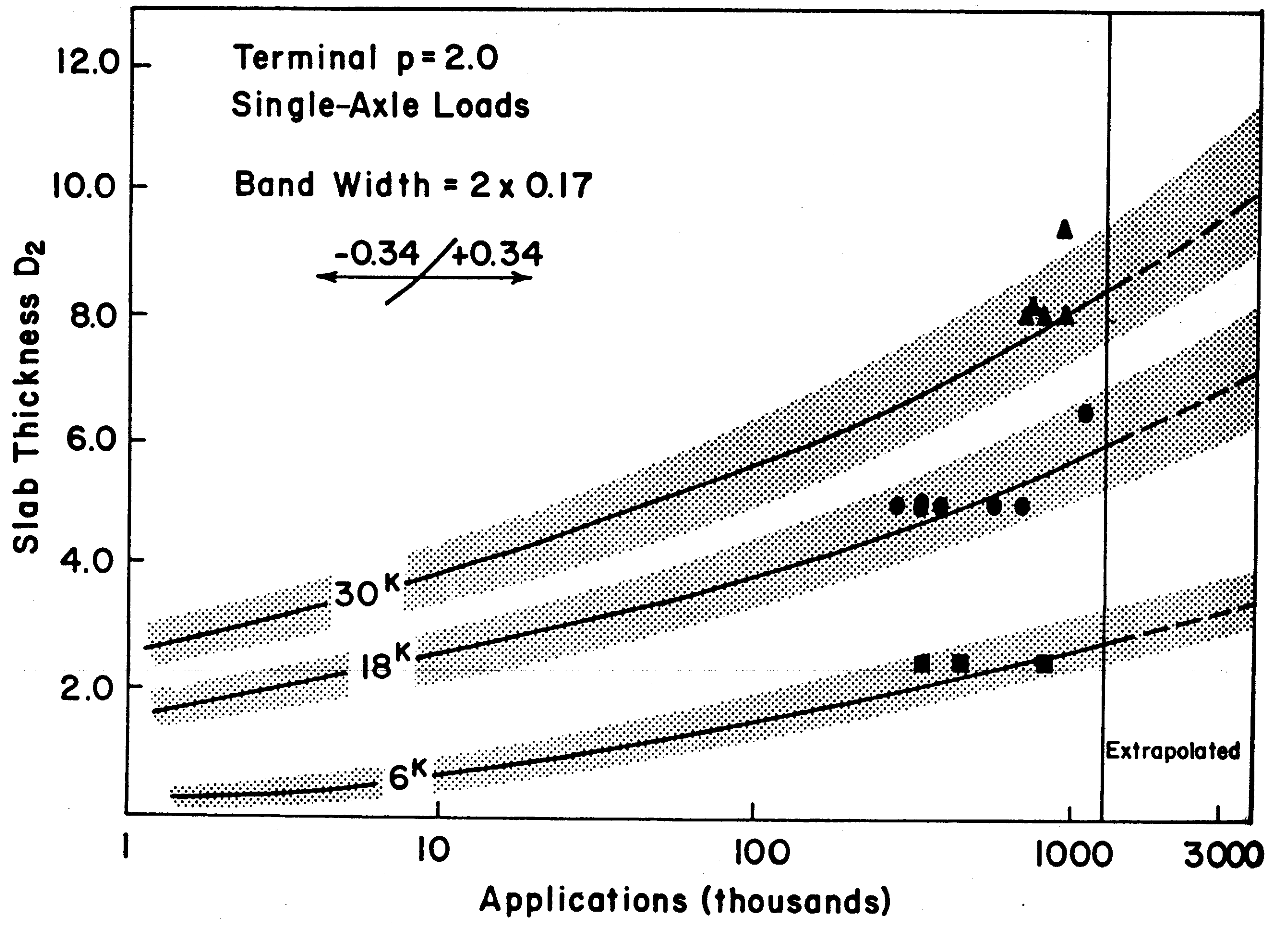


Figure 8. Illustration of error of prediction of basic AASHO design model. [from 41]

Reinforcement Design - Slab reinforcement is designed using the "subgrade drag" theory. The basic expression is as follows:

$$P_s = \frac{L\ F}{2fs} * 100 \qquad (5)$$

where P_s = amount of steel required as a percentage of the concrete cross-sectional area
L = distance between free transverse joints, ft
F = coefficient of friction between the bottom of the slab and the top of the underlying subbase or subgrade
fs = allowable working stress in the steel, psi

Based on these facts, some limitations of the design procedure in minimizing significant distress types in rigid pavement are summarized as follows:

1. Variability - A serious limitation of the AASHTO design procedure is that equations 2 and 4 are based upon very short pavement sections where construction and material quality was highly controlled. Typical highway projects which are normally several miles in length contain much greater construction and material variability, and hence show more variability in performance along the project in the form of localized failures. Projects designed using the Guide would, therefore, have the tendency to show significant localized failures before the average project serviceability index drops to P_t, unless a higher level of reliability was selected for the design.

2. Loss of Foundation Support - The Road Test used a specific set of pavement materials and one roadbed soil. Many of the Road Test sections showed severe pumping of the subbase. Therefore, the equation 2 and 4 are biased towards this condition though the effects of drainage can now be adjusted to some extent.

3. Design Period - Design periods under consideration usually range from 20 to 40 years. The number of years and 1,114,000 applications, upon which equations 2 and 4 are based, represent only a fraction of the load applications that would be expected on those high volume pavements over the design period (10 to 100 million 18-kip ESAL). Even if these equations can be extrapolated for the large difference in the number of load applications, there are several climatic effects that occur with time (as represented by age) to cause severe deterioration of the pavements even without heavy load applications (i.e., corrosion of steel, joint freeze-up, D-cracking, reactive aggregate, etc.). Therefore, in similar or more severe climates, the pavements would be expected to endure fewer load applications and fewer years than predicted by equation 4. In mild climates, pavements would be expected to perform much better than predicted.

4. Joint Design - Only one type of joint design was used at the Road Test. If other types are used, such as joints without dowels (as evidenced by the performance of the transverse cracks), or with some unsual type of load transfer devices, the pavement life would be significantly changed. The type of base would also affect load transfer and thus performance. Basic deficiencies in the joint design recommendations are little or no guidance for (1) joint spacing; (2) rational determination for dowel size and spacing (3) corrosive resistant dowels; (4) when mechanical LTDs are required; and (5) load transfer system other than dowels.

5. Reinforcement Design - The mathematical expression used for longitudinal reinforcement design is a major simplification of the actual forces encountered. The most significant limitation arises if the unrestrained slab length assumed in reinforcement design (i.e., distance between joints, L) is altered through a partial or complete seizing of one or more joints. This could cause a significant increase (double or more) in the steel stress, which may result in yielding or rupture of reinforcement at an intermediate crack between joints. Also, the loss of effective reinforcement through corrosion is not provided for in the procedure. It is expected therefore that long joint spacings in cold regions accompanied by joint seizure would result in rupture of the reinforcement with subsequent faulting and spalling of cracks.

6. Climate - Concrete pavement performance is not independent of climatic conditions, and there is evidence to indicate that climatic conditions could have a significant effect on pavement life.[33] Since the Road Test was conducted over a period of only 2 years, climatic effects were not as significant as if the same traffic had been applied over a longer period of, say, 20 to 40 years. Steel corrosion requires several years to develop into a serious condition, so joint lockup and subsequent yielding of the steel reinforcement for JRCP pavements would logically not occur for at least several years after initial construction. Figure 9 shows the results of a life prediction model where age and traffic data were available over both short and long time periods.[42] An interaction between age and traffic can be observed in that at old ages and heavy traffic there is much greater pavement damage.

7. Load Equivalency Factors - The load equivalency factors relate specifically to the road test materials, pavement composition, climate, present serviceability index loss and subgrade soils. The accuracy of extrapolations applied to other regions, materials and distresses, etc., is not known, and remains questionable.

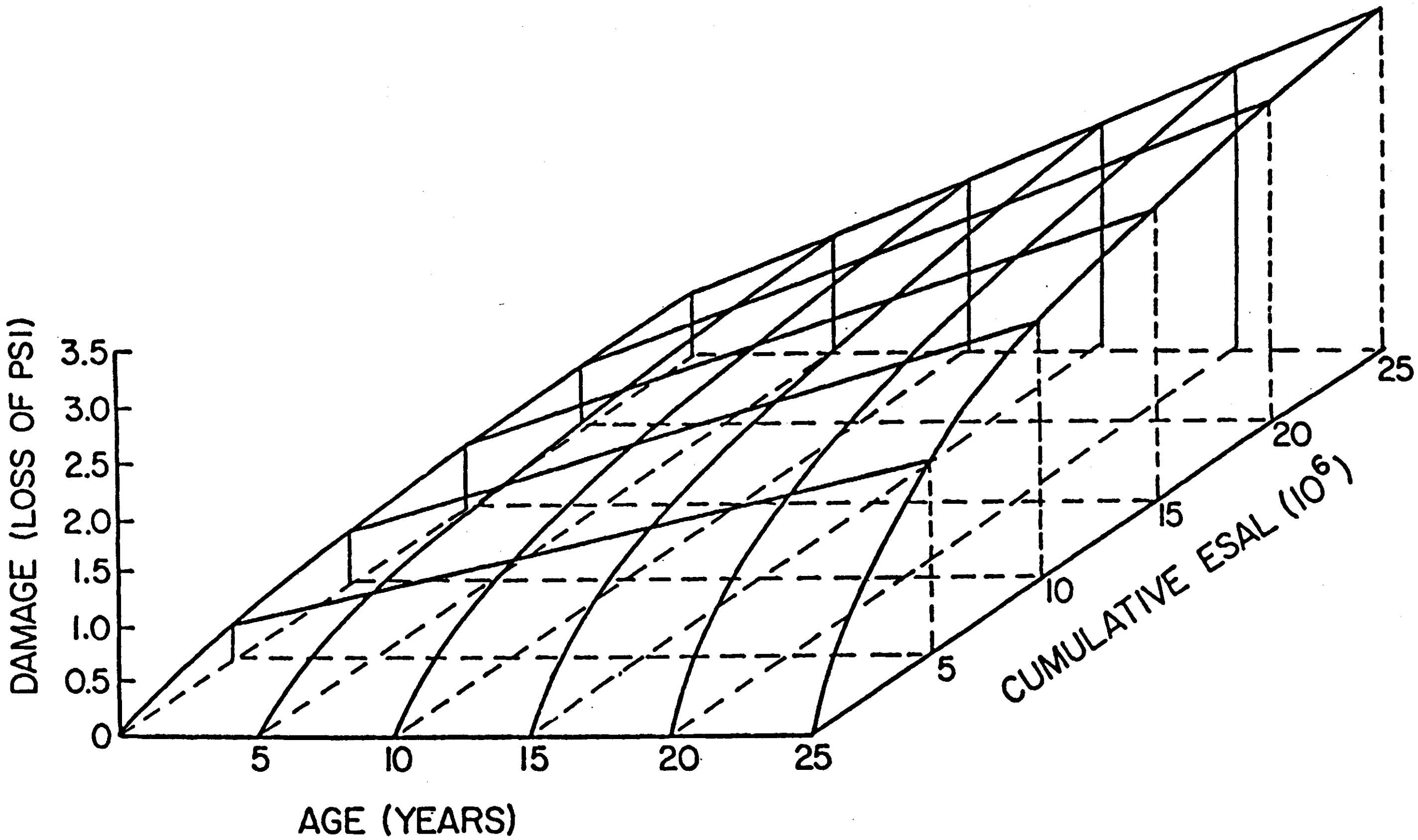


Figure 9. Sensitivity of Illinois damage (serviceability) model to cumulative load repetitions and age (three-dimensional illustration).[42]

Analytical Evaluation.

The analytical evaluation was conducted using two approaches: Predicted vs. Actual ESALs and Specific Design Evaluation.

Predicted vs. Actual ESALs. The actual number of ESALs was compared to the predicted ESAL due to the measured loss in present serviceability index using the AASHTO performance equation (equation 4). This was done for each section of pavement of JPCP and JRCP in the COPES database. The actual pavement thicknesses, material properties, serviceability at the time of the study, and actual traffic from the COPES database were input into the equation. The drainage coefficient value for the equation was set at 1.0. The value of the J factor was assumed as 3.2 for joints with dowels and 4.1 with aggregate interlock (without dowels). The data retrieval and computations were completed by utilizing the SPSS statistical package.[43] The analysis was run at the 50 percent level of reliability.

The pavement sections in the COPES databank were divided into four broad climatic zones and the results compared by zone. Following is the classification and data locations for the four climatic zones.

Climatic Zone	Annual Rainfall (cms)	Freezing Index (degree-days)	JPCP Location	JRCP Location
Wet-Nonfreeze	equal or greater than 70	less than 100	GA, LA	IL, LA
Wet-Freeze	equal or greater than 70	equal or greater than 100	IL	IL, MN
Dry-Nonfreeze	less than 70	less than 100	CA	-
Dry-Freeze	less than 70	equal or greater than 100	UT	MN, NB

The plots of predicted vs. actual ESALs for all JPCP and JRCP for each climatic region are given in figure 10 and figure 11, respectively. A summary of the results of the predicted vs actual ESALs for JPCP and JRCP is given in table 23. The significance and comparison of the results are discussed in the following sections.

1. JPCP: The results are highly dependent on climate. Almost all the sections in the dry-nonfreeze region performed better than the original AASTHO model predicted (52 out of 53 sections, or 98 percent). The average predicted ESALs to existing present serviceability index in this region is 6.4 million (or 69.5 percent) less than the actual ESALs. Those sections in the dry-freeze and wet-nonfreeze climates did perform generally as predicted with 60 and 71 percent of sections acceptable, respectively.

The JPCP sections in the wet-freeze climatic region (same as AASHO Road Test) performed worse than the AASHO model predicted with only 9 out of 36 sections (or 25 percent) acceptable. The average predicted ESALs in wet-freeze region is 7.36 million (or 92 percent) greater than the actual ESALs.

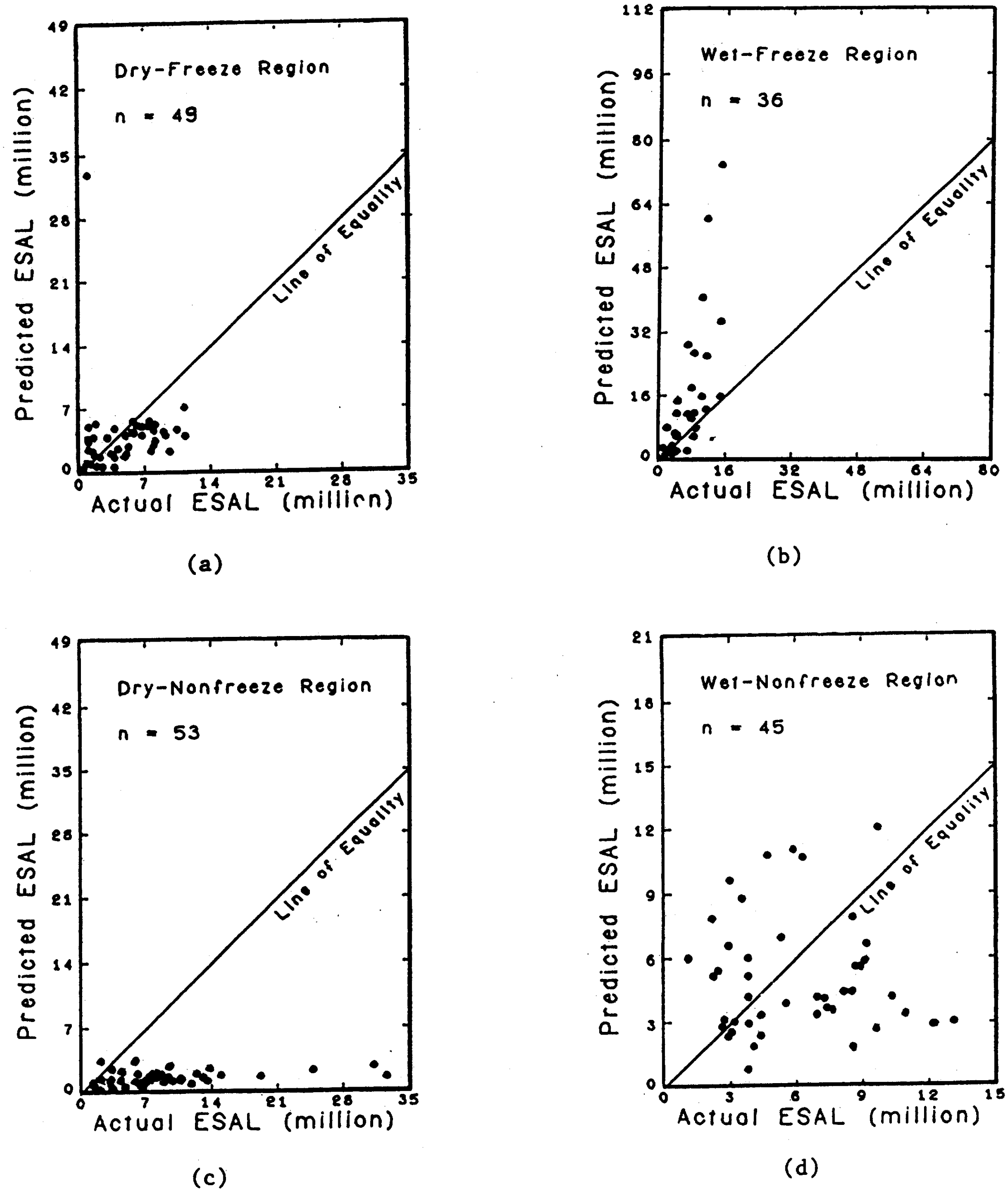
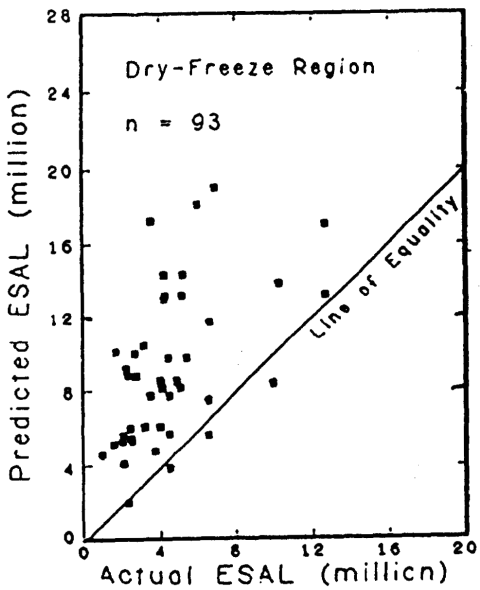
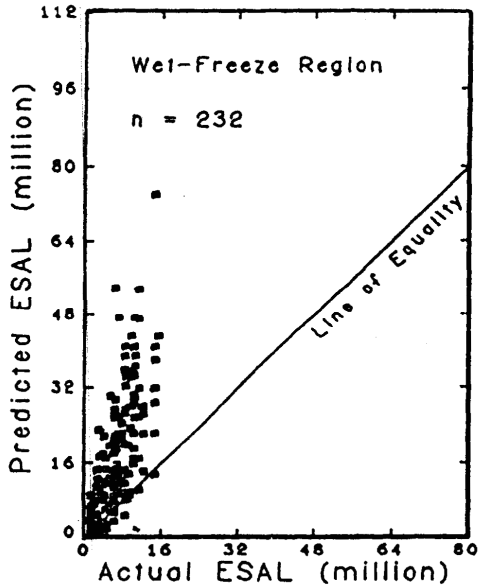


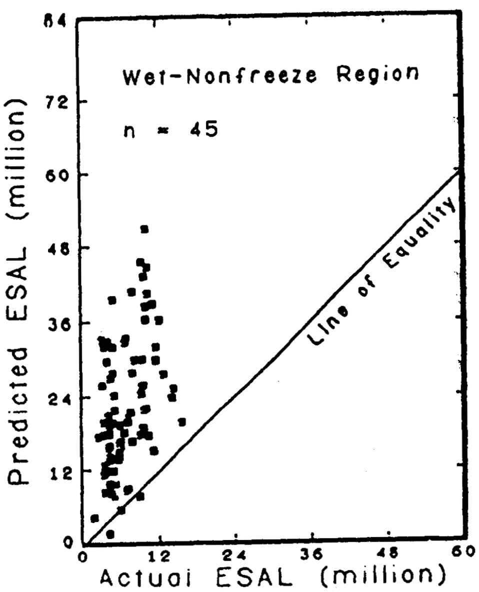
Figure 10. Predicted ESAL vs. actual ESAL for JRCP using original AASHO Road Test PSI prediction model.



(a)



(b)



(c)

Figure 11. Predicted ESAL vs. actual ESAL for JRCP using original AASHO Road Test PSI prediction model.

Table 23.
Summary of results for original AASHO
Road Test PSI prediction model.

Climatic Region (1)	# of Cases (2)	# of Section Acceptable* (3)	Percent Acceptable (4)=(3)/(2)	Mean Difference** (5)	Mean Percent Difference*** (6)
JPCP:					
Wet - Nonfreeze	45	27	60%	-0.911	17.1%
Wet - Freeze	36	9	25%	7.355	92.0%
Dry - Nonfreeze	53	52	98%	-6.405	-69.5%
Dry - Freeze	49	35	71%	-0.763	86.9%
overall	183	123	67%	-0.836	25.4%
JRCP:					
Wet - Nonfreeze	93	3	3%	15.460	282.0%
Wet - Freeze	232	34	15%	9.886	168.8%
Dry - Nonfreeze	-****	-	-	-	-
Dry - Freeze	49	4	8%	4.584	143.0%
Overall	374	41	11%	10.577	193.6%

* "Acceptable" means the actual number of 18-kip ESALs is equal or greater than the predicted ESALs, i.e., the pavement section performed as good as or better than the AASHO model predicted, otherwise is "unacceptable".

** Difference = Predicted ESAL - Actual ESAL, in millions

*** Percent Difference = [(Predicted ESAL - Actual ESAL)/Actual ESAL] x 100%

**** No JRCP sections available in dry-nonfreeze region in COPES.

2. JRCP: The results in Table 23 show that the JRCP sections did not perform as well as the original AASHO model predicted in any climatic zone. Only 41 out of 374 sections (or 11 percent) performed better than predicted. The average predicted ESALs for the all JRCP sections is 10.58 million (or 193.6 percent) greater than the actual ESALs.

Climatic factors affect JRCP more severely than JPCP. Many of these JRCP sections have deteriorated from causes other than traffic loading, e.g., build-up of compressive stress with long joint spacing, resulting in corrosion of dowels and mesh, "D" cracking or reactive aggregate, infiltration of incompressibles into joints causing blowups and joint spalling, etc. Thus, the AASHO model does not provide adequate designs. The inherent variability in the prediction of pavement section performance indicates there is a justification to increase the reliability level if the equation will provide adequate designs for all those sections.

Specific Design Evaluation. The specific design evaluation was conducted using climatic data from the COPES database and the nine national climatic zones. The results obtained using nine zones showed that significant difference occurred only between the four major zones: wet-freeze, wet-nonfreeze, dry-freeze and dry-nonfreeze. These results are given in appendix A. Due to the lack of significance and the complexity of presenting results, only the four major zones are presented herein.

A number of pavement design situations were developed for both JPCP and JRCP over the four climatic regions. The prevailing climatic values of each climatic region as summarized from COPES database are shown in table 24. The design factors that were varied included two subgrade soils (fine-grained and coarse-grained), with and without dowels (JPCP only) and shorter and longer joint spacings (JRCP only). The "fine-grained" subgrade soils were defined as the A-7-5 or A-7-6 in the AASHTO soil classification, while the A-2-6 or A-2-7 were for the "coarse-grained" subgrade soils. The drainage characteristics of these two types of subgrade soils was considered as "poor" for the fine-grained soils and "good" for the coarse-grained soils, respectively. A resilient modulus of 3,000 psi and 7,000 psi (211 and 492 kg/cm^2) was assumed in this analysis for the fine- and coarse-grained soils, respectively. The elastic k-values for fine- and coarse-grained soils were assumed to be 100 pci and 190 pci (2.8 and 5.3 kg/cm^3), respectively, when the subgrade was compacted to 70 to 90 percent degree of saturation. The new AASHTO Guide was then used to develop pavement designs for JPCP and JRCP for each of the design cell "situations". The design life was 20 years and the design traffic was 15 million 18-kip ESAL in the design lane.

Specific soil, subbase, concrete properties, etc. for the designs are shown in table 25. The climatic design inputs for the new AASHTO Guide for each of the four climatic zones are given in table 26. The values of the drainage coefficient C_d and loss of support factor LS recommended in the Guide were used. The dowel size and the reinforcement (JRCP only) were also designed as recommended. The designs were developed at reliability levels of 50, 80 and 90 percent.

These designs were then evaluated by using the predictive models developed from the COPES database for pumping, faulting, cracking, joint deterioration, and present serviceability rating (PSR). These models

Table 24.

Specific variables in four climatic regions averaged from COPES database sections.

JPCP:	Climatic region	Dry - Freeze	Wet - Freeze
	Annual Precipitation, cms	40	84
	Freezing Index, degree-days	250	625
	Mean Temperature, ^{o}C	11	11
	Temperature Range, ^{o}C	41	41
	Climatic region	Dry - Nonfreeze	Wet - Nonfreeze
	Annual Precipitation, cms	40	120
	Freezing Index, degree-days	0	0
	Mean Temperature, ^{o}C	19	19
	Temperature Range, ^{o}C	25	30
JRCP:	Climatic region	Dry - Freeze	Wet - Freeze
	Annual Precipitation, cms	55	78
	Freezing Index, degree-days	1125	1125
	Mean Temperature, ^{o}C	8	8
	Temperature Range, ^{o}C	45	43
	Climatic region	Dry - Nonfreeze	Wet - Nonfreeze
	Annual Precipitation, cms	-*	120
	Freezing Index, degree-days	-*	0
	Mean Temperature, ^{o}C	-*	17
	Temperature Range, ^{o}C	-*	34

* No sections available.

Table 25.

Design input parameters for AASHTO performance equation for the COPES four climatic regions.

Parameter	JPCP	JRCP
Reliability level, %	50/80/90	50/80/90
Design period, years	20	20
Traffic, million 18-kip ESAL	15	15
* Subgrade soil	fine/coarse	fine/coarse
** Subbase type	4" CTB	6" granular
k-value @ top of subbase, pci	300/590	200/420
Initial serviceability	4.5	4.5
Terminal serviceability	2.5	2.5
*** Modulus of rupture, psi	650	650
Concrete E value, psi	4,000,000	4,000,000
Joint spacing, ft	15	27/40
Dowels at joints	yes/no	yes
J factor	3.2/4.1	3.2

* Subgrade M_R = 3,000 psi for fine-grained soil and 7,000 psi for coarse-grained soil

** Subbase E = 1,000,000 psi for CTB and 30,000 psi for granular

*** Third-point loading, at 28 days

Table 26.

Climatic design inputs for AASHTO guide
for the four climatic regions from COPES.

JPCP	Climatic zones	Dry - Freeze				Wet - Freeze			
	Subgrade soil	fine		coarse		fine		coarse	
	Dowel bars	no	yes	no	yes	no	yes	no	yes
	C_d value	.95	.95	1.13	1.13	.85	.85	1.05	
1.05									
	LS factor	.5	.5	.25	.25	1.0	1.0	.5	.5
	* Corrected k-value	175	175	400	400	100	100	290	290
	Climatic zones	Dry - Nonfreeze				Wet - Nonfreeze			
	Subgrade soil	fine		coarse		fine		coarse	
	Dowel bars	no	yes	no	yes	no	yes	no	yes
	C_d value	.95	.95	1.13	1.13	.8	.8	1.0	
1.0									
	LS factor	.5	.5	.25	.25	1.0	1.0	.5	.5
	Corrected k-value	175	175	400	400	100	100	290	290

JRCP	Climatic zones	Dry - Freeze		Wet - Freeze	
	Subgrade soil	fine	coarse	fine	coarse
	Dowel bars	yes	yes	yes	yes
	C_d value	.95	1.13	.85	1.05
	LS factor	.5	.25	1.0	.5
	Corrected k-value	120	300	70	230
	Climatic zones	Dry - Nonfreeze		Wet - Nonfreeze	
	Subgrade soil	fine	coarse	fine	coarse
	Dowel bars	yes	yes	yes	yes
	C_d value	-	-	.8	1.0
	LS factor	-	-	1.0	.5
	Corrected k-value	-	-	70	230

* k-value in pci

actually represent the database mathematically. They provide "average" projections with about one half the actual showing worse performance and the other half better performance. The critical level for each kind of distress mentioned above which normally generates the need of rehabilitation activities are given as follows:

Distress	JPCP	JRCP
Pumping	2 (med severity)	2 (med severity)
Faulting	0.15 in	0.3 in
Cracking	818 ft/mi (all severities)	1,500 ft/mi (med. and high severity)
Joint deterioration	140 joints/mi	53 joints/mi
PSI	3.0	3.0

The predictions for both JPCP and JRCP over three levels of reliability are shown in each of the design cells from table 27 to table 32.

1. JPCP: For the 50 percent reliability level, the required slab thickness for JPCP with dowels for each kind of subgrade soil is about 1.3 in (3.3 cm) less than that without dowels. Pavements with dowels also decrease the joint faulting substantially. The required slab thickness for both kinds of load transfer varies from 1.4 to 1.8 in between the fine-grained and coarse-grained subgrade soils. In general, the results show that the AASHTO Guide designs provide adequate structural designs for JPCP except in the wet-freeze region. The medium to high severity pumping and cracking distresses for JPCP with dowels in that climate shows the structural designs are not adequate. The results also show that the higher the level of reliability (or safety factor) the better the expected performance of the design according to the predictive models from COPES. Figure 12 shows the pumping climatic regions with varying reliability levels. Figure 13 illstrates how present serviceability index changes over the climatic regions with varying levels of reliability. It should be noted that the predictions from the models represent the average condition of the expected performance for the section. Thus, the adjustment factors provided in the 1985 Guide can be used to provide adequate JPCP designs for most conditions.

2. JRCP: All the JRCPs were designed with dowels. In the 50 percent reliability level, the required slab thickness for both shorter and longer joint spacings varies from 1.3 to 1.7 in (3.3 to 4.3 cm) between the fine-grained and coarse-grained subgrade soils. The results show that the AASHTO Guide did not provide adequate designs for JRCP, especially with coarse-grained subgrade soil. The pumping and cracking distresses show high severity in the freeze climate as well as medium severity in wet-nonfreeze region (See figures 14 and 15) and the present serviceability indices fall below the criteria, 3.0. The higher the level of reliability the better the expected performance of the design. This can be used to improve the level of performance. Some components of the pavement, however, showed serious failure and did not improve with the higher level of reliability. This included joint deterioration with 40-ft (12 m) or more joint spacing. For example, the 27-ft (8.2 m) joint spacing gives much better performance. The AASHTO Design Guide does not provide adequate, coherent guidance on the joint design.

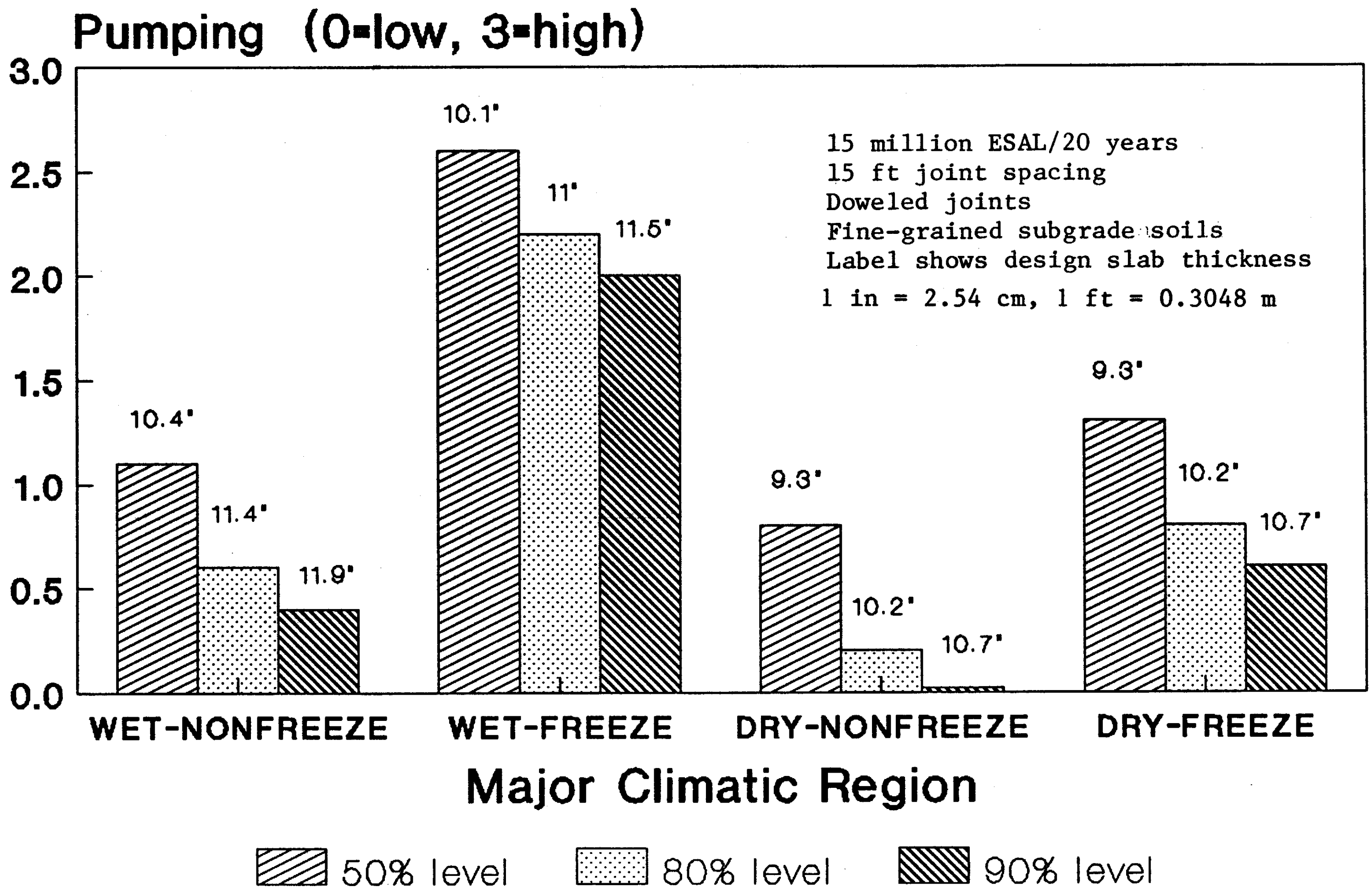


Figure 12. Pumping vs. climatic regions with varying reliability levels for AASHTO JPCP designs.

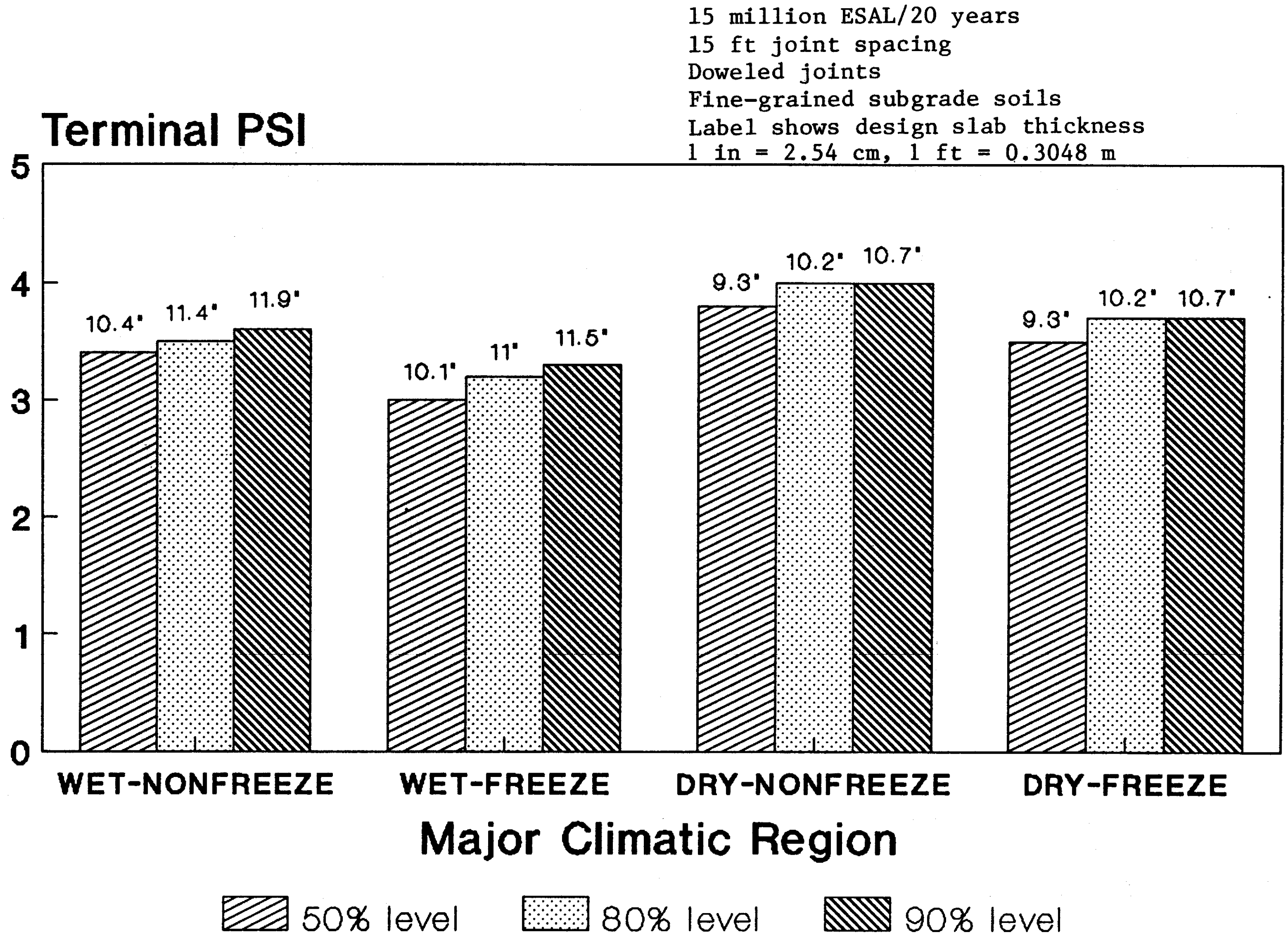
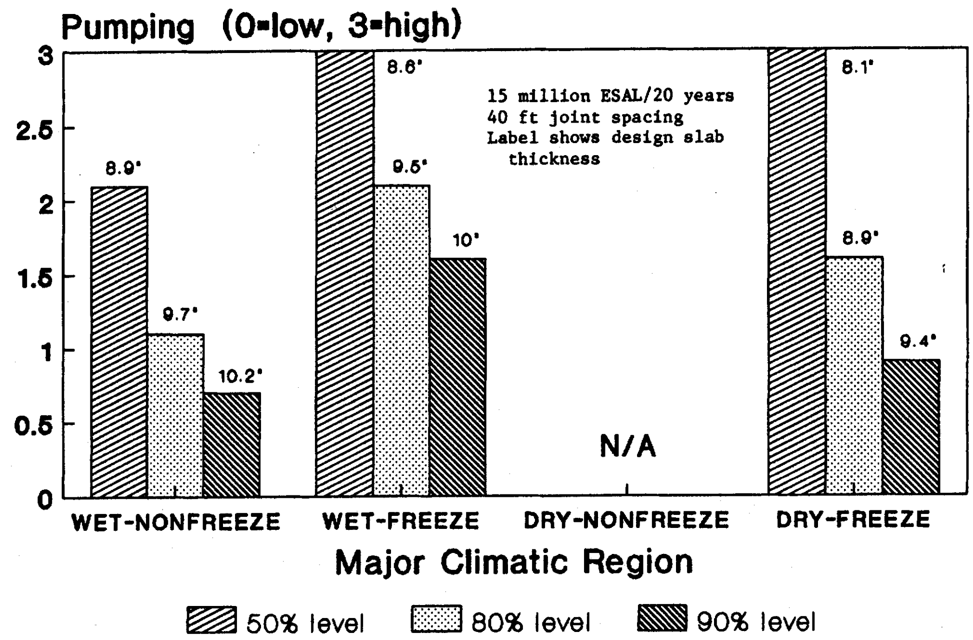
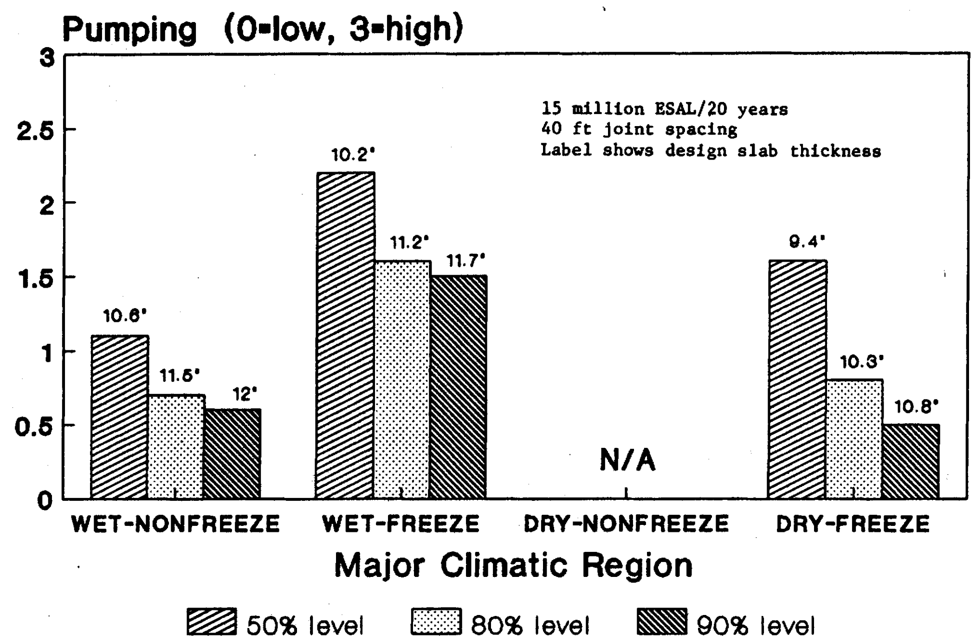


Figure 13. Terminal PSI vs. climatic regions with varying reliability levels for AASHTO JPCP designs.

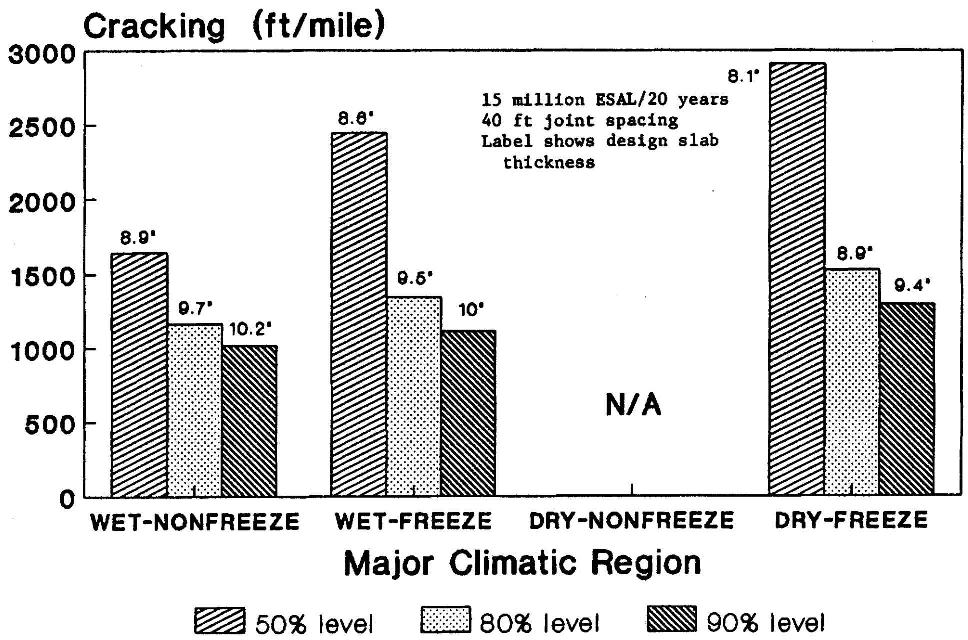


(a) coarse-grained subgrade soil

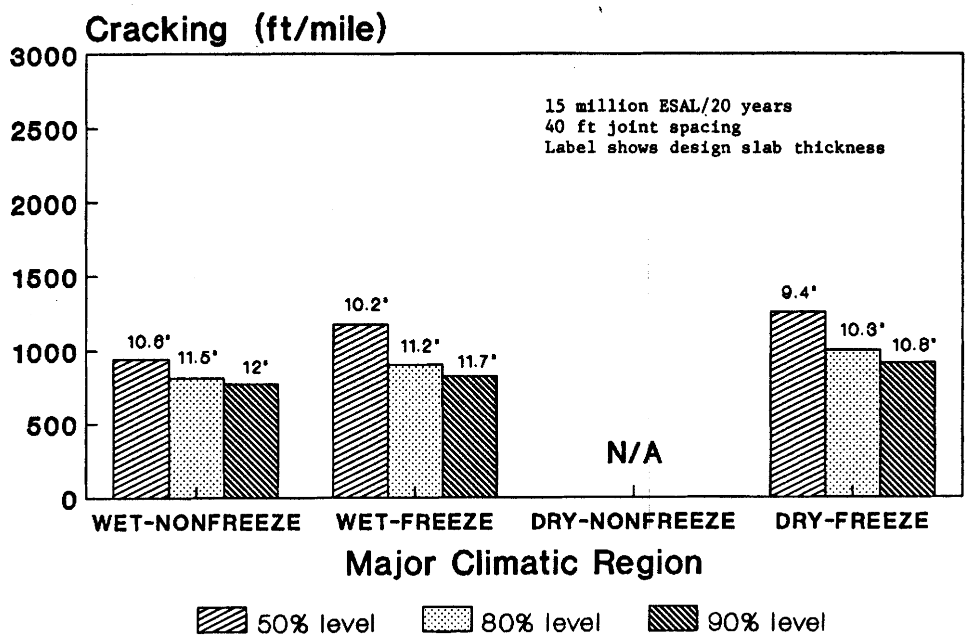


(b) fine-grained subgrade soil

Figure 14. Pumping vs. climatic regions with varying reliability levels for AASHTO JRCP designs.



(a) coarse-grained subgrade soil



(b) fine-grained subgrade soil

Figure 15. Cracking vs. climatic region with varying reliability levels for AASHTO JRCP designs.

Table 27.

Predictions for JPCP designs using AASHTO guide for major climatic zones - 50% reliability level.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 4 in CTB
Joint spacing: 15 ft
Level of reliability: 50%

Climatic zones	Dry-freeze				Wet-freeze			
Subgrade soil	fine		coarse		fine		coarse	
Dowel diameter, in	0	1.125	0	1.00	0	1.25	0	1.00
Slab thickness, in	10.6	9.3	9.2	7.9	11.5	10.1	9.8	8.5
Pumping	.6	1.3	.6	1.7	2	2.6	1.9	2.8
Faulting, in	.1	.05	.11	.07	.12	.05	.13	.09
Cracking, ft/mile	151	368	249	713	115	315	251	853
Joint deter., jts/mile	12	12	12	12	12	12	12	12
PSI	3.7	3.5	3.6	3.4	3.3	3.0	3.1	2.8

Climatic zones	Dry - Nonfreeze				Wet - Nonfreeze			
Subgrade soil	fine		coarse		fine		coarse	
Dowel diameter, in	0	1.125	0	1	0	1.25	0	1.125
Slab thickness, in	10.6	9.3	9.2	7.9	11.8	10.4	10	8.7
Pumping	0	.8	0	1.1	.5	1.1	.4	1.3
Faulting, in	.06	0	.07	.03	.06	0	.07	.01
Cracking, ft/mile	51	117	72	144	41	80	70	134
Joint deter., jts/mile	12	12	12	12	12	12	12	12
PSI	4.0	3.8	3.9	3.7	3.6	3.4	3.4	3.2

1 in = 2.54 cm, 1 ft = 0.3048m, 1 mi = 1.6 km

Table 28.

Predictions for JPCP designs using AASHTO guide for major climatic zones - 80% reliability level.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 4 in CTB
Joint spacing: 15 ft
Level of reliability: 80%

Climatic zones	Dry-freeze				Wet-freeze			
Subgrade soil	fine		coarse		fine		coarse	
Dowel diameter, in	0	1.25	0	1.125	0	1.375	0	1.125
Slab thickness, in	11.6	10.2	10.1	8.7	12.5	11	10.7	9.3
Pumping	.2	.8	0	.9	1.7	2.2	1.5	2.2
Faulting, in	.1	.02	.1	.05	.11	.02	.12	.07
Cracking, ft/mi	89	193	143	358	66	159	131	384
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	3.9	3.7	3.7	3.5	3.4	3.2	3.2	3.0

Climatic zones	Dry - Nonfreeze				Wet - Nonfreeze			
Subgrade soil	fine		coarse		fine		coarse	
Dowel diameter, in	0	1.25	0	1.125	0	1.375	0	1.25
Slab thickness, in	11.6	10.2	10.1	8.7	12.9	11.4	11	9.6
Pumping	0	.2	0	.4	.1	.6	0	.7
Faulting, in	.06	0	.06	.01	.05	0	.06	0
Cracking, ft/mi	31	63	46	93	26	49	44	85
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	4.2	4.0	4.0	3.8	3.8	3.5	3.6	3.3

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 mi = 1.6 km

Table 29.

Predictions for JPCP designs using AASHTO guide for major climatic zones - 90% reliability level

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 4 in CTB
Joint spacing: 15 ft
Level of reliability: 90%

Climatic zones	Dry-freeze				Wet-freeze			
Subgrade soil	fine		coarse		fine		coarse	
Dowel diameter, in	0	1.375	0	1.125	0	1.375	0	1.25
Slab thickness, in	12.1	10.7	10.6	9.2	13.1	11.5	11.3	9.8
Pumping	0	.6	0	.6	1.5	2.0	1.2	1.9
Faulting, in	.09	0	.1	.05	.11	.02	.12	.05
Cracking, ft/mi	70	143	109	249	49	115	91	251
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	4.0	3.7	3.8	3.6	3.5	3.3	3.3	3.1

Climatic zones	Dry - Nonfreeze				Wet - Nonfreeze			
Subgrade soil	fine		coarse		fine		coarse	
Dowel diameter, in	0	1.375	0	1.125	0	1.5	0	1.25
Slab thickness, in	12.1	10.7	10.6	9.2	13.5	11.9	11.6	10.1
Pumping	0	0	0	0	0	.4	0	.4
Faulting, in	.05	0	.06	0	.05	0	.06	0
Cracking, ft/mi	25	48	37	72	20	39	34	67
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	4.3	4.0	4.1	3.9	3.9	3.6	3.7	3.4

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 mi = 1.6 km

Table 30.

Predictions for JRCP designs using AASHTO guide for major climatic zones - 50% reliability level.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 6 in granular

Level of reliability: 50%

Climatic zones	Dry-freeze				Wet-freeze			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	9.4		8.1		10.2		8.6	
Dowel diameter, in	1.125		1		1.25		1.125	
Joint spacing, ft	27	40	27	40	27	40	27	40
Area of steel, in^2 ft	.047	.069	.04	.06	.051	.075	.043	.064
Pumping	1.6	1.6	3	3	2.2	2.2	3	3
Faulting, in	.07	.12	.16	.17	.06	.11	.09	.14
Cracking, ft/mi	1322	1300	2932	2904	1166	1169	2437	2432
Joint deter., jts/mi	0	61	0	61	0	61	0	61
PSI	3.1	3.0	2.7	2.6	3.3	2.9	3.2	2.8

Climatic zones	Dry - Nonfreeze	Wet - Nonfreeze			
Subgrade soil	-	fine		coarse	
Slab thickness, in	-	10.6		8.9	
Dowel diameter, in	-	1.375		1.125	
Joint spacing, ft	-	27	40	27	40
Area of steel, in^2/ft	-	.053	.078	.044	.066
Pumping	-	1.1	1.1	2.1	2.1
Faulting, in	-	0	.05	.03	.08
Cracking, ft/mi	-	942	944	1636	1639
Joint deter., jts/mi	-	0	35	0	35
PSI	-	3.3	3.2	2.9	2.9

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 mi = 1.6 km

Table 31.

Predictions for JRCP designs using AASHTO guide for major climatic zones - 80% reliability level.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 6 in granular

Level of reliability: 80%

Climatic zones	Dry-freeze				Wet-freeze			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10.3		8.9		11.2		9.5	
Dowel diameter, in	1.25		1.125		1.375		1.125	
Joint spacing, ft	27	40	27	40	27	40	27	40
Area of steel, in^2/ft	.051	.076	.044	.066	.056	.083	.047	.07
Pumping	.8	.8	1.6	1.6	1.6	1.6	2.1	2.1
Faulting, in	.02	.07	.03	.07	.02	.07	.03	.08
Cracking, ft/mi	1001	999	1547	1536	887	887	1346	1343
Joint deter., jts/mi	0	61	0	61	0	61	0	61
PSI	3.5	3.4	3.1	3.0	3.4	3.3	3.2	3.1

Climatic zones	Dry - Nonfreeze	Wet - Nonfreeze			
Subgrade soil	-	fine		coarse	
Slab thickness, in	-	11.5		9.7	
Dowel diameter, in	-	1.375		1.25	
Joint spacing, ft	-	27	40	27	40
Area of steel, in^2/ft	-	.057	.085	.048	.072
Pumping	-	.7	.7	1.1	1.1
Faulting, in	-	0	.04	0	.01
Cracking, ft/mi	-	820	818	1163	1156
Joint deter., jts/mi	-	0	35	0	35
PSI	-	3.5	3.4	3.2	3.1

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 mi - 1.6 km

Table 32.

Predictions for JRCP designs using AASHTO guide for major climatic zones - 90% reliability level.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 6 in granular

Level of reliability: 90%

Climatic zones	Dry-freeze				Wet-freeze			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10.8		9.4		11.7		10	
Dowel diameter, in	1.375		1.125		1.5		1.25	
Joint spacing, ft	27	40	27	40	27	40	27	40
Area of steel, in^2/ft	.054	.08	.047	.069	.058	.086	.05	.074
Pumping	.5	.5	.9	.9	1.5	1.5	1.6	1.6
Faulting, in	0	.05	0	.05	0	.05	0	.03
Cracking, ft/mi	906	906	1265	1271	822	822	1099	1100
Joint deter., jts/mi	0	61	0	61	0	61	0	61
PSI	3.6	3.5	3.4	3.3	3.5	3.4	3.3	3.2

Climatic zones	Dry - Nonfreeze	Wet - Nonfreeze			
Subgrade soil	-	fine		coarse	
Slab thickness, in	-	12		10.2	
Dowel diameter, in	-	1.5		1.25	
Joint spacing, ft	-	27	40	27	40
Area of steel, in^2/ft	-	.06	.089	.051	.075
Pumping	-	.6	.6	.7	.7
Faulting, in	-	0	.03	0	0
Cracking, ft/mi	-	775	775	1017	1020
Joint deter., jts/mi	-	0	35	0	35
PSI	-	3.6	3.5	3.4	3.3

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 mi = 1.6 km

3.3 Zero-Maintenance Procedure for JPCP

The Zero-Maintenance Design Procedure for JPCP was developed in 1977.[46,47] The term "zero-maintenance" refers to low structural maintenance, which includes full-depth repair, slab replacement, and overlay.

In 1982 a new premium pavement design nomograph based on a revised serviceability/performance model for JPCP design was developed for FHWA.[48] A review of this nomograph is presented in section 3.3.2.

3.3.1 1977 Zero-Maintenance Design Procedure

The Zero-Maintenance design procedure is based on field performance and theoretical studies. The procedure consists of the determination of material properties and the design of slab thickness, subbase, joints, shoulder and subdrainage. The structural design is based on both a slab fatigue analysis and a serviceability/performance analysis. Some minor changes have been made on the Zero-Maintenance Design Procedure since its original development in 1977. The evaluation of this section was based on the original edition.

<u>Conceptual Evaluation</u>.

The fundamental basis of the Zero-Maintenance design procedure for structural design of JPCP is based on both the pavement serviceability/performance analysis and the PCC slab fatigue damage analysis.

1. Serviceability/performance analysis: The serviceability/performance criterion for structural design is to consider the types of distress that affect rideability such as joint faulting, cracking, joint spalling, and differential settlement of slabs. A serviceability/performance equation was developed using long term field performance data and stepwise multiple regression techniques. The field performance data included 25 original sections from the AASHO Road Test that were in service as part of I-80 in Illinois until 1974 plus 10 additional projects ranging in age from 9 to 25 years from nationwide locations. The basic equation is given as follows:

$$W'_{18} = [\ r \ln (3/y - 1) + B\ ]\ 10^6 \qquad (6)$$

where W'_{18} = total equivalent 18-kip single axle loads to reduce the serviceability index from P1 to P2

$$B = -\ 50.08826 - 3.77485H + 30.64386\ H^{0.5}$$

$$r = -6.69703 + 0.13879\ H^2$$

$$y = P2 + \frac{3.0}{e^{-B/r} + 1} - P1$$

P2 = terminal serviceability index
P1 = initial serviceability index
H = PCC slab thickness, in

Equation 6 was then extended using the Westergarrd edge stress equation to include other variables such as the k-value and PCC modulus of rupture. The edge loading point was considered critical due to the results of fatigue analysis and field observations of distress. The extension process was similar to the way in which the Spangler corner equation was incorporated into the original AASHO Road Test model as described in the AASHTO Interim Guide.[36] The final equation became:

$$\log_{10} W_{18} = \log W'_{18} + (3.892 - 0.706P2) * \log [(F28/690) * A / B] \quad (7)$$

where

$$A = 4 \text{ Log } [(8.789\ H^{0.75} / M] + 0.359$$

$$B = 4 \text{ Log } [(Z^{0.25} (0.540\ H^{0.75}) / M] + 0.359$$

$$M = [1.6\ a^2 + H^2] - 0.675\ H$$

a = radius of applied edge load, inches

F28 = modulus of rupture used in design (28 day, 3rd point loading adjusted for variability) = FF - C(Fcv/100)FF
FF = mean modulus of rupture at 28 days, 3rd point loading, psi
Fcv = coefficient of variation of modulus of rupture, percent
C = 1.03, a constant representing a confidence level of 85 percent
Z = E/K
E = modulus of elasticity of PCC, psi
k = modulus of foundation support on top of subbase, pci

A "climatic regional factor" is used in the procedure to adjust for different pavement performance in different climatic regions. It is defined as follows:

$$RF = W_{18} \text{ (computed)} / W_{18} \text{ (actual)} \quad (8)$$

where RF = climatic regional factor

W_{18} (computed) = total computed number of equivalent 18-kip single-axle loads to reduce serviceability index from an initial value to a terminal value determined from performance equation 7

W_{18} (actual) = total accumulated number of equivalent 18-kip single axle loads to pass over pavement, determinted from traffic data

The mean RF calculated from the heavily trafficked JPCP projects from nationwide locations and the AASHO sections is shown below.

Region	Design RF
Wet/Freeze	1.0
Dry/Freeze	1.0
Wet/Non-freeze	0.9
Dry/Non-freeze	0.6

This shows that JPCP located in dry-nonfreeze locations have a much greater traffic life than in wet-freeze areas. Those four climatic regions were chosen based upon precipitation, potential evapotranspiration, and frost heave and freeze thaw damage as defined in Reference 46. A nomograph is provided in the procedure to solve equation 7.

2. PCC fatigue analysis: The PCC fatigue criterion in the procedure is to estimate the amount of slab cracking. Traffic loadings, slab curling, joint spacing, and foundation support are directly considered in the analysis. The longitudinal edge of the slab was considered to be the critical point of fatigue crack initiation. The fatigue damage is computed for a given month, both day and night, and summed monthly over the entire design period using the Miner's accumulative damage hypothesis as follows:

$$DAMAGE = \sum_{k=1}^{p} \sum_{j=1}^{2} \sum_{i=1}^{m} n_{ijk} / N_{ijk} \qquad (9)$$

where
- DAMAGE = total accumulated fatigue damage over the design period occurring at the slab edge
- n_{ijk} = number of applied axle load applications of i^{th} magnitude during the day or night (j) for the k^{th} month
- N_{ijk} = number of allowable axle load applications of i^{th} magnitude over day or night for the k^{th} month determined from PCC fatigue curve
- i = a counter for magnitude of axle load, both single and tandem axle
- j = a counter for day and night (j = 1 day and j = 2 night)
- k = a counter for months over the design period
- m = total number of single-and tandem-axle load group
- p = total number of months in the design period

The n_{ijk} in the fatigue damage equation 9 is computed using the traffic data for the month under consideration using the following expression:

$$n_{ijk} = (ADT_m)(T/100)(DD/100)(LD/100)(A)(30)(P/100)$$
$$(C/100)(DN/100)(TF/100)(CON/100) \qquad (10)$$

where
- ADT_m = average daily traffic at the end of the specific month under consideration
- T = percent truck of ADT
- DD = percent traffic on direction of design lane
- LD = lane distribution factor, percent trucks in the design lane in one direction
- A = mean number of axles per truck
- P = percent axles in i^{th} load group
- C = percent of total axles in the lane that are within 6 inches of edge
- DN = percent of trucks during day or night
- TF = factor to either increase or decrease truck volume for the specific month
- CON = one for single axles, two for tandem axles

The N_{ijk} is computed from PCC fatigue considerations. The combined stress occurring at the slab edge for a given axle load is computed considering both traffic load and slab curling for the given month for either day or night conditions. The stress is computed using regression models developed from a finite element program and are given in reference 46. The total stress at the bottom of the slab edge with the load located at the edge is computed as follows:

$$STRT = STRL + (R)STRC \qquad (11)$$

where STRT = total resultant stress in the longitudinal direction at the bottom of the PCC slab edge when the wheel load is located at the slab edge (load is single-axle or tandem-axle)
STRL = stress at bottom of PCC slab edge when load is located at slab edge (no thermal curling stress)
STRC = stress at bottom of PCC slab edge caused by curling of slab due to thermal gradient (no traffic load)
R = adjustment factor for STRC so that it can be combined with STRL to give correct STRT combined stress

The flexural fatigue life of PCC used in the design procedure is given as follows:

$$\log_{10}N = 16.61 - 17.61\ STRT / F \qquad (12)$$

where N = number of load applications to flexural failure of the PCC
STRT = total stress at bottom of PCC slab, psi
F = modulus of rupture of PCC, psi

The F in equation 12 is determined monthly over the design period and adjusted for concrete variability and time-modulus of rupture relationship. The above fatigue life expression represents a confidence level in determining mean fatigue life of approximately one decade of load applications. Detailed description for the equations and guidelines on obtaining all required inputs are provided in reference 46. A computer program named JCP-1 is available in both mainframe or IBM PC compatible version with documentation for the user to run the trial fatigue analysis.

The computed DAMAGE value from equation 9 was correlated with measured cracking from the field projects as shown in figure 16. The maximum allowable fatigue consumption (or DAMAGE) as accumulated monthly over the entire design analysis period at the slab edge, midway between joints, can be varied depending on allowable cracking. For design of low maintenance pavements, a value of 0.0001 is recommended.

The minimum design slab thickness is determined by meeting the limiting design criteria of <u>both serviceability and fatigue</u>. The essential features of the Zero-Maintenance procedure are summarized as follows:

1. Traffic - The estimation of total accumulated 18-kip ESAL in the design lane over the analysis period is computed using the following expression:

$$W_{18} = (ADT)(T/100)(DD/100)(LD/100)(TY)(365)(A)(PE/100) \qquad (13)$$

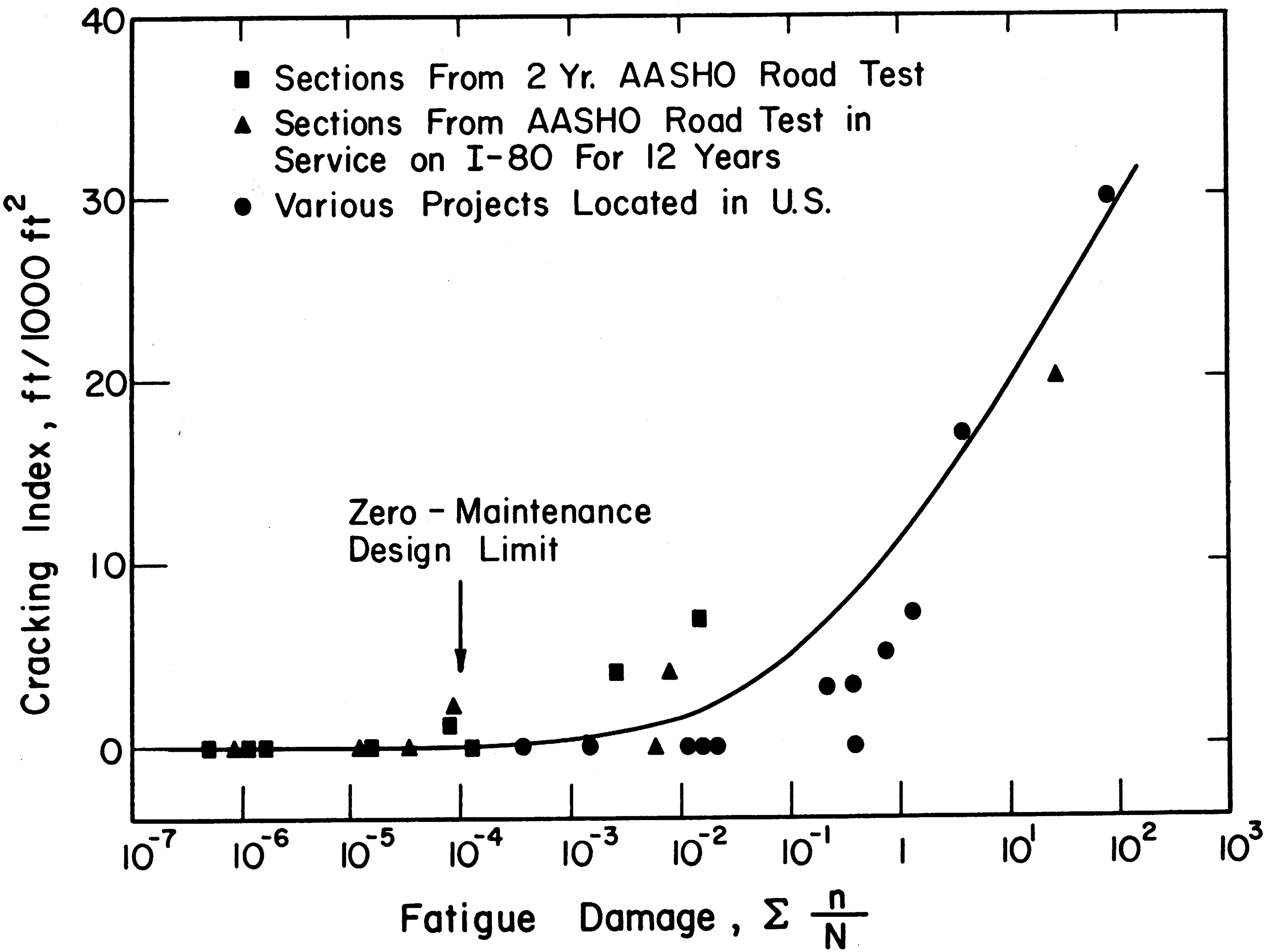


Figure 16. Effect of fatigue "damage" on cracking of plain jointed concrete pavement.

where W_{18} = total accumulated 18-kip ESAL from the time the pavement was opened to traffic to end of the analysis period.

TY = analysis period, years

ADT = average daily traffic (two directions) over period TY

T, DD, LD, A = as defined for equation 10

PE = sum of the product of the percent of axles (both single and tandem in each load group) and the corresponding load equivalency factor (for terminal serviceability of 3.0)

2. Mean truck lateral distance - The mean truck lateral distance is important in determining the number of "edge" loads for the PCC slab. Previous studies indicates that the mean distance varies from 12 to 21 in (30.5 to 53.3 cm) from the slab edge when there is a paved shoulder and no lateral obstructions. The lateral distribution is approximately normally distributed with a standard deviation of 10 in (25.4 cm).

3. Variation of PCC modulus of rupture - The design PCC modulus of rupture adjusted for concrete variability that is used in design is obtained from the following expression:

$$F_{28} = FF - C * \frac{Fcv}{100} * FF \qquad (14)$$

where FF = mean modulus of rupture of the PCC at 28 days, 3rd point loading, psi

Fcv = coefficient of variation of modulus of rupture, percent

C = 1.03, a constant representing a confidence level of 85 percent%

A time-modulus of rupture relationship is used in the procedure to obtain the PCC modulus of rupture at any time. Therefore, the PCC modulus of rupture used to determine fatigue damage at a given time is given by the relationship:

$$F = F_A * F_{28} \qquad (15)$$

where $F_A = 1.22 + 0.17 \log T_2 - 0.05 (\log T_2)^2$

and T_2 = time since the pavement slab was constructed, years

4. Foundation support - Procedures are provided to estimate the subgrade k-value as a function of soil type and degree of saturation. The k-value must be input for each month of a typical year for use in the fatigue analysis. Three types of subbases, granular, cement-treated and asphalt-treated, are suggested in the procedure. The k-value at the top of subbase is estimated from charts which were developed using elastic layer theory.

5. Joint design - Included in the Zero-Maintenance procedure are recommendations for joint type, spacing, shape, sealant, load transfer device, randomized or skewed transverse joints, and stabilized subbases. Three types of joints, contraction, construction and expansion, are recommended in the procedure. Contraction joints are recommended for regular transverse and longitudinal joints. The design procedure directly considers joint spacing based on slab curling, however, a maximum limit for joint spacing is 20 ft (6 m). It is highly recommended that spacing be

limited to about 15-17 ft (4.6 - 5.2 m) if dowel bars are used and 12-15 ft (3.7-4.6 m) or less if no dowel bars are used. The use of dowel bars are strongly recommended in all wet climates and heavy traffic routes. Sizes and spacing of dowel bars are provided along with recommendations for the use of corrosion-proof bars in wet/freeze climates. The use of stabilized subbases (monthly k-value preferably greater than 100 pci (2.76 kg/cm^3)), relatively thick PCC slabs, full depth PCC or asphalt concrete shoulders and subsurface drainage, and short joint spacing are also recommended in addition to dowel bars. Specific dimensions of the joint width and depth for field molded and preformed sealants are provided.

6. Shoulders and subsurface drainage - PCC and asphalt concrete shoulders were recommended in the procedure. General guidelines were provided for the design of those components.

Some specific limitations of the design procedure are summarized as follows:

1. Stress analysis - Concrete slabs are subjected to warping and curling stresses besides the traffic loading. The Zero-Maintenance procedure does consider thermal curling but does not take into consideration moisture gradient warping stresses.

2. Erodability of subbase - The procedure provides for loss of support along the edge of the slab, but this must be input by the designer using recommendations given in the procedure. No validation is provided for these recommendations. The effect of different types of stabilized subbase on the erodability are not considered.

3. Climatic regional factor - The climatic regional factor recommended in the procedure was developed empirically. This climatic regional factor provides improvement in prediction but is not sufficiently verified.

4. Joint design - The guidelines and recommendations for joint design in the Zero-Maintenance procedure are based on previous experience. No analytical design procedure is available for the user to design other joint components for the specific pavement conditions.

5. Variability - The procedure requires materials of high quality and construction of good quality control. The variations in construction quality and material engineering properties may have a significant effect on reducing the maintenance-free life of the pavements. The procedure considers the variation of the PCC modulus of rupture. No provision is made in the procedure for nonhomogenity and variability in other engineering properties of the concrete, subbase and subgrade materials as well as the traffic estimation.

6. Validation Of Predictive Models - Although the models improve the performance prediction over that of the AASHO Road Test, the extent of data upon which the serviceability/performance model or the fatigue cracking/DAMAGE relationship were developed is very limited.

Analytical Evaluation.

The analytical evaluation consists of two parts: the comparison of predicted vs. actual ESALs, and Specific Design Evaluation.

Predicted vs. actual ESALs. The actual number of ESALs was compared to the predicted ESALs due to the measured loss in present serviceability index using Zero-Maintenance performance equation (i.e. equation 6). This was done for each section of pavement of JPCP in the COPES database. The total 18-kip ESAL is predicted based on the heaviest traveled lane from the time the pavement was open to the traffic until the date of the survey when the serviceability index was determined. The data retrieval and computations were completed utilizing the SPSS statistical package.[43]

The pavement sections in the COPES database were divided into four broad climatic zones, e.g., wet-freeze, wet-nonfreeze, dry-freeze and dry-nonfreeze, as defined in section 3.2.1, and the results were compared by zone. The predicted 18-kip (80 kN) ESALs were adjusted by the recommended climatic regional factors. The plots of predicted vs. actual ESALs for JPCP for each climatic region are shown in figure 17. A summary of the results of the predicted vs. actual ESALs for JPCP is given in table 33. A comparison of the results is summarized as follows:

The results are somewhat dependent on climate. The sections in the dry regions performed generally as the Zero-Maintenance performance equation predicted (approximately 45 to 59 percent of pavement sections acceptable). The average predicted ESALs to existing serviceability index is 0.095 and 0.208 million less than the actual ESALs in the dry-nonfreeze and dry-freeze region, respectively. Those pavement sections in the wet climates performed generally worse than the performance equation predicted with the range from only 18 to 29 percent of sections acceptable. The average predicted ESALs to existing serviceability index is 2.827 and 2.026 million higher than the actual ESALs in the wet-nonfreeze and wet-freeze region, respectively.

The results in table 33 show that the JPCP sections did not perform as well as the Zero-Maintenance performance equation predicted in wet climatic regions. Compared to the results for the original AASHTO model in section 3.2, this model, which is based on longer term data from different climates, predicted performance better overall, which would be expected.

Specific Design Evaluation. A number of design situations were developed using the Zero-Maintenance design procedure. The axle-load distribution used for the design is shown in table 34. The design factors varied included the fine- and coarse-grained subgrade soils, with and without dowels, 15-ft (4.6 m) joint spacing, and 4 in (10 cm) cement-treated subbase. Table 35 shows the specific inputs for JPCP designs. Two sets of the monthly k-value data were used as inputs for each subgrade soil to characterize the freeze and nonfreeze climates. The design slab thickness was determined through a trial analysis using the computer program JCP-1. These designs then were evaluated using the NCHRP Project 1-19 COPES "PREDICT" program to predict their mean distresses and performance. The predictions for JPCP for each of the major climatic zones are shown in table 36.

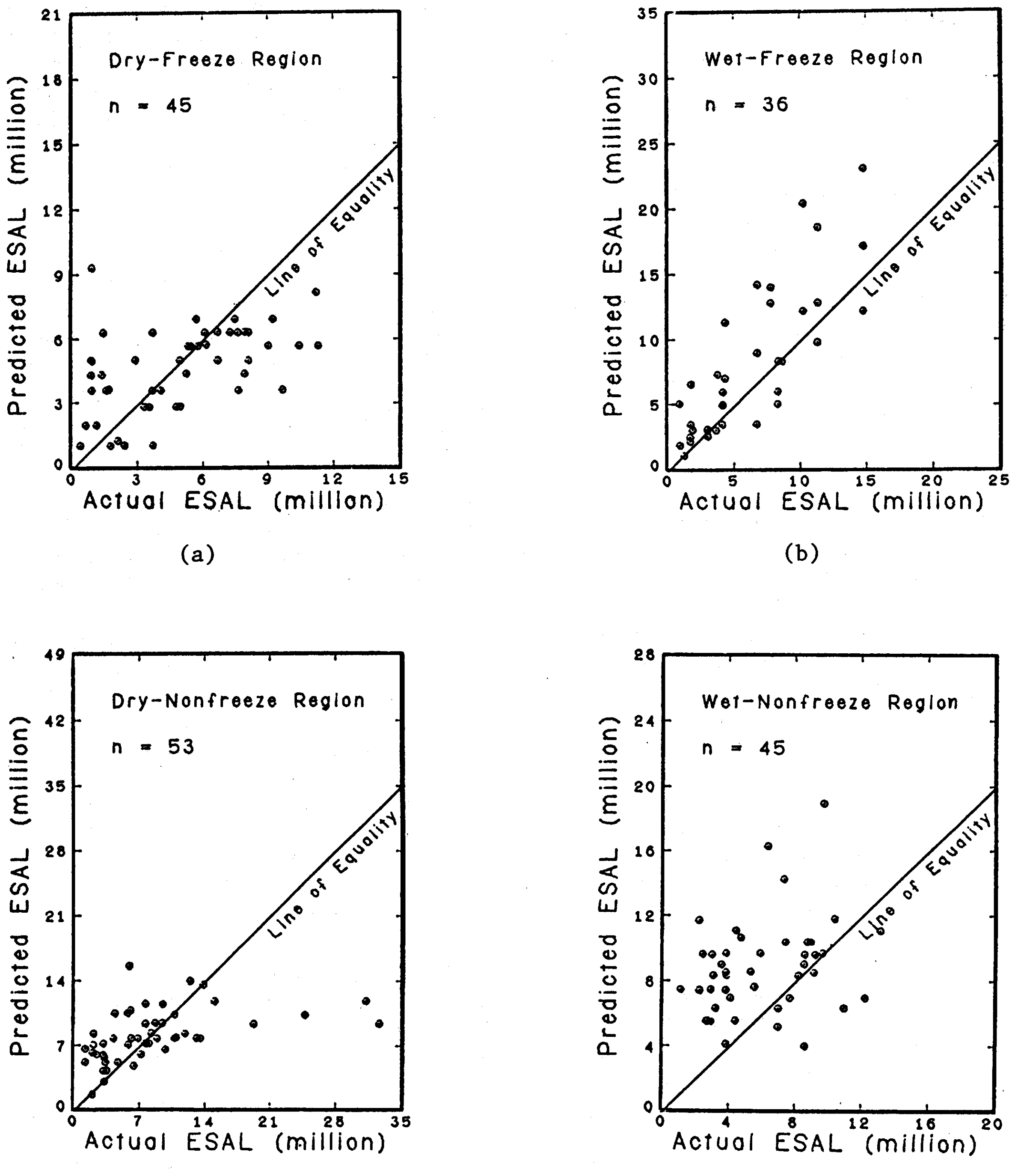


Figure 17. Predicted ESAL vs Actual ESAL for JPCP.

Table 33.

Summary of results for 1977 Zero-Maintenance procedure performance equation for JPCP.

Climatic Region (1)	# of Section (2)	# of Section Acceptable* (3)	Percent Acceptable (4)=(3)/(2)	Mean Difference** (5)	Mean Percent Difference*** (6)
Wet - Nonfreeze	45	8	18%	2.827	91.3%
Wet - Freeze	36	10	29%	2.026	56.7%
Dry - Nonfreeze	53	24	45%	-0.095	48.5%
Dry - Freeze	49	29	59%	-0.208	69.4%
overall	183	71	39%	1.010	66.3%

* "Acceptable" means the actual number of 18-kip ESALs is equal or greater than the predicted ESALs, i.e., the pavement section performed as good as or better than the Zero-Maintenance Procedure performance equation predicted, otherwise is "unacceptable".

** Difference = Predicted ESAL - Actual ESAL, in millions

*** Percent Difference = [(Predicted ESAL - Actual ESAL)/Actual ESAL] x 100%

Table 34.

Axle-load distributions.

Axle	Axle load (kips)	Percentage
Single	0-3	7.28
	3-7	16.28
	7-8	7.75
	8-12	15.01
	12-16	4.75
	16-18	1.94
	18-20	1.32
	20-22	1.02
	22-24	0.42
	24-26	0.14
	26-28	0.06
	28-30	0.02
	30-32	0.01
	32-34	0.01
Tandem	0-6	0.37
	6-12	9.76
	12-18	4.36
	18-24	5.68
	24-30	8.92
	30-32	4.90
	32-36	7.70
	36-40	1.88
	40-44	0.27
	44-48	0.09
	48-52	0.04
	52-56	0.02
	Total	100.00

Table 35.

Design inputs for Zero-Maintenance design procedure.

Parameter	JPCP design
Design period, years	20
Initial serviceability	4.5
Terminal serviceability	3.0
Month open to traffic	October
ADT at beginning of design period	25000
ADT at end of design period	35000
Percent trucks of ADT, %	10
Total 18-kip ESAL, million	15
Percent trucks in design lane, %	75
Percent directional distribution, %	50
Mean axles per truck	2.75
Percent trucks during daylight, %	60
Mean truck lateral distance, inches	18
* Subgrade soil	fine-grained/coarse-grained
** Subbase type	4" CTB
Mean k-value @ top of subbase, pci	210/330
Erodability of foundation, inches	12
*** Mean PCC modulus of rupture, psi	650
CV of PCC modulus of rupture, %	12
PCC coeff. of thermal expansion, ins./^{o}F	5×10^{-6}
Concrete E value, psi	4,000,000
Joint spacing, ft	15
Dowels at joint	yes/no

* Subgrade k-value = 100 pci for fine-grained soil and 190 pci for coarse-grained soil

** Subbase E = 1,000,000 psi for CTB

*** Third-point loading, at 28 days

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 psi = 0.0703 kg/cm^2,
1 pci = 0.0277 kg/cm^3, ^{o}F = (^{o}C x 1.8) + 32

Table 36.

Predictions for JPCP designs using Zero-Maintenance design procedure for major climatic zones.

Design Traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 4 in CTB
Joint spacing: 15 ft

Climatic zones	dry-freeze (III-A)				wet-freeze (I-A)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	11.75	11.75	11.7	11.7	11.75	11.75	11.7	11.7
Dowel diameter, in	0	1.25	0	1.25	0	1.25	0	1.25
Pumping	1.2	1.2	0.4	0.4	2.0	2.0	1.2	1.2
Faulting, in	.11	.04	.11	.03	.12	.04	.11	.04
Cracking, ft/mi	122	122	101	101	118	118	97	97
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	3.8	3.8	3.9	3.9	2.8	2.8	2.9	2.9

Climatic zones	Dry - Nonfreeze (III-C)				Wet - Nonfreeze (I-C)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	11.8	11.8	11.7	11.7	11.9	11.9	11.75	11.75
Dowel diameter, in	0	1.25	0	1.25	0	1.25	0	1.25
Pumping	0	0	0	0	.6	.6	0	0
Faulting, in	.06	0	.06	0	.06	0	.06	0
Cracking, ft/mi	43	43	38	38	33	33	30	30
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	4.1	4.1	4.1	4.1	3.4	3.4	3.4	3.4

1 in = 2.54 cm, 1 ft = 0.3048 m, 1 mi = 1.6 km

PCC slab fatigue analysis controls the structural designs. The JPCP with fine-grained subgrade soil gives about the same slab thickness as the coarse-grained subgrade soil. Pumping was predicted to be more severe with fine-grained subgrade soil than with coarse-grained, and especially severe in the wet-freeze region. The presence of dowel bars significantly reduces the amount of faulting in the joints. The same result is shown in the predicted versus actual ESALs evaluation. Dry climates generally give better performance than wet climates. The terminal serviceability index was predicted as high as 4.1 in a dry-nonfreeze region while predicted lower than 3.0 in the wet-freeze zone. This shows that the structural designs generally provide zero-maintenance performance for JPCP except in the wet-freeze region.

Severe pumping in wet-freeze climates especially with fine-grained subgrade soils is the major possible distress of the JPCP designs. Joint deterioration was not substantial in any climate. The procedure does not provide analytical procedures to predict the erodability of the subbase or joint design for each specific project situation.

3.3.2 1982 Zero-Maintenance Design Nomograph

The 1982 new serviceability/performance design nomograph [48] was developed for maintenance-free performance for premium JPCP.

Conceptual Evaluation.

The 1982 serviceability/performance design nomographs were developed using additional data from 76 pavement sections located in 11 states. Mechanistic variables were introduced by using Westergaard's edge stress equation in the regression analysis. Climatic variables such as freezing index and average annual precipitation were also included in the regression analysis. The Westergaard's edge stress is computed as follows:

$$s_e = \frac{0.572P}{D^2} [4 \log_{10}(l/b) + 0.359] \qquad (16)$$

where $l = [(E D^3) / 12k(1-u^2)]^{0.25}$

$b = [1.6 a^2 + D^2]^{0.5} - 0.675 D$

and s_e = edge stress, psi

l = radius of relative stiffness
P = 9000, total applied load, lbs
D = slab thickness, in
E = slab modulus of elasticity, psi
k = effective modulus of subgrade reaction, pci
u = 0.20, slab Poisson's ratio
a = 7.07 in, radius of a circle equal in area to the load area (17.9 cm)

The procedure provides a safety factor by selecting a design confidence level for concrete strength. This design safety factor determines the allowable concrete working stress by dividing the modulus of rupture. The concrete working stress is computed as follows:

$$f_t = Sc / DSF \tag{17}$$

where

Sc = average modulus of rupture, 28-day, 3rd point loading, psi

DSF = design safety factor

$= 1 / (1 - CV * Z)$

CV = coefficient of variation of modulus of rupture

Z = standardized normal deviate

The safety factor ranges from approximately 1.1 for an 85 percent level to 1.8 for a 99.99 percent level (for CV = 0.12). High confidence level was recommended by the procedure for use with high design traffic volumes.

The final serviceability/performance models which generate the design nomographs are shown as follows:

$$P2 = P1 - 3e^{-(r/N)^B} \tag{18}$$

$$r = 19.7F + 9.4DOW - 48.9 \tag{19}$$

$$N = ESAL + 0.00256\ AGE\ FI \tag{20}$$

$$B = 1/(2.42\ F - 0.004FI - 0.009AP) \tag{21}$$

where

$P2$ = 3.0, terminal serviceability index

$P1$ = 4.5, initial serviceability index

$F = f_t / s_e$

f_t = allowable concrete working stress, psi

s_e = Westergaard's edge stress, psi

DOW = 0, if dowels used
1, if dowels used

$ESAL$ = total 18-kip ESAL applications, million

AGE = pavement design life, year

FI = annual mean freezing index, degree-day

AP = average annual precipitation, cm

Values of 4.5 and 3.0 were used for the initial and terminal serviceability indices, respectively, in the design nomograph. Required slab thickness is determined by entering the nomograph with the F-ratio and other variables.

The design procedure provides guidelines to determine whether or not the subgrade soil is frost susceptible. Increasing the subbase thickness with nonfrost-susceptible materials is recommended to protect the susceptible subgrade soil from frost penetration. A figure is given in the procedure to estimate the frost penetration below the pavement surface as well as to estimate the additional subbase thickness required for the pavement structure.

The 1982 Zero-Maintenance design nomographs recommend three types of transverse joints: contraction, construction and expansion. A simple procedure is provided to determine the maximum joint spacing for a given slab thickness, PCC modulus of rupture and level of axle-load distribution. After the slab thickness has been determined based on the serviceability/ performance nomographs, the maximum transverse joint spacing can be established by selecting a level of cracking (in terms of cracking index) which is acceptable for the pavement design.

Recommendations are given on the dimensions and spacing of dowel bars. The procedure accepts larger spacing of dowel bars in the center of the lane for traffic lanes other than the truck lane. The use of corrosion-proof dowel bars is recommended in freeze climates where deicing salt will be applied. Guidelines for the joint formation, joint geometry and sealant and tie bars (for longitudinal joints) design are also provided. It is recommended to install longitudinal doweled contraction joints in multiple lanes minimum/maximum (50 ft) construction to prevent longitudinal cracking caused by excessive shrinkage stresses.

Tied PCC shoulders are strongly recommended in the procedure for their low-maintenance performance. General guidelines are given for the design of shoulder slab thickness, tie bars, joints and subbase. Guidelines covering the drainage system configuration are also provided in the procedure.

The specific limitations of the Zero-Maintenance design nomographs are similar to which presented for the 1977 Zero-Maintenance design procedure. They are summarized as follows:

1. Design criteria - The design nomograph was developed based only on the mechanistic-empirical serviceability/performance model. The design nomograph does not take the PCC slab fatigue damage into consideration directly.

2. Erodability of subbase - No direct consideration is given in the nomographs for the erodability of different types of subbase material. However, the design nomographs suggest the use of a lean concrete base with a permeable drainage layer placed beneath the lean concrete base to eliminate the erosion of the base material.

3. Joint design - The design nomograph provides a simple procedure to determine the maximum transverse joint spacing by selecting an acceptable level of cracking index. However, most of the joint design guidelines provided in the nomographs are still based on previous experience. No analytical design procedure is available for the user to design other joint components for the specific pavement condition.

4. Variability - The design nomographs provide a safety factor by considering the variation of the PCC modulus of rupture. No provision is made in the procedure for non-homogeneity and variability in other engineering properties of the concrete, subbase and subgrade materials or in the traffic estimation.

5. Validation of serviceability/performance model - Although the serviceability/performance model was developed from database located in 11 states (thus improving the performance prediction over that of the AASHO Road Test) the extent of data upon which the serviceability/performance model is still very limited.

Analytical Evaluation.

The Specific Design Evaluation approach was used. A number of design situations were developed using the 1982 Zero-Maintenance design nomographs over the four major climatic zones. The design traffic for 20-year design period was 15 million ESALs. The design factors varied included the fine- and coarse-grained subgrade soils, with and without dowels, 15-ft (4.6 m) joint spacing, and 4-in (10 cm) cement-treated subbase. Climatic variables such as the freezing index and the annual precipitation were directly input into the nomographs from table 22.

Table 37 shows the specific inputs for JPCP designs. A 90 percent confidence level was selected for the design. The design traffic was the heavy axle-load distribution (table 34). The design slab thickness was determined using the nomograph. These designs then were evaluated using the NCHRP Project 1-19 COPES "PREDICT" program to predict their performance. The predictions for JPCP for each of the major climatic zones are shown in table 38.[44]

The serviceability/performance criteria controlled the slab thickness design since it was the only criteria used. The JPCP with fine-grained subgrade soil requires approximately 0.4 in (1 cm) of slab thickness more than coarse-grained subgrade soil. The JPCP with dowel bars replaces approximate 0.7 in (1.8 cm) of slab thickness in freeze climatic zones and 1.0 in (2.5 cm) nonfreeze climatic zones. Climate inputs have a significant effect on design slab thickness. The difference in design slab thicknesses can be as high as 2.5 in (63 cm) between freeze and nonfreeze climatic zones. However, the difference between wet and dry climatic zones is only from 0.1 to 0.3 in (0.25 to 0.8 cm).

Severe pumping in wet climates especially with fine-grained subgrade soils is the major distress of the JPCP designs. The design nomographs do not provide analytical procedures to predict the erodability of the subbase as well as the joint design for each specific project situation. The joint faulting predicted is as high as 0.12 in (0.3 cm) for undoweled JPCP with fine-grained subgrade soil and in freeze climates. The presence of dowel bars significantly reduces the amount of faulting at the joints. Severe cracking was predicted for doweled JPCP with fine-grained subgrade soil in nonfreeze climates: 632 and 793 ft./mile (192 and 242 m) for wet (I-C) and dry (III-C) climates, respectively. This shows that these JPCP structural designs with fine-grained subgrade soil and with dowel bars are not adequate when designed in nonfreeze climates. The terminal serviceability index was predicted from 3.4 to 3.7 in the dry climatic zones while predicted from 2.6 to 2.9 in the wet climatic zone. This shows that the serviceability/performance design nomographs generally provide zero-maintenance performance for JPCP in dry climates but not in wet climates.

Table 37.

Design input parameters for 1982 Zero-Maintenance design nomographs.

Parameter	JPCP design
Design period, years	20
Initial serviceability	4.5
Terminal serviceability	3.0
Total 18-kip ESAL, million	15
* Subgrade soil type	fine-grained/coarse-grained
** Subbase type	4" CTB
Mean k-value @ top of subbase, pci	210/330
*** Average PCC modulus of rupture, psi	650
CV of PCC modulus of rupture, %	12
Concrete E value, psi	4,000,000
Dowels at joint	yes/no
Freezing Index, degree-days	
(I-A) wet-freeze zone	650
(I-C) wet-nonfreeze zone	0
(III-A) dry-freeze zone	650
(III-C) dry-nonfreeze zone	0
Annual Precipitation, cms	
(I-A) wet-freeze zone	89
(I-C) wet-nonfreeze zone	142
(III-A) dry-freeze zone	30
(III-C) dry-nonfreeze zone	61

* Subgrade k-value = 100 pci for fine-grained soil and 190 pci for coarse-grained soil

** Subbase E = 1,000,000 psi for CTB

*** Third-point loading, at 28 days

1 in = 2.54 cm, 1 psi = 0.07031 kg/cm^2, 1 pci = 0.02768 kg/cm^3

Table 38.

Predictions for JPCP designs using 1982 Zero-Maintenance design nomographs for major climatic zones.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Subbase type: 4 in CTB *
Working stress: 550 psi **
Joint Spacing: 15 ft.

Climatic zones	Dry-Freeze (III-A)				Wet-Freeze (I-A)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10.9	10.2	10.5	9.9	11.2	10.6	10.8	10.2
Dowel diameter, in	0	1.25	0	1.25	0	1.25	0	1.25
Pumping	1.6	1.9	0.9	1.2	2.2	2.5	1.6	1.9
Faulting, in	.12	.04	.12	.05	.12	.04	.12	.04
Cracking, ft/mi	213	377	217	354	166	262	169	268
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	3.7	3.6	3.7	3.6	2.7	2.6	2.7	2.6

Climatic zones	Dry-Nonfreeze (III-C)				Wet-Nonfreeze (I-C)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	8.9	8.0	8.6	7.6	9.1	8.1	8.7	7.8
Dowel diameter, in	0	1.25	0	1.25	0	1.25	0	1.25
Pumping	1.3	2.1	0.7	1.7	2.0	2.9	1.5	2.3
Faulting, in	.08	0	.08	0	.08	0	.08	0
Cracking, ft/mi	260	793	169	297	185	632	128	212
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	3.6	3.5	3.6	3.4	2.9	2.8	2.9	2.8

* The k-value at the top of the CTB is 210 pci for fine-grained subgrade soils and 330 pci for coarse-grained subgrade soils, respectively.

** The working stress is computed using a S_c = 650 psi with CV = 0.12 and Z = 1.28 (90 % of confidence level).

1 in = 1.54 cm, 1 ft = 0.3048 m, 1 mi = 1.6 km

3.4 California DOT Procedure for JPCP

The California Department of Transportation (Caltrans) Highway Design Manual was prepared by the Caltrans Office of Planning and Design.[49] Each section of this manual was issued at different dates from 1964 to 1986. Only jointed plain concrete pavement (JPCP) is used in California. The current design procedure is a series of standard concrete pavement cross sections and guidelines which were adopted from other States and local agencies in 1982.

__Conceptual Evaluation__.

Structural Design. The original structural design procedure used in California since 1967 was basically a modified PCA design procedure. This procedure was abandoned after the adoption of a series of standard concrete pavement cross sections and guidelines from other States and local agencies in 1982. Those selected cross sections were verified by using the 1981 AASHTO Interim Guide.

The design inputs for the JPCP design are the Traffic Index, subgrade strength in terms of an R-value, subgrade soil Plasticity Index (PI), and subgrade permeability. The concrete properties were built into the design tables. The Traffic Index (TI) is calculated using the following equation:

$$TI = 9.0\ (EAL/1000000)^{0.119} \tag{20}$$

where TI = traffic index
EAL = total 20 year equivalent 18-kip single-axle loads in design lane

The R-value of the subgrade was correlated to the k-value in order to utilize the PCA method without the necessity of having to conduct this expensive and time consuming subgrade strength test. The required minimum R-value of the subgrade is 10 and the maximum subgrade soil PI value is 12 for JPCP design.

The Caltrans procedure provides a range of structural design thicknesses for the PCC slab from a minimum of 6 in for a TI range of 6-7, to a maximum of 10.2 in (25.9 cm) for a TI greater than 12. Four types of bases: lean concrete base (LCB), asphalt concrete base (ACB), asphalt treated permeable base (ATPB) and cement treated permeable base (CTPB), are recommended. Subbase is not required if the subgrade R value is greater than 40. The structural design of JPCP is accomplished through the use of tables and the standard pavement cross sections. The Caltrans procedure recommends the use of a tapered cross section in the passing (inner) lane to avoid steps in the structural section on multilane facilities (where the traffic load between adjacent lanes indicates 2 different thickness of PCC) with a constant thickness of base.

Subdrainage Design - The Caltrans procedure provides guidelines for the design of pavement subdrainage. All JPCP designs are required to use either the treated permeable bases or the positive drainage collector and outlet system for rapid removal of infiltrated water.

Joint Design - Longitudinal joints, transverse joints and pressure relief joints are recommended in the Caltrans procedure. The transverse joints are recommended to have a random spacing of 12-15-13-14 ft (3.6-4.6-4.0-4.3 m) and to be skewed counterclockwise. No guidelines are provided for the design of joint shape as well as joint sealants. Mechanical load transfer devices at joints are not utilized.

Specific limitations of the Caltrans procedure are summarized as follows:

1. Slab Thickness - The maximum design slab thickness for a given Traffic Index of 12, the equivalent of 11.2 million 18-kip ESAL or higher, is 10.2 in (25.9 cm). The structural design may not be adequate on very high truck volume pavements having 30 to 100 million 18-kip ESAL.

2. Warping and Curling - The Caltrans procedure indirectly takes into consideration the effects of warping and curling in the slab due to variations of moisture and temperature gradients by specifying a short joint spacing (reduced from the previous 12-13-19-18 ft (3.6-4.0-5.8-5.5 m)). However, there is no way to adjust joint spacing for the different levels of support of stiff LCB to permeable asphalt treated base.

3. Joint Design - Longitudinal joints, transverse joints and pressure relief joints are recommended in the Caltrans procedure. These guidelines and standard sections for joint design in the Caltrans procedure are based on previous experience. No analytical design procedure is available for the designer to design for the specific project conditions. No recommendations are provided for the design of joint shape, joint sealant, and load transfer devices at joints.

4. Climate - Climate is a significant factor in pavement performance, but is not included in the design procedure.

5. Variability - No provision is made in the procedure for non-homogeneity and variability in material properties or the variations in design variables.

<u>Analytical Evaluation</u>.

Several JPCP designs were generated using the Caltrans procedure and the inputs are shown in table 39. A Traffic Index of 12.4 was used as it was the equivalent of 15 million 18-kip ESAL for a 20-year design period. All JPCPs were designed with 15-ft (4.6 m) joint spacing and without dowels. The computer program, PREDICT, was used to estimate the performance of the designs for the four major climatic zones, wet-freeze, dry-freeze, wet-nonfreeze, and dry-nonfreeze. Table 40 shows the predictions for these zones.

Since climatic conditions are not considered, the procedure only gives one design for all climatic zones. The maximum thickness of 10.2 inches slab was selected for the design Traffic Factor of 12.4. The subbases included 6 in (15 cm) of cement treated subbase plus 8.4 in (21 cm) of subbase for fine-grained subgrade soil vs. 6 in (15 cm) of cement treated subbase for coarse-grained subgrade soil. The fine-grained subgrade soil gives the same required slab thickness for JPCP as the coarse-grained

Table 39.

Design inputs for the Caltrans procedure for JPCP design.

Parameter	JPCP design
Design period, year	20
* Traffic Index	12.4
Subgrade soil	fine-/coarse-grained
R value of subgrade	25/43
** Subbase type	6 in CTB & 8.4 in ASB/6 in CTB

* Traffic Index = $9.0(EAL/1000000)^{0.119}$

** CTB = cement-treated base

ASB = aggregate subbase

1 ft = 0.3034 m

1 in = 2.54 cm

1 mi = 1.6 km

Table 40.

Predictions for JPCP designs using Caltrans procedure for major climatic zones.

Traffic Index: 12+ (15 million 18-kip ESAL/20 years)
Subbase type: 6 in CTB & 8.4" ASB for fine-grained subgrade soil
6 in CTB for coarse-grained subgrade soil
Joint spacing: 15 ft
Doweled at joint: No

Climatic zones	dry-freeze (III-A) *		wet-freeze (I-A)	
Subgrade soil	fine	coarse	fine	coarse
Slab thickness, in	10.2	10.2	10.2	10.2
Pumping	1.9	1.1	2.7	1.9
Faulting, in	.12	.12	.13	.12
Cracking, ft/mi	228	186	222	181
Joint deter., jts/mi	12	12	12	12
PSI	3.7	3.7	2.7	2.7

Climatic zones	Dry - Nonfreeze (III-C)		Wet - Nonfreeze (I-C)	
Subgrade soil	fine	coarse	fine	coarse
Slab thickness, in	10.2	10.2	10.2	10.2
Pumping	0.5	0	1.4	.5
Faulting, in	.07	.06	.07	.07
Cracking, ft/mi	70	59	57	47
Joint deter., jts/mi	12	12	12	12
PSI	3.9	4.0	3.2	3.3

* national nine climatic zones

1 ft = 0.3034 m

1 in = 2.54 cm

1 mi = 1.6 km

subgrade soil. The cement treated subbase was used only because PREDICT is not valid for permeable base. Severe pumping and low PSI value was predicted in wet-freeze zone with fine-grained subgrade. However, if the permeable subbase was used, the extent of pumping would be greatly reduced. The Caltrans procedure requires subdrainage for all JPCP designs, but the effect of subdrainage is not shown in the predicted results because the PREDICT program does not consider subdrainage as a factor in JPCP evaluation.

The results show that the Caltrans procedure generally provides adequate structural designs for JPCP except in the wet-freeze climate where serviceability drops off to 2.7. However, this may be alleviated with a permeable base layer.

The procedure does not permit the use of dowels for load transfer. Faulting in the freeze zones is quite high. Again, however, the use of a permeable base should greatly reduce the amount of pumping, which would reduce faulting at the joints.

3.5 PCA Procedure for JPCP and JRCP

The well known version of the Portland Cement Association (PCA) thickness design procedure for concrete pavements was first published in 1966.[52] The current revised version [53] was released in 1984 with new pavement foundation erosion criteria and the conventional fatigue criteria provided for thickness design. The PCA has also published design procedures for subgrades and subbase, joint design, and distributed steel.[54,55,56] The procedures are basically mechanistically founded. They are based on theoretical studies, research experience, and observations of performance of pavements in service. The thickness design procedure applies to three types of concrete pavements: JPCP, JRCP and CRCP.

Conceptual Evaluation.

The fundamental basis of the PCA procedure for structural design of concrete pavements is based on the fatigue damage concept which includes the erosion of support of the slab.

1. Fatigue Criterion - The fatigue criterion is set up to control fatigue cracking due to truck load repetitions. The fatigue criterion used in the procedure is similar to the previous PCA procedure except that edge-loading condition is now used. The critical stresses are of much higher magnitude with this approach. A modification in the high-load-repetitions in the previous fatigue curve was made to eliminate the discontinuity at a stress ratio of 0.5 that sometimes caused unrealistic effects. A lateral truck load placement condition of 6 percent of trucks at the slab edge is assumed. The allowable number of load repetitions for a given axle load is determined based on the stress ratio (SR) (flexural edge stress divided by the 28-day modulus of rupture).

The fatigue curve was incorporated into the design charts. Equations for the fatigue curve are:

Stress Ratio	Equation	
> 0.55	$\log_{10} N = (0.9718 - SR)/0.0828$	(21)
0.45 to 0.55	$N = (4.2577/(SR-0.4325))^{3.268}$	(22)
< 0.45	$N = \text{unlimited}$	(23)

where N equals allowable number of load repetitions. Use of the fatigue damage is based on the Miner's hypothesis that fatigue damage not consumed by repetitions of one load is available for repetitions of other loads. For each design, the maximum limit of the total fatigue consumed is set to be 100 percent.

2. Erosion Criterion - The erosion criterion is set up to control the failure of pavements due to excessive pumping, erosion of the foundation, and joint faulting. A power term was established which provided better correlation with the AASHO Road Test performance data than slab deflections. The power term, or the rate of work, is defined as a function of the pressure at the slab-foundation interface, the radius of relative

stiffness of the slab, the slab corner deflection, the truck speed, and the subbase stiffness. The development of the erosion criterion was also correlated to the studies on joint faulting.[57,58] The truck load placement condition with 6 percent of trucks at edge is again assumed. The equation for erosion damage is:

$$\text{Percent erosion damage} = 100\ \Sigma n_i\ (C/N_i) \qquad (24)$$

where n_i = expected number of axle-load repetitions for axle-group i

N_i = allowable number of repetitions for axle-group i

C = 0.06 for pavements without shoulder, 0.94 for pavements with shoulder

The essential features of the PCA procedure are summarized as follows:

1. Traffic - The data on average daily truck traffic (ADTT) in both directions and the axle-load distribution of the truck traffic are used to compute the repetitions of various single- and tandem-axle loads expected during the design period. The procedure provides several typical axle-load distributions for use when actual axle-load data are not available.

2. Load safety factor - The axle loads are multiplied by a load safety factor (LSF) which is provided in the procedure to compensate for the possibility of unprotected heavy truck overloads and normal construction variations in material properties and layer thickness. A value of 1.2 is recommended for pavements designed with uninterrupted traffic flow and high volumes of truck traffic.

3. Concrete properties - The concrete flexural strength at 28-days is determined from modulus of rupture test at third-point loading. Variation in concrete strength is considered by reducing the modulus of rupture by one standard deviation (a coefficient of variation of 15% is assumed) which is incorporated into the design charts and tables. The procedure also incorporates a value of 4,000,000 psi (281,240 kg/cm^2) for the concrete modulus of elasticity and an adjustment factor for the effect of concrete strength gain after 28 days in the design charts and tables.

4. Subgrade and subbase support - Three types of subbases, untreated, cement-treated and lean concrete, are suggested in the procedure. The k-value at the top of subbase is estimated from information on the thickness and type of the subbase and the k-value of the subgrade soil (deflection based criteria).

5. Joint design - PCA provides several joint design recommendations in the procedure.[55] Included in the procedure are recommendations for joint type, spacing, shape, load transfer device (LTD), randomized and skewed transverse joints, and stabilized subbases. Contraction joints are recommended for regular transverse joints spaced at approximately 40 ft (12.2 m) or less for JRCP and up to 20 ft (6.1 m) for JPCP. Specific dimensions for the joint sealant reservoir for various joint spacings are provided. Dimensions and spacing for doweled load transfer devices are provided along with recommendations for corrosion proofing of the LTDs. The

use of stabilized subbases or dowels or both, and use of skewed joints and randomized joint spacing is recommended for heavy traffic volume JPCP to prevent faulting (no specific criteria given). LTDs are recommended for all JRCP designs.

6. Reinforcement design - The recommendations for the design of longitudinal steel in JRCP are similar to those provided by AASHTO Guide.[39]

Specific possible limitations of the design procedure are summarized as follows:

1. Stress Analysis - In addition to traffic loading, concrete slabs are also subjected to warping and curling stresses. The PCA procedure does not take into consideration the increase in load stresses caused by warping and curling in the slab due to variations of moisture and temperature gradients.

2. Friction - The tensile stress caused by friction between the slab and the subbase, and the tensile stresses caused by full or partial seizure of the load transfer devices across contraction and expansion joints are not considered.

3. Foundation support - The effect of nonuniform support resulting from nonuniform subgrade and subbase materials, and partial loss of support due to slab curl are not considered in the stress analysis in the procedure.

4. Subbase - The effect of a type of subbase is considered only through its effect on the k-value based on deflection. For a stiff subbase, this approach will give unconservative results for reducing actual stresses in the slab (e.g., stiff base does not reduce stress as it does deflection). Though it is acknowledged that the erodibility of conventional subbases is different from that of high strength subbases, the difference in type of subbase used is not addressed by the procedure. Lean concrete subbases are considered to be highly nonerodible. However, erosion might still occur below the lean concrete layer, which could result in loss of support.

5. Climate and drainage - Climate and drainage are significant factors in pavement performance, but are not included in the design procedure. The procedure suggests that the erosion criterion be modified by local experience.

6. Variability - Little provision is made in the procedure for non-homogeneity and variability in engineering properties of the concrete, subbase and subgrade materials, except through the load safety factor and through the reduction in one coefficient of variation of concrete modulus of rupture.

7. Joint design - These recommendations for joint design in the PCA procedure are based on previous experience. No analytical design procedure is available to allow the designer to design for the specific project conditions. No recommendations are provided for

different types of field molded joint sealant. The procedure does not provide guidance when dowels should or should not be used. A 2-in additional slab thickness for undoweled pavement is not supported.

8. Reinforcement Design - Reinforcement design recommendations are similar to those provided by AASHTO Guide, hence, similar limitations would exist (see section 3.2).

9. Functional performance - No provisions are made for the effect of time and environment on the functional performance of the pavement, or for correlation between functional (roughness) performance and fatigue of pavement slabs.

Analytical Evaluation.

The analytical evaluation was carried out by using the Specific Design Evaluation approach. A number of design situations were developed using the PCA design procedure. The axle-load distribution used for the design is shown in table 34. The different design factors used included fine- and coarse-grained subgrade soils, 4 in (10 cm) CTB (for JPCP) and 6 in (15.2 cm) granular subbases (for JRCP), with and without dowels (for JPCP only), and shorter and longer joint spacings (JRCP only). Table 41 shows the specific input parameters and their values for both JPCP and JRCP designs. All pavements were designed with a load safety factor of 1.2 and without concrete shoulders. Reinforcement steel was designed using the 6x12 W4xW4 welded smooth wire fabric.

These designs then were evaluated using the NCHRP Project 1-19 COPES "PREDICT" program to predict their distresses and performance. The specific climatic variables for the nine climatic zones which were input to the program are shown in table 22. Previous evaluation results have shown that significant difference exists only between the four major zones: wet-freeze, wet-nonfreeze, dry-freeze and dry-nonfreeze. The predictions for JPCP and JRCP for each of the major climatic zones are shown in table 42 and table 43.

1. Jointed Plain Concrete Pavement: The fatigue analysis controls the design with dowels while the erosion analysis controls the design without dowels. Since climate and drainage factors are not considered, the PCA procedure only gives one design for all climatic zones. The required slab thickness for JPCP with fine-grained subgrade soil is about 0.5 in (1.3 cm) greater than that with coarse-grained subgrade soil. More severe pumping was predicted with fine-grained subgrade than with coarse-grained.

The required slab thickness for JPCP with dowels is approximately 1.5 in (3.8 cm) less than that without dowels. JPCP designs with dowels give less faulting distress than designs without dowels. However, severe cracking was predicted in freeze areas for JPCP with doweled joints. This shows that the structural design is not adequate for JPCP with dowels in freeze areas (cracking is transverse cracking which is not related to joint design). Severe pumping also was predicted in the wet-freeze climatic zones. The present serviceability index was predicted lower than 3.0 in the wet-freeze zone. The erosion criterion was not adjusted for different climates in these designs. There are no guidelines provided to adjust the erosion criterion, although they can be changed.

Table 41.

Design input parameters for the PCA design procedure.

Parameter	JPCP	JRCP
Design period, years	20	20
Design ADTT, both directions	3,000	3,000
* Subgrade soil	fine/coarse	fine/coarse
** Subbase type	4" CTB	6" granular
k-value @ top of subbase, pci	280/450	140/220
Concrete shoulder	no	no
*** Modulus of rupture, psi	650	650
Concrete E value, psi	4,000,000	4,000,000
Joint spacing, ft	15	27/40
Doweled at joint	yes/no	yes
Load safety factor	1.2	1.2

* Subgrade k-value = 100 pci for fine-grained soil and 190 pci for coarse-grained soil

** Subbase E = 1,000,000 psi for CTB and 30,000 psi for granular

*** Third-point loading, at 28 days

1 inch = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 42.

Predictions for JPCP designs using PCA design procedure for varying climatic zones for major climatic zones.

Design ADTT: 3000 (both directions)
Design period: 20 years
Subbase type: 4 in CTB
Joint spacing: 15 ft

Climatic zones	dry-freeze (III-A)				wet-freeze (I-A)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10.5	9	10	8.5	10.5	9	10	8.5
Dowel diameter, in	0	1.125	0	1.125	0	1.125	0	1.125
Pumping	1.8	2.6	1.2	2.2	2.6	3.0	2.0	3.0
Faulting, in	.12	.07	.12	.07	.12	.07	.13	.07
Cracking, ft/mi	244	1021	263	1134	237	1007	256	1119
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	3.7	3.4	3.7	3.4	2.7	2.4	2.6	2.4

Climatic zones	Dry - Nonfreeze (III-C)				Wet - Nonfreeze (I-C)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10.5	9	10	8.5	10.5	9	10	8.5
Dowel diameter, in	0	1.125	0	1.125	0	1.125	0	1.125
Pumping	.4	1.3	0	.8	1.2	2.1	.6	1.6
Faulting, in	.06	.01	.07	.01	.07	.01	.07	.02
Cracking, ft/mi	73	196	73	157	59	167	59	126
Joint deter., jts/mi	12	12	12	12	12	12	12	12
PSI	3.9	3.6	3.9	3.6	3.2	3.0	3.2	2.9

1 in = 2.54 cm

1 ft = 0.3048 m

1 psi = 0.07031 kg/cm^2

1 psi/in = 0.02768 kg/cm^3

Table 43.

Predictions for JRCP designs using PCA design procedure for major climatic zones.

Design ADTT: 3000 (both directions)
Design period: 20 years
Subbase type: 6 in granular

Climatic zones	dry-freeze (III-A)				wet-freeze (I-A)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10		9.5		10		9.5	
Dowel diameter, in	1.25		1.25		1.25		1.25	
Joint spacing, ft	27	40	27	40	27	40	27	40
Area of steel, in^2/ft	.08	.08	.08	.08	.08	.08	.08	.08
Pumping	0	0	0	0	2.6	2.6	2.4	2.4
Faulting, in	.02	.06	0	.01	.08	.12	.01	.06
Cracking, ft/mi	903	1035	983	1153	1287	1418	1213	1383
Joint deter., jts/mi	0	50	0	50	0	50	0	50
PSI	3.6	3.5	3.5	3.4	3.1	3.0	3.1	3.0

Climatic zones	Dry - Nonfreeze (III-C)				Wet - Nonfreeze (I-C)			
Subgrade soil	fine		coarse		fine		coarse	
Slab thickness, in	10		9.5		10		9.5	
Dowel diameter, in	1.25		1.25		1.25		1.25	
Joint spacing, ft	27	40	27	40	27	40	27	40
Area of steel, in^2/ft	.08	.08	.08	.08	.08	.08	.08	.08
Pumping	0	0	0	0	2.2	2.2	1.9	1.9
Faulting, in	0	.06	0	0	.05	.1	0	.04
Cracking, ft/mi	901	1033	981	1151	1055	1187	1064	1234
Joint deter., jts/mi	0	35	0	35	0	35	0	35
PSI	3.5	3.4	3.4	3.3	2.9	2.9	2.9	2.8

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

The results, in general, show that the PCA procedure designs provide adequate structural designs for JPCP in the nonfreeze climates but not in freeze climates, especially with doweled joints.

2. Jointed Reinforced Concrete Pavement: All JRCP were designed with dowels. The fatigue analysis generally controls the designs. The required slab thickness for JRCP with coarse-grained subgrade soil is 0.5 in (1.3 cm) less than that with fine-grained subgrade soil. Both fine- and coarse-grained subgrades give about the same performance for JRCP designs. Fine-grained subgrades have a little more pumping but none was predicted for JRCP designs in dry climates. Severe pumping and lower present serviceability index was predicted in the wet climates. The 40 ft (12.2 m) joint spacing gives a large amount of joint deterioration while the 27 ft (8.2 m) joint spacing gives very good performance.

The results generally show that the PCA procedure provides adequate structural designs for JRCP in the dry climates, although a fair amount (over 1000 ft per mile (189 m/km)) of deteriorated transverse cracks is projected. Pumping is predicted to occur severely in wet climates for the JRCP especially with fine-grained subgrade soils. Joint deterioration for long joint spacings (40 ft (12.2 m)) was substantial in all climates.

3.6 RPS-3 Texas SDHPT Procedure for JRCP

The rigid pavement design system computer program, designated RPS, was developed in 1971. [59] The currently available program, named RPS-3, is the third revised version and was released in 1975.[60] The design procedure in this program is very similar to the 1972 version of the AASHO Interim Guide, but contains a few differences and some important additions.[37] The program has the capability to generate the designs for JRCP and CRCP, with AC or PCC overlay, and provides cost information for economic comparison of alternatives. A detailed description for the input variables of the program is provided in the RPS-3 User's Manual.[60] The output of the program gives a summary table having up to 23 alternate pavement designs in the order of increasing total overall cost. The selection of the optimal design is based on the minimum total overall cost.

Conceptual Evaluation.

Structural Design - The structural design equation used in the RPS-3 procedure was derived from the results of the AASHO Road Test. A complete derivation of the design equation is provided in the RPS-1 report [59]. The derivation is identically the same as the AASHTO Design Guide Equation 3.3. The design equation is as follows:

$$\text{Log } W_{18m} = 7.35 \text{ Log } (DT_m) - 0.05782 + G / [\, 1 + (16.196*10^6)/(DT_m{}^{8.46})\,] \qquad (25)$$

where $DT_m = 183.9 \,/\, [(690\, s_{cm} / f_c)^{0.5222}\,]$

and $s_{cm} = (\, J\, L \,/\, D^2\,)(1 - a_1 \,/\, \ell\,)$

W_{18m} = the modified number of 18-kip single-axle loads that a pavement with different physical properties will sustain

L = load in pounds

a_1 = 2 a, where a is radius of a circle equal in area to the load area

ℓ = radius of relative stiffness (see equation 16)

f_c = allowable flexural strength of concrete

G and J are defined in equations 2 and 4. The J factor in this equation is 3.2 for the jointed concrete pavement. A pavement "life-term" factor of 0.9155 was used in RPS-1 to be multiplied to the logarithm of predicted applications in equation 22 to reduce the long-term traffic for gradual deterioration from climatic exposure. This causes an increase in the design slab thickness. The k-value at the top of the subbase in the procedure is determined by the prediction models developed using elastic layered theory for deflection. The k-value also is modified with an erodability factor E_f accounting for the loss of support due to void spaces generated along the edges and the joints. The reliability concept is employed in this procedure to consider variability and uncertainty in design. Table 44 summarizes the reliability factor values for the RPS-3 procedure for levels of reliability compared to those for the 1986 AASHTO Guide.

Table 44.

Summary of the reliability design factor for specified reliability levels in RPS-3 and AASHTO guide.

Design Procedure	Reliability level 50%	80%	90%	95%	99%	99.9%
RPS-3 Procedure	1.0	1.8415	-	2.6450	3.3267	4.090
AASHTO Guide *	1.0	1.79	2.42	3.12	4.99	8.45

* Overall standard deviation is 0.30 (for rigid pavements).

Joint Design - No guidance is provided with regard to joint spacing. The user can restrain the lower and upper joint spacing limit. Since the total cost of transverse joints decreases as the required amount of steel increases, the program determines the joint spacing by optimizing the area of steel and the joint spacing in order to give minimum total cost of the joints and reinforcement. Tie bars will be designed in longitudinal joints if mesh type reinforcement used. No guidelines are provided for the design of joint layout, joint shape and joint sealants. No recommendation is provided in the procedure with regard to the type, dimension and spacing for load transfer devices.

Reinforcement Design - The longitudinal steel is designed with the subgrade drag theory by the equation similar to the AASHTO Design Guide equation 5.

$$A_s = [\ D\ wc\ Ld\ Fa\ ]\ /\ 24\ fs \qquad (26)$$

where A_s = area of steel, in^2 /ft. of slab width

D = thickness of slab, in.

w_c = weight of concrete, lbs/ft^3

L_d = distance between free transverse joints, ft.

F_a = average value of coefficient of support resistance

f_s = allowable unit stress in reinforcement,psi

The procedure recommends coefficients of resistance 1.8 and 1.5 for the stabilized and granular subbases, respectively. The working stress is taken to be 0.75 times the yield point strength. Two types of reinforcement, deformed rebar steel and welded wire mesh steel plus tie bar, are available in the design procedure.

Overlay Design - The AC overlay model in the procedure was developed using the layered elastic theory. The Corps of Engineers method is used for the PCC overlay design. A detailed description for both models is provided in the procedure.

Cost Analysis - The RPS-3 program results in alternate designs which are compared and optimized in the program by the single decision criterion of overall costs of the designs. Relative comparisons among designs are made with all future costs discounted back to present worth. A compound interest model is used to discount the future costs with the interest rate input by the user. Detail cost analysis is also provided by the procedure.

Specific limitations of the design procedure are summarized as follows:

1. General - Most of the limitations previously presented for the AASHTO Guide for design of JRCP (see section 3.2) are applicable to this procedure.

2. Joints - The joint design deficiencies listed for the AASHTO Guide (see section 3.2) are applicable to this procedure. Recommendations are not provided for any other type of joint. No guidance is provided for the joint spacing. The procedure just optimizes the joint spacing value and the area of steel in order to decrease the cost of transverse joints and reinforcement and achieves the minimum total cost of the design. Long joint spacing, though decreasing the cost for dowels, will produce more deteriorated joints and cracks. No recommendation is provided in the procedure for the load transfer design.

3. Reinforcement Design - The procedure uses a similar equation as in the AASHTO Guide. This equation is a major simplification of the actual forces encountered. It is expected that long joint spacing in cold areas accompanied by joint seizure would result in rupture of the reinforcement with subsequent faulting and spalling of the crack from heavy traffic. No recommendation is provided for the control of steel corrosion. The limitation for the reinforcement design for the AASHTO Guide (see section 3.2) is also applicable.

4. Miscellaneous - No recommendations for drainage design and shoulder design.

However, there are some factors which would lead to a longer life than predicted by the design equation. These factors are the adjustment (by a factor of 0.9115) of the life term and the provision of the reliability concept.

<u>Analytical Evaluation</u>.

Several JRCP designs were generated using the RPS-3 program and the inputs in table 45. The design factors varied included 4 in (10.2 cm) of bituminous aggregates mixture (BAM) versus 6 in (15.2 cm) of granular subbase. The traffic levels for 20 year design was 5 and 15 million 18-kip ESAL, and different levels of reliability were used from 50 to 95 percent. JRCP was then designed for these different conditions using the RPS-3 program. All JRCP were designed without overlays.

The NCHRP Project 1-19 program, PREDICT, was used to estimate the performance of the designs for the four major zones, namely, wet-freeze (I-A), dry-freeze (III-A), wet-nonfreeze (I-C), and dry-nonfreeze (III-C).[44] table 46 and table 47 show the predictions for each of the four zones at 50 percent reliability level. Table 48 shows the predictions for the wet-freeze zone (I-A cell) with varying levels of reliability.

1. Thickness Design. Since climatic conditions are not considered, the procedure only gives one design for all climatic zones. The required slab thickness for JRCP with BAM subbase is about 1/2 in less than that with granular subbase. Severe pumping was predicted in wet areas and especially in the wet-freeze climatic zones.

Designs with a BAM subbase shows more cracking and a lower present serviceability rating than with a granular subbase at 5 million ESAL traffic

Table 45.

Design input parameters for RPS-3 procedure for JRCP designs.

Parameter	JRCP design
Reliability level, %	50/80/95
Design period, years	20
Traffic, million 18-kip ESAL	5/15
* Subgrade soil	fine-grained
** Subbase type	4 in BAM / 6 in granular
k-value @ top of subbase, pci	280/200
Initial serviceability	4.5
Terminal serviceability	2.5
*** Modulus of rupture, psi	650
Concrete E value, psi	4,000,000
Doweled at joint	yes
J factor	3.2
Erodability factor	1.0/0.5

* Subgrade M_R = 3,000 psi for fine-grained soil

** Subbase E = 650,000 psi for BAM and 30,000 psi for granular

*** Third-point loading, at 28 days

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 46.

Predictions for JRCP designs using RPS-3 procedure
5 million ESAL.

Design traffic: 5 million 18-kip ESAL
Design period: 20 years
Joint spacing: 50 ft
Level of reliability: 50%

Climatic zone	dry-freeze (III-A)		wet-freeze (I-A)	
Slab thickness, in	8.5	9	8.5	9
Subbase type	BAM	granular	BAM	granular
Dowel diameter, in	1.25	1.25	1.25	1.25
Area of steel, in^2/ft	.083	.069	.083	.069
Pumping	.3	0	2.2	1.8
Faulting, in	.07	.06	.09	.08
Cracking, ft/mi	441	431	588	481
Joint deter., jts/mi	38	38	38	38
PSI	3.5	3.6	3.3	3.3

Climatic zone	dry-nonfreeze (III-C)		wet-nonfreeze (I-C)	
Slab thickness, in	8.5	9	8.5	9
Subbase type	BAM	granular	BAM	granular
Dowel diameter, in	1.25	1.25	1.25	1.25
Area of steel, in^2/ft	.083	.069	.083	.069
Pumping	.5	0	2.0	1.6
Faulting, in	.07	.06	.09	.07
Cracking, ft/mi	440	429	529	456
Joint deter., jts/mi	24	24	24	24
PSI	3.5	3.6	3.1	3.2

1 in = 2.54 cm

1 ft = 0.3048 m

1 psi = 0.07031 kd/cm^2

1 psi/in = 0.02768 kg/cm^3

Table 47.

Predictions for JRCP designs using RPS-3 procedure - 15 million ESAL.

Design traffic: 15 million 18-kip ESAL
Design period: 20 years
Joint spacing: 50 ft
Level of reliability: 50%

Climatic zone	dry-freeze (III-A)		wet-freeze (I-A)	
Slab thickness, in	10	10.5	10	10.5
Subbase type	BAM	granular	BAM	granular
Dowel diameter, in	1.25	1.25	1.25	1.25
Area of steel, in^2/ft	.098	.083	.098	.083
Pumping	0	0	2.6	2.3
Faulting, in	.1	.09	.16	.14
Cracking, ft/mi	729	1012	1113	1202
Joint deter., jts/mi	44	44	44	44
PSI	3.8	3.6	3.3	3.2

Climatic zone	dry-nonfreeze (III-C)		wet-nonfreeze (I-C)	
Slab thickness, in	10	10.5	10	10.5
Subbase type	BAM	granular	BAM	granular
Dowel diameter, in	1.25	1.25	1.25	1.25
Area of steel, in^2/ft	.098	.083	.098	.083
Pumping	0	0	2.2	1.9
Faulting, in	.09	.09	.13	.12
Cracking, ft/mi	727	1010	881	1076
Joint deter., jts/mi	29	29	29	29
PSI	3.7	3.5	3.1	3.0

1 in = 2.54 cm

1 ft = 0.3048 m

1 psi = 0.07031 kg/cm^2

1 psi/in = 0.02768 kg/cm^3

Table 48.

Predictions for JRCP designs with varying levels of reliability using RPS-3 procedure.

Climatic zone: wet-freeze
Design period: 20 years

Design traffic = 5 million 18-kip ESAL

Reliability level, %	50		80		95	
Slab thickness, in	8.5	9	10	10	11	11.5
Subbase type	BAM	granular	BAM	granular	BAM	granular
Joint spacing, ft	50	50	50	50	40	50
Dowel diameter, in	1.25	1.25	1.25	1.25	1.25	1.25
Area of steel, in^2/ft	.083	.069	.098	.083	.083	.09
Pumping	2.2	1.8	1.8	1.3	1.0	.9
Faulting, in	.09	.08	.08	.07	.04	.05
Cracking, ft/mi	588	481	342	251	124	145
Joint deter., jts/mi	38	38	38	38	43	38
PSI	3.4	3.3	3.6	3.6	4.0	3.8

Design traffic = 15 million 18-kip ESAL

Reliability level, %	50		80		95	
Slab thickness, in	10	10.5	11.5	12	13	13.5
Subbase type	BAM	granular	BAM	granular	BAM	granular
Joint spacing, ft	50	50	40	50	40	40
Dowel diameter, in	1.25	1.25	1.25	1.25	1.25	1.25
Area of steel, in^2/ft	.098	.083	.09	.098	.098	.09
Pumping	2.6	2.3	1.8	1.7	1.5	1.4
Faulting, in	.16	.14	.08	.11	.06	.06
Cracking, ft/mi	1113	1202	557	836	425	726
Joint deter., jts/mi	44	44	50	44	50	50
PSI	3.3	3.2	3.6	3.4	3.7	3.5

1 in = 2.54 cm

1 ft = 0.3048 m

1 psi = 0.07031 kg/cm^2

1 psi/in = 0.02768 kg/cm^3

level, however, for 15 million ESAL the reverse is true. The performance of the designs is improved as the design level of reliability increases, except for joint deterioration. The reliability design factors in Table 3.22 show that the RPS-3 design equation has greater variance at 50 to 90 percent reliability level and smaller variance at 90 to 99.9 percent compared to AASHTO.

The results show that the procedure generally provides adequate structural designs for JRCP although there is projected a fair amount (over 1000 ft) of deteriorated transverse cracks per mile.

2. Joint Design. The procedure optimizes joint spacing and amount of area of steel in order to decrease the cost of transverse joints and achieve the minimum total cost of the design. The joint spacings chosen by the program for the JRCP design are 40 ft to 50 ft (12.2 to 15.2 m) in length. Dowel size, which is not recommended in the procedure, was assumed to be 1.25 inches in diameter.

Table 46 and Table 47 show that the joint deterioration occurs more severely in freeze regions (38 to 44 jts./mile for both traffic levels) than in nonfreeze regions (24 to 29 jts./mile) for 50-ft joint spacing. The number of deteriorated joints per mile is higher for the 40 ft joint spacing than for the 50 ft joint spacing.

The procedure does not give any guidance for the load transfer device design for the joints, nor for noncorrosive LTD design. The joint design in this procedure is not adequate to provide enough joint performance.

3. Reinforcement design. Mesh type reinforcement was chosen by the program for JRCP design with the yield stress of 75000 psi. Concrete unit weight input value is assumed to be 145 lb/ft^3 (2323 kg/cm^3). The coefficient of resistance is 1.8 and 1.5 for the BAM and granular subbases, respectively. The results show that the coefficient of resistance between the slab and its supporting subbase is very critical to the area of steel. The amount of area of steel for the BAM subbase design is 0.014 to 0.015 in^2/ft. (0.296 to 0.317 cm^2/m) greater than the granular subbase.

The rupture of the reinforcement would occur as expected in freeze area where considerable deicing salts are used during the winter. Long joint spacing with more cracks also interacts with the loss of effective reinforcement through corrosion and the rupture of the reinforcement. No recommendation is provided for the control of steel corrosion.

Significantly increased slab thickness and the corresponding increase in reinforcement for higher levels of reliability apparently reduces deterioration of the resulting cracks.

In general, the reinforcement design provided by the RPS-3 procedure provides adequate reinforcement for JRCP in areas where no joint lockup occurs.

3.7 Development of Illinois CRCP Predictive Model

It was believed that the performance of JPCP and JRCP is so different from CRCP that this evaluation could not rely on jointed pavement predictive models. The first known predictive model based upon actual CRCP was developed by utilizing the Illinois CRCP database under the research project IHR-901.[63] The Illinois CRCP database contains information on 113 pavement sections on the Interstate Highways System in Illinois. All sections were surveyed in 1977 and 24 sections were also resurveyed in 1985 and added to the database.

The database includes a variety of factors such as: total number of accumulated 18-kip ESAL from 0.7 to 30.8 million per truck lane with a mean value of 5.6 million; age from 3 to 20 years with a average 10.2 years; type of distress including patches, punchouts, steel ruptures cracks and "D" cracking; slab thickness from 7 to 10 in (17.8 to 25.4 cm); type of subbase including bituminous-aggregate mixture (BAM), cement aggregate mixture (CAM) and granular; reinforcement content from 0.5 to 0.7 percent (or the area of steel from 0.041 to 0.062 in^2/in width of slab); AC shoulders and some with underdrains. Almost all the sections have AC type of shoulders.

The most critical types of distress in CRCP including punchouts, steel ruptures, and existing patches of punchouts or steel ruptures combined as failures per mile or "FAIL". Also, pavement sections with "D" cracking were eliminated to remove this influence from the database. The model was developed using a combination of multiple linear regression and nonlinear regression techniques as included in the SPSS statistical package.[43] Multiple linear regression was utilized to determine which independent variables were significantly affecting the dependent variables. The non-linear regression was then utilized to compute the coefficients and exponents for the final predictive model.

The final Illinois prèdictive model for failure of CRCP is as follows:

$$\begin{aligned} FAIL = {} & 0.0001673\ ESAL^{1.9838}\ THICK^{-4.2772}\ ASTEEL^{-5.0} \\ & + 0.4127\ ESAL^{1.9553}\ (\ 0.01584\ BAM + 1.9080\ CAM \\ & - 0.02005\ BAR\ ) \end{aligned} \tag{27}$$

where FAIL = total number of punchouts plus steel ruptures plus number of patches per lane mile
ESAL = accumulated 18-kip equivalent single-axle loads outer lane, millions
THICK = slab thickness, in.
ASTEEL = area of reinforcement, in^2 /in width of slab
BAM & CAM = both zero (0), if subbase material is granular
1 & 0, if subbase material is BAM
0 & 1, if subbase material is CAM
BAR = 0, if deformed welded steel fabric used
1, if deformed rebars used

Statistics: $R^2 = 0.62$
SEE = 2.86 failures/mile (standard error of estimate)
n = 137 (number of observations)

The results show that the pavement sections in the Illinois database have a average failures per mile value of 2.26 (1.4 failures/km) with a standard deviation of 4.615. This model must be used with caution since it is empirical and invalid when used beyond the range of data from which it was developed.

There are some deficiencies in the Illinois CRCP database listed as follows:

- Only one climatic zone: wet freeze.
- Only one type of subgrade soil: fine-grained.
- Only one type of shoulder: AC shoulder.
- There exist situations in which there was not a sufficient range of some of the variables (e.g. slab thickness, subbase type, reinforcement content).
- Negative predicted values occurs if either of the design components is larger (i.e. thicker slab, higher reinforcement, etc.) than that in the database.

A sensitivity analysis of the CRCP model was conducted and some results are shown in figure 18. This graph illustrates the extent of failures per mile versus amount of reinforcement with varying thicknesses of slab for a given traffic level. The results show that thickness and reinforcement content have a very large effect on the failure of the CRCP. Other sensitivity analyses show that ESAL also has a large effect.

The granular subbase generally gave the same amount of failures per mile as the BAM as shown in other analyses. However, the CAM has predicted by far the worst performance and has many failures per mile. There were only a few sections with CAM subbase in the database, and they may be unrepresentative of overall CAM performance.

It is believed that the empirical model is reasonable and useful for evaluations of CRCP designs that fall within the data from which it was developed. Poor results will be obtained out of this range. Improved mechanistic-empirical models should be developed.

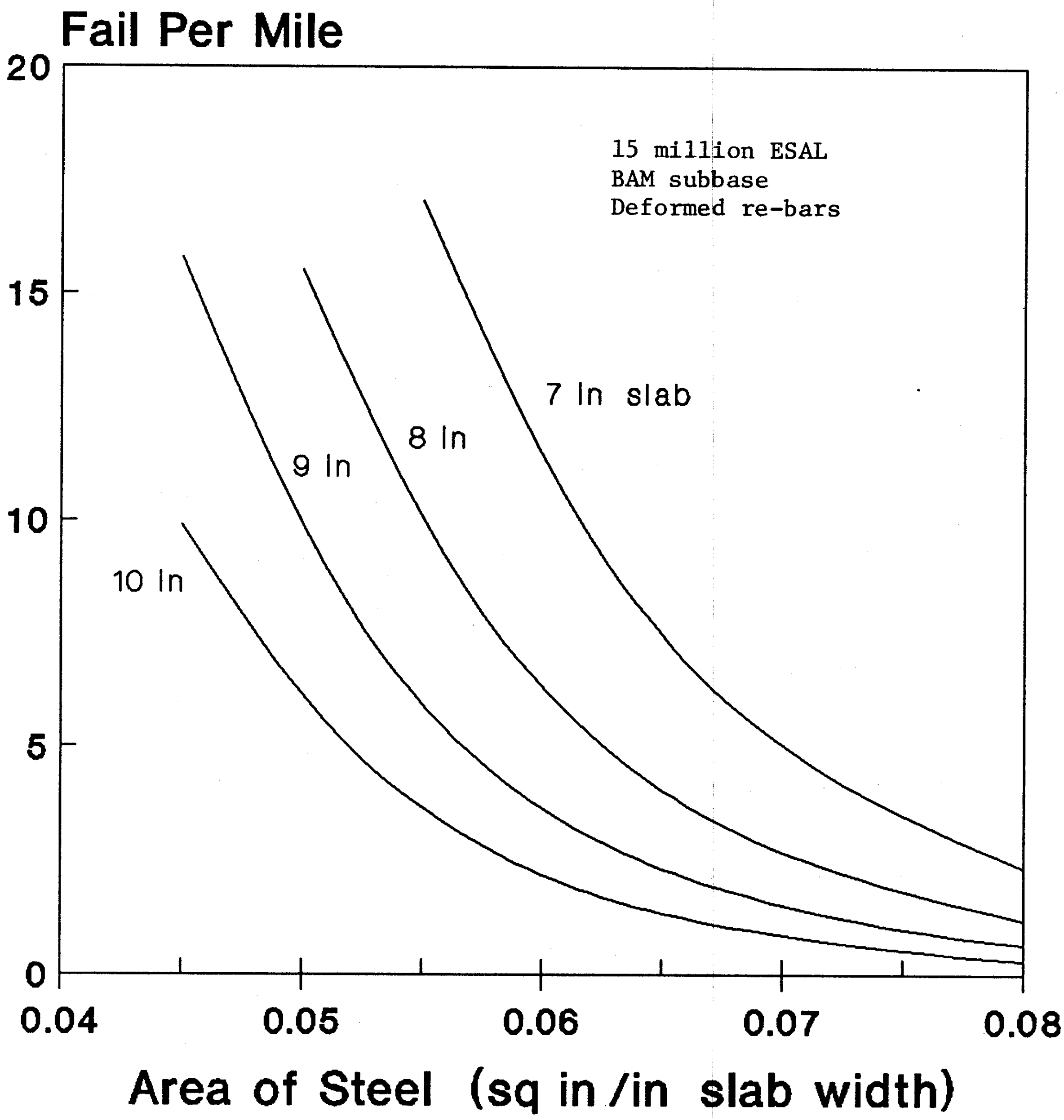


Figure 18. Sensitivity of CRCP failures per mile vs. area of steel.

3.8 1986 AASHTO Design Guide for CRCP

The AASHO Interim Guide for concrete pavement design developed in 1962 was originally for jointed concrete pavements only. However, the Interim Guide (1972) and the new Design Guide (1985) contain a design procedure for the design of continuously reinforced concrete pavements (CRCP).[38,39]

3.8.1 Conceptual Evaluation.

Structural design - The structural design of CRCP is based on equation 4 which was developed from the AASHO Road Test data for jointed concrete pavements and modified by the inclusion of the corner stress model of Spangler.

The only difference between CRCP and JPCP or JRCP design is the use of the J factor (or joint continuity factor). The new Guide recommends the values of J factor which are 3.2 for jointed pavements with dowels and 2.9 to 3.2 (3.2 is recommended) for CRCP without a tied concrete shoulder (edge loading condition) instead of the 2.2 value recommended in the previous 1972 version. If tied concrete shoulders are used, the value of J will be 2.3 to 2.9, with a value of 2.6 recommended. The effect of a lower J factor results in the reduction in slab thickness due to the reduction in calculated maximum corner stress in the concrete slab.

Reinforcement Design - The design objective of longitudinal reinforcement is to limit the crack spacing, crack width and the steel stress in CRCP to minimize punchouts. The design inputs include concrete tensile strength, concrete shrinkage, concrete thermal coefficient, reinforcing bar diameter, steel thermal coefficient, design temperature drop as well as the wheel load tensile stress developed by the construction equipment or truck traffic. The higher indirect tensile strength recommended by the Guide, 86 percent of concrete modulus of rupture, compared to the commonly used value (about 65 percent) would result in increased reinforcement. Three limiting criteria are given as follows:

1. The maximum crack spacing should be 8 ft to minimize the incidence of crack spalling, and a minimum of 3.5 ft to minimize the development of punchouts.

2. The allowable crack width should not exceed 0.04 in to protect against the spalling and water penetration.

3. The limiting stress of 75 percent of the ultimate tensile strength of the steel is set to guard against steel rupture.

A complete set of worksheets and nomographs for the procedure to solve the required reinforcement is provided in the new Guide.

The design for transverse steel in CRCP, if provided, is exactly the same as in JRCP using equation 5.

Joint Design - Construction joints are recommended for CRCP in the AASHTO Guide. The guidelines for the design of transverse contraction joints are also applicable to construction joints.

Specific limitations of the design procedure are summarized as follows:

1. General - Nearly all limitations previously presented for the AASHTO Guide for design of JPCP and JRCP are applicable to the design of CRCP, such as lack of consideration of material variability, loss of foundation support, design period, climate, load equivalency factors, and limiting criteria. The applicability of the equations to CRCP developed from jointed pavements at the AASHO Road Test has never been verified, and has some obvious serious problems. These problems include the use of corner stress for thickness design, closely spaced cracks with higher deflections which behaves more flexibly, and the amount of reinforcement has a very great effect on performance.

2. Joint continuity factor - The joint continuity factor values for CRCP recommended by the AASHTO Guide are more adequate than the previously used value of 2.2, but still no known justification or verification exists.

3. Joints - Construction joints and terminal anchorage systems are critical factors in CRCP, and many failures have occurred at these locations. The 1986 Guide provides general guidance for the design of construction joints but no guidance for the terminal anchorage system.

4. Reinforcement Design - The reinforcement design procedure for CRCP recommended by the AASHTO Guide considers more important factors in the equations than the previous version. The corrosion of the reinforcement in freeze regions where deicing salts are used was not considered in the design. The direct consideration of friction factor between base and slab is not possible.

3.8.2 Analytical Evaluation.

The analytical evaluation was conducted using the specific design evaluation approach. A number of pavement design situations were developed for CRCP for the wet-freeze zone, which is the predominant climatic zone in the Illinois CRCP database. The design factors included only a fine-grained subgrade soil, a 4 in (10.2 cm) bituminous aggregate mixture (BAM) and a 6 in (15.2) granular subbase, and a J factor from that recommended for many years (J = 2.2) to that currently recommended (J = 3.2). Specific soil, subbase, concrete properties, climatic design inputs, etc. for the designs are shown in table 49. The traffic levels for 20 year design was 5 and 15 million 18-kip ESAL. Different levels of reliability were used from 50 to 90 percent. CRCP was then designed for these different conditions using the new AASHTO Guide.

The CRCP predictive model developed from the Illinois database was used to predict the extent of deterioration in terms of failures per mile. Failure rates over 10 per mile are indicative of severe maintenance requirements. A reasonable value for design may be about 5 per mile.[63] The predictions for CRCP with varied J values are shown in tables 50 and 51.

Table 49.

Design input parameters for AASHTO performance equation for CRCP.

Parameter	CRCP design
Reliability level, %	50/80/90
Design period, year	20
Traffic, million 18-kip ESAL	5/15
* Subgrade soil	fine-grained
** Subbase type	4 in BAM / 6 in granular
k-value @ top of subbase, pci	280/200
Initial serviceability	4.5
Terminal serviceability	2.5
*** Modulus of rupture, psi	650
Concrete E value, psi	4,000,000
J factor	3.2/2.6/2.2
**** C_d coefficient	0.85
LS factor	1.0

* Subgrade M_R = 3,000 psi for fine-grained soil and 7,000 for coarse-grained soil

** Subbase E = 650,000 psi for BAM and 30,000 psi for granular

*** Third-point loading, at 28 days

**** Wet-Freeze region

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 50.

Summary of results for CRCP designs using AASHTO Guide (J=3.2).

18-kip ESAL (million)	Design Reliability (%)	Subbase Type	Slab Thickness (inch)	Reinforcement (Area of Steel)* min.	max.	Fail per Mile ** use max. Rein.	use min. Rein.	Required Rein. Determined by CRCP Model ***	Does the Design Provide Enough Reinforcement?
USE #6 BAR									
15	50	granular	10.3	0.044	0.059	0.7	8.5	0.048	yes
15	50	bam	10.2	0.045	0.060	1.9	9.1	0.051	yes
15	80	granular	11.2	0.052	0.069	-0.9	1.4	0.044	yes
15	80	bam	11.1	0.053	0.075	0.4	2.6	0.048	yes
15	90	granular	11.7	0.058	0.075	-1.2	-0.2	0.043	yes
15	90	bam	11.6	0.058	0.076	0.0	1.2	0.046	yes
5	50	granular	8.7	0.030	0.041	3.2	15.9	0.038	yes
5	50	bam	8.6	0.030	0.042	3.1	16.8	0.039	yes
5	80	granular	9.5	0.037	0.050	0.7	3.7	0.035	yes
5	80	bam	9.4	0.038	0.051	0.8	3.5	0.036	yes
5	90	granular	9.9	0.041	0.055	0.3	1.7	0.033	yes
5	90	bam	9.8	0.041	0.055	0.4	2.0	0.034	yes
USE #5 BAR									
15	50	granular	10.3	0.04	0.054	2.0	14.7	0.048	yes
15	50	bam	10.2	0.041	0.054	3.5	14.7	0.051	yes
15	80	granular	11.2	0.047	0.063	-0.5	3.5	0.044	yes
15	80	bam	11.1	0.048	0.063	0.9	4.4	0.048	yes
15	90	granular	11.7	0.052	0.069	-1.0	0.9	0.043	yes
15	90	bam	11.6	0.053	0.069	0.3	2.1	0.046	yes
5	50	granular	8.7	0.026	0.037	5.4	32.7	0.038	no
5	50	bam	8.6	0.026	0.037	5.9	34.5	0.039	no
5	80	granular	9.5	0.033	0.045	1.3	6.7	0.035	yes
5	80	bam	9.4	0.033	0.046	1.3	7.1	0.036	yes
5	90	granular	9.9	0.036	0.049	0.6	3.5	0.033	yes
5	90	bam	9.8	0.037	0.05	0.7	3.3	0.034	yes

* Area of steel, square inches of steel per inch width of PCC slab.

** Fail = number of edge punchouts plus number of steel ruptures plus number of patches.

*** Based on criterion of 5 failures per mile.

Design period: 20 years
Climatic region: wet freeze

A value of J = 3.2 was used in the structural design equation.

1 in = 2.54 cm

Table 51. Summary of results for CRCP designs using AASHTO Guide (J = 2.2).

18-kip ESAL (million)	Design Reliability (%)	Subbase Type	Slab Thickness (inch)	Reinforcement (Area of Steel) * min.	max.	Fail per Mile ** use max. Rein.	use min. Rein.
USE #6 BAR							
15	50	granular	8.4	0.027	0.038	49.0	277.9
15	50	bam	8.3	0.028	0.039	46.4	245.0
15	80	granular	9.2	0.034	0.047	10.2	58.2
15	80	bam	9.1	0.035	0.048	10.8	53.9
15	90	granular	9.7	0.039	0.052	4.1	22.4
15	90	bam	9.6	0.039	0.053	5.1	24.8
5	50	granular	7	0.015	0.023	153.6	1303.0
5	50	bam	6.9	0.015	0.023	163.5	1385.9
5	80	granular	7.8	0.022	0.031	21.6	120.7
5	80	bam	7.6	0.022	0.031	24.3	135.0
5	90	granular	8.1	0.024	0.035	9.9	66.4
5	90	bam	8	0.025	0.035	10.6	57.2
USE #5 BAR							
15	50	granular	8.4	0.023	0.034	86.6	621.6
15	50	bam	8.3	0.024	0.034	92.6	529.9
15	80	granular	9.2	0.03	0.042	19.2	110.2
15	80	bam	9.1	0.031	0.043	19.0	99.2
15	90	granular	9.7	0.034	0.047	7.8	46.1
15	90	bam	9.6	0.035	0.048	8.5	42.8
5	50	granular	7	0.012	0.02	309.1	3976.8
5	50	bam	6.9	0.013	0.02	328.8	2834.4
5	80	granular	7.8	0.018	0.028	36.0	329.5
5	80	bam	7.6	0.018	0.027	48.5	368.4
5	90	granular	8.1	0.021	0.031	18.3	129.6
5	90	bam	8	0.021	0.031	19.5	136.8

* Area of steel, square inches of steel per inch width of PCC slab.

** Fail = no. of edge punchouts + no. of steel ruptures + no. of patches.

Design period: 20 years

Climatic region: wet - freeze.

A value of 2.2 for J factor was used in the structural design equation.

1 in = 2.54 cm

1. Structural design: For the two values of J factor, the required slab thickness for CRCP with BAM subbase is about the same as (or 0.1 in (0.25 cm) less than) that with granular subbase. BAM and granular subbases give about the same amount of failures per mile, according to the predictive model.

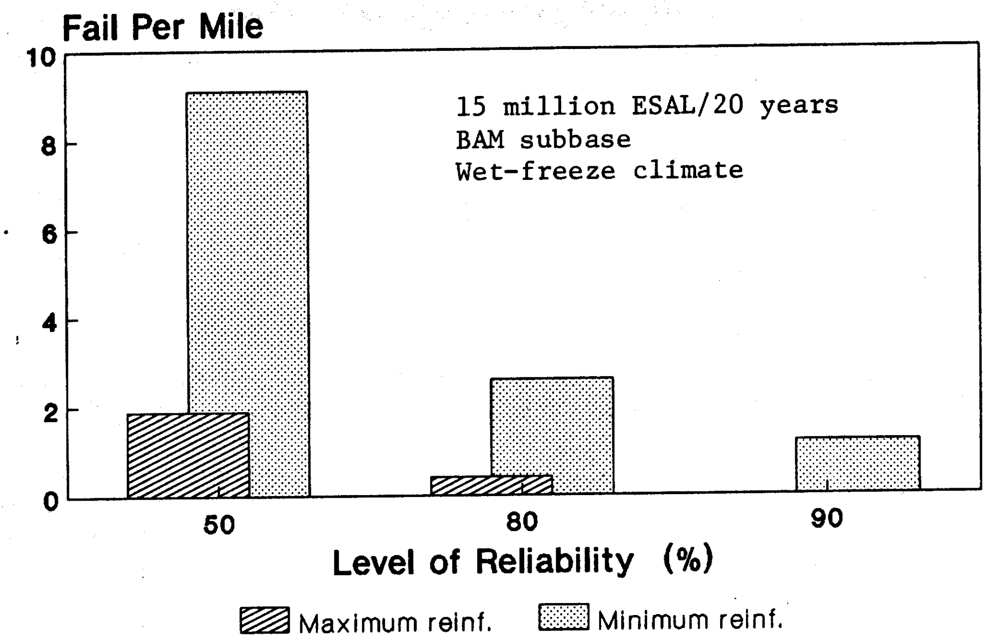
The required slab thickness for the J=2.2 is 1.7 to 2 in (4.3 to 5.1 cm) less than that for J=3.2. However, there are many failures per mile when J=2.2, which was used in design for many years. The high number of failures reflect the typical high failure rate for 7-8 in (17.8-20.3 cm) CRCP in many States over the past 20 years.

The level of design reliability has a very large effect on the failures per mile as expected. In Table 50, all of the designs have acceptable performance, except those using minimum reinforcement at the 50 percent level of reliability. In general, the results show that the Guide designs provide adequate structural designs for CRCP when J=3.2, which is recommended by the new AASHTO Guide. The Illinois CRCP predictive model only considers the condition where the pavements are without tied concrete shoulders, since all the pavement sections are with AC shoulder (edge loading condition) in the database.

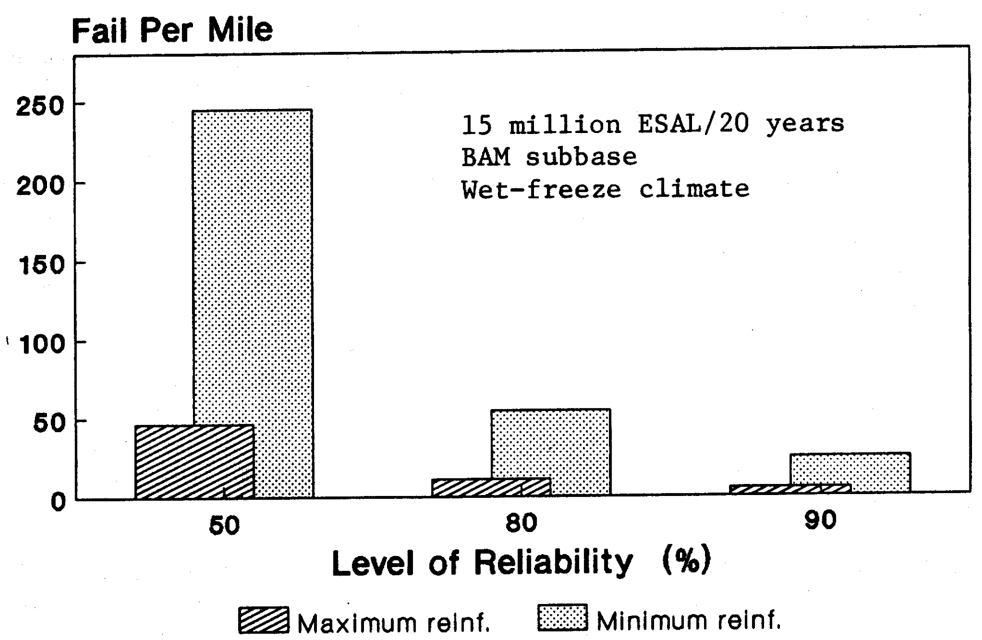
The results also show the failures per mile for CRCP for the lower level of traffic (5 million ESAL) is larger than that for the higher one (15 million ESAL).

2. Reinforcement design: For J=3.2 and all three levels of reliability, the AASHTO Guide gives the value of minimum and maximum reinforcement for each design from 0.044 and 0.059 in^2/in (or 0.43 and 0.57 percent of steel) to 0.058 and 0.076 in^2/in (or 0.5 and 0.66 percent of steel) for designs with 15 million ESAL traffic and #6 bars, and from 0.03 and 0.041 in^2/in (or 0.34 and 0.47 percent of steel) to 0.041 and 0.055 in^2/in (or 0.42 and 0.56 percent of steel) with 5 million traffic ESAL and #6 bars. The results show that area of reinforcement is very critical to the performance of the designs. The maximum and minimum values of reinforcement are recommended to control crack spacing, but the minimums give much higher failure rates. Figure 19 shows the failure rates vs. reliability level with different J values. The area of reinforcement for using #5 bars is less than that using #6 bars because the smaller bars provide more bond area, thus, more effectively reduce the crack width of pavements. However, the predictive model does not take the steel bond area into consideration.

The required area of steel for edge loading condition (i.e. J=3.2) to overcome the 5 failures per mile (3.1 failures/km) performance criteria was determined by the Illinois predictive model for each design situation. The AASHTO reinforcement designs were then compared to those values from the predictive model. The results given in table 50 show that the ranges of minimum and maximum of AASHTO reinforcement design cover the required area of steel from the predictive model at the 50 percent of reliability level. Furthermore, the AASHTO Guide designs provide increasingly greater reinforcement than the required values when the reliability level is 80 percent or more. The overall results show that the Guide designs generally provide adequate reinforcement designs for CRCP for the edge loading condition for design with a reliability of 80 percent or greater, and that the minimum reinforcement level may be slightly too low.



(a) J = 3.2



(b) J = 2.2

Figure 19. CRCP failures per mile vs. reliability level for AASHTO designs.

3.9 RPS-3 Texas SDHPT Procedure for CRCP

The RPS-3 procedure for CRCP design employs the AASHO Road Test performance model which was originally developed for jointed concrete pavements. The reinforcement design model is taken from the final report of NCHRP Project 1-11.[61]

3.9.1 Conceptual Evaluation

Structural Design - The structural design equation for CRCP in the RPS-3 procedure is exactly the same as for JRCP. The only difference between CRCP and JRCP design is the use of the joint continuity factor (or J factor) which is 2.2 for CRCP instead of 3.2 for JRCP. The user can specify or input the type of pavement for CRCP design only, or for both JRCP and CRCP. Reliability concepts included in RPS-3 are similar to the AASHTO Guide (but not exactly the same).

Joint Design - The RPS-3 procedure mentions construction and warping joints for CRCP. But no guidance is provided with regard to the design of construction joint or warping joint in the available reports.

Reinforcement Design - The longitudinal reinforcement in CRCP is designed using the following equation (from RPS-1 version):

$$A_s = 12\ D\ (1.3 - 0.2\ Fa\ )\ Ts\ /\ fs \qquad (28)$$

where Ts is the tensile strength of concrete, in psi. Other terms are as previously defined in section 3.6. The working reinforcement stress is taken to be 0.75 times the yield point strength. The procedure suggests a minimum area of steel for CRCP of 0.4 percent. The transverse reinforcement requirement, if provided, is determined by the same equation 23 used for JRCP. Either deformed rebar steel or welded wire mesh steel plus tie bars is available in the design procedure.

The design of overlays and the cost analysis are basically identical to that for JRCP.

Specific limitations of the RPS-3 CRCP procedure are summarized as follows:

1. General - As the RPS-3 CRCP procedure employs the AASHO Road Test results which were originally developed for jointed concrete pavements, the same limitations previously presented for the 1986 AASHTO Guide for JRCP and CRCP (see section 3.2) are applicable. The applicability of the equations to CRCP developed from jointed pavements at the AASHO Road Test has never been verified, and has some obvious, serious problems. These problems include the use of corner stress for thickness design, a slab with closely spaced cracks with higher deflections and greater flexibility, and the larger amount of reinforcement. These have a very large effect on performance.

2. Joint continuity factor - The joint continuity factor value of 2.2 and has been used in design for many years.[62] No known justification or verification of this value exists.

3. Joints - Construction and terminal joints are very important in CRCP, and many failures have occurred at such locations. No guidance was found in the procedure with regard to the design of a construction joint or the terminal anchorage system.

4. Reinforcement Design - The expression given by equation 25 is a greatly simplified estimation of required steel percentage. Some of the important factors which have the effect on the crack spacing and width were not considered. The corrosion of the reinforcement in freeze regions where deicing salts are used was not considered in the design.

5. No provisions are provided for the design of shoulders or a drainage system.

Based on these limitations, it is doubtful that the RPS-3 procedure will provide an adequate guide for pavement design for various national climatic conditions.

3.9.1 Analytical Evaluation.

A number of CRCP designs were developed for the wet-freeze zone using the RPS-3 program and the inputs are shown in table 52. The design factors varied included 4 in (10.2 cm) of bituminous aggregate material (BAM) versus 6 in (15.2 cm) granular subbase, and #5 and #6 deformed rebar steel reinforcement. The traffic levels for 20 year design were 5 and 15 million 18-kip ESAL, and different levels of reliability were used from 50 to 95 percent. CRCP was then designed for these different conditions using the RPS-3 program. No overlays were allowed in this evaluation.

The CRCP predictive model developed from the Illinois database was used to predict the extent of deterioration in terms of failures per mile. A reasonable value of 5 failures per mile was used as the performance criterion for design. The predictions for CRCP designs are shown in table 53.

1. Structural design. The program rounded up the value of slab thickness to the nearest 1/2 in (1.3 cm). The required slab thickness for CRCP with BAM subbase is generally 1/2 in less than that with granular subbase. BAM and granular subbases give about the same amount of failures per mile.

The required slab thickness for CRCP using RPS-3 for J=2.2 is approximately 0 (at 50 percent reliability level) to 1/2 inches (at 80 percent level) greater than that using 1986 AASHTO Guide (see table 50). However, there are far fewer failures per mile for RPS-3 designs due to the contribution of the greater area of reinforcement.

The level of design reliability does not have a large effect on the failures per mile because of the relatively low failures per mile at all

Table 52.

Design input parameters for RPS-3 procedure for the CRCP designs.

Parameter	CRCP design
Reliability level, %	50/80/95
Design period, years	20
Traffic, million 18-kip ESAL	5/15
* Subgrade soil	fine-grained
** Subbase type	4 in BAM / 6 in granular
k-value @ top of subbase, pci	280/200
Initial serviceability	4.5
Terminal serviceability	2.5
*** Modulus of rupture, psi	650
Concrete E value, psi	4,000,000
J factor	2.2
Erodability factor	1.0/0.5

* Subgrade M_R = 3,000 psi for fine-grained soil

** Subbase E = 650,000 psi for BAM and 30,000 psi for granular

*** Third-point loading, at 28 days

Wet-Freeze region

1 in = 2.54 cm

1 ft = 0.3048 m

1 psi = 0.07031 kg/cm^2

1 psi/in = 0.02768 kg/cm^3

Table 53. Summary of results for CRCP designs using RPS-3 procedure.

18-kip ESAL (million)	Design Reliability (%)	Subbase Type	Slab Thickness (inch)	Reinforcement (Area of Steel) *	Fail ** per mile	Required Reinf. Determined by CRCP Model ***	Does The Design Provide Enough Reinforcement?
USE #6 BAR							
15	50	granular	8.5	0.068	1.0	0.057	yes
15	50	bam	8.0	0.061	5.8	0.062	no
15	80	granular	9.5	0.076	-0.7	0.052	yes
15	80	bam	9.5	0.071	1.0	0.054	yes
15	95	granular	11.0	0.088	-1.4	0.045	yes
15	95	bam	10.5	0.079	0.2	0.050	yes
5	50	granular	7.0	0.056	1.6	0.045	yes
5	50	bam	7.0	0.053	2.4	0.045	yes
5	80	granular	8.5	0.068	0.1	0.039	yes
5	80	bam	8.0	0.061	0.7	0.041	yes
5	95	granular	9.5	0.076	-0.1	0.035	yes
5	95	bam	9.0	0.068	0.2	0.037	yes
USE #5 BAR							
15	50	granular	8.5	0.068	1.0	0.057	yes
15	50	bam	8.0	0.060	6.0	0.062	no
15	80	granular	9.5	0.077	-0.8	0.052	yes
15	80	bam	9.5	0.071	1.0	0.054	yes
15	95	granular	11.0	0.088	-1.4	0.045	yes
15	95	bam	10.5	0.079	0.2	0.050	yes
5	50	granular	7.0	0.056	1.6	0.045	yes
5	50	bam	7.0	0.053	2.3	0.045	yes
5	80	granular	8.5	0.068	0.1	0.039	yes
5	80	bam	8.0	0.060	0.7	0.041	yes
5	90	granular	9.5	0.077	-0.1	0.035	yes
5	90	bam	9.0	0.068	0.2	0.037	yes

* Area of steel, square inches of steel per inch width of PCC slab.

** Fail = no. of edge punchouts + no. of steel ruptures + no. of patches.

*** Based on 5 failures per mile criterion.

Design period: 20 years.

Climatic region: wet - freeze.

1 in = 2.54 cm

reliability levels. Table 53 shows that both traffic levels give about the same amount of failures per mile.

In general, the results show that the RPS-3 design procedure provides adequate structural designs for CRCP for a wet-freeze zone.

2. Joint and terminal system design. The program requires the construction joints for CRCP and gives a spacing of 2640 ft (805 m). No further recommendations are provided in the procedure for the design of construction joints or a terminal system.

3. Reinforcement design. The RPS-3 program gives the resulting reinforcement from 0.068 in^2/in (or 0.8 percent steel) to 0.079 in^2/in (or 0.75 percent) for designs with 15 million ESAL traffic and #6 bars, and from 0.056 in^2/in (or 0.8 percent) to 0.068 in^2/in (or 0.76 percent) with 5 million traffic and #6 bars. The results show that percent of steel is larger than that currently used by various highway agencies (typical values 0.5 to 0.7 percent as well as the average value 0.63 percent from Illinois database).[63] The area of reinforcement for using #5 bars is almost the same as that using #6 bars since the bond area effect is not considered in the design.

The required amount of reinforcement to overcome the 5 failures per mile performance criterion was determined by the Illinois predictive model for each design situation. The amount of reinforcement designed using the RPS-3 procedure was then compared to those values calculated from the predictive model. The results given in table 53 show that the the RPS-3 reinforcement designs generally provide greater area of steel value than those calculated from the predictive model. It shows that the RPS-3 procedure provides adequate reinforcement design for CRCP.

3.10 Associated Reinforcing Bar Producers-CRSI Procedure for CRCP

The Associated Reinforcing Bar Producers-Concrete Reinforcing Steel Institute (ARBP-CRSI) design manual for CRCP was published in 1981.[66] It contains the design procedure which was first introduced in the Second International Conference on Concrete Pavement Design at Purdue University in 1981.[67] The thickness design equation is similar to the AASHTO Interim Guide which was developed from the AASHO Road Test jointed concrete results but with a few changes.[36] However, the reinforcement design procedure is the same as employed in the 1986 AASHTO Guide.[39]

3.10.1 Conceptual Evaluation.

Structural Design - The structural design equation used in the ARBP-CRSI procedure was derived from the results of the AASHO Road Test and revised by the inclusion of the corner stress model of Spangler. The design equation is as follows:

$$\begin{aligned}\log W_{18} = {} & 7.35 \log(D+1) - 0.06 - \\ & 0.176 / [\ 1 + (1.624*10^7) / (D+1)^{8.46}\ ] + \\ & 3.42 \log [\ (\ fw / 215.6\ J\ ) * \\ & (\ D^{.75} - 1.132\ ) / (\ D^{.75} - 18.42/(Ec/k)^{.25})]\end{aligned} \tag{29}$$

where W_{18} = number of 18-kip equivalent single axle load applications to a terminal serviceability index of 2.5

D = thickness of pavement slab, inches

fw = working stress in concrete, psi

J = load transfer coefficient
- = 3.2 for edge load
- = 2.56 for interior load

Z = E/K_d

E = concrete modulus of elasticity, psi

K_d = design k-value, pci

The ARBP-CRSI procedure recommends the value of J factor which is 3.2 for the CRCP without tied concrete shoulders (edge loading condition) and 2.56 for the CRCP with tied concrete shoulders or the lane is in the interior of pavement (interior loading condition). The effect of a lower J factor is about 1 in reduction in slab thickness due to approximately 20 percent reduction in working stress in the concrete slab. The design k-value (K_d) which is at the top of the subbase is based on deflection and is determined using a chart given in the manual. A correction is applied to the k-value on top of the subbase to consider the erodibility potential of the subbase materials. Climatic variables and the reliability concept were not considered in this structural design equation. A greater than 50 percent reliability is introduced through the reduction of the modulus of rupture to a "working stress."

Joint Design - Longitudinal joints and transverse construction joints are recommended in the procedure. Considerable information is provided by the procedure on those types of joints. The procedure recommends that a minimum of 1 percent longitudinal steel be provided across the transverse construction joints. The procedure also recommends placing the additional 6-ft long bar, of the same size as the longitudinal bars, between every other longitudinal reinforcing bar. The amount of longitudinal reinforcement is therefore increased 50 percent across the construction joints. Both the anchorage system and the wide flange expansion joint are recommended in the procedure as the CRCP terminal system to restrain or allow for movement of the free end of the CRCP slab. Coating for the wide flange beam is suggested in the corrosive areas near the ocean or where deicing salts are used extensively. No recommendation is provided for the sealant for the wide flange beam joints.

Reinforcement Design - The design of longitudinal reinforcement is the same as employed in the 1985 AASHTO Guide.[39] Only deformed steel bars are recommended in the procedure for design. A set of worksheets and nomographs for use to solve the required reinforcement is provided in the ARBP-CRSI procedure.[66] The design for transverse steel in CRCP, if provided, is the same as used in 1986 AASHTO Guide.

Cost Analysis - A simple cost analysis is provided in the manual for the designer to select the most economical combination of subbase type and thickness, and CRCP slab thickness appropriate to conditions.

Drainage Design - General recommendations for the consideration of drainage for CRCP is given in the procedure, however, detailed guidelines for the design of drainage are not provided.

Specific limitations of the design procedure are summarized as follows:

1. General - Because the ARBP-CRSI procedure uses the same performance equation from the AASHO Road Test, nearly all limitations previously presented for the AASHTO Guide for design of jointed concrete pavements (see section 3.2) and CRCP (see section 3.8) are applicable.

2. Joint Continuity Factor - The joint continuity factor values for CRCP recommended by the ARBP-CRSI procedure are more nearly accurate than the previous widely used value of 2.2, but still have no known justification or verification.

3. Joints - Construction joints and terminal anchorage systems are critical factors in CRCP because many failures have occurred at these locations. The ARBP-CRSI procedure provides recommendations for construction joints and terminal anchorage systems. These recommendations for joint design in the procedure are based on experience. No analytical design procedure is available to allow the designer to design for specific project conditions.

4. Reinforcement Design - The reinforcement design recommendations are the same as provided by the 1986 AASHTO Guide, hence, similar limitations apply (see section 3.2).

5. Shoulders - No recommendations are provided in the manual for shoulder design.

3.10.2 Analytical Evaluation.

A number of pavement design situations were developed for CRCP for the wet-freeze zone, which is the predominant climate in the Illinois CRCP database. The design factors included only a fine-grained subgrade soil, 4 in (10.2 cm) bituminous aggregates mixture (BAM) and 6 in (15.2 cm) granular subbases, and a J factor of 3.2 (edge loading condition). Specific soil, subbase, concrete properties, etc. for the designs are shown in table 54. The traffic levels for a 20-year design period were 5 and 15 million 18-kip ESAL. CRCP was then designed for these different conditions using the ARBP-CRSI procedure.

The CRCP predictive model developed from the Illinois database was then used to predict the extent of deterioration in terms of failures per mile. Failure rates over 10 per mile are indicative of severe maintenance requirements. A reasonable value for design is 5 failures per mile (3.1 failures/km). The predictions for CRCP with both J values are shown in table 55.

1. Structural design. The required slab thickness for CRCP with BAM subbase is about the same as (or 0.1 in (0.25 cm) less than) that with granular subbase. BAM and granular subbases give about the same amount of failures per mile.

Table 54.

Design inputs for the ARBP-CRSI procedure for CRCP design.

Parameter	CRCP design
Design period, year	20
Traffic, million 18-kip ESAL	5/15
* Subgrade soil	fine-grained
** Subbase type	4 in BAM / 6 in granular
k-value @ top of subbase, pci	210/160
Initial serviceability	4.5
Terminal serviceability	2.5
*** Modulus of rupture, psi	650
Concrete E value, psi	4,000,000
J factor	3.2/2.56
LS factor	1.0/1.0

* Subgrade M_R = 3,000 psi for fine-grained soil, and 7,000 psi for coarse-grained soil

** Subbase E = 650,000 psi for BAM and 30,000 psi for granular

*** Third-point loading, at 28 days

Wet-freeze climate

1 in = 2.54 cm

1 ft = 0.3048 m

1 psi = 0.07031 kg/cm^2

1 psi/in = 0.02768 kg/cm^3

Table 55. Summary of results for CRCP designs using ARBP-CRSI procedure.

18-kip ESAL (million)	Subbase Type	Slab Thickness (inch)	Reinforcement (Area of Steel) * min.	Reinforcement (Area of Steel) * max.	Fail per Mile ** use max. Rein.	Fail per Mile ** use min. Rein.	Required Reinf. Determined by CRCP Model ***	Does the Design Provide Enough Reinforcement?
USE #6 BAR								
15	granular	9.5	0.037	0.050	5.9	32.5	0.051	no
15	bam	9.4	0.037	0.050	7.6	35.4	0.054	no
5	granular	8.0	0.023	0.033	14.1	86.7	0.040	no
5	bam	7.9	0.023	0.033	15.0	91.6	0.041	no
USE #5 BAR								
15	granular	9.5	0.032	0.045	11.2	69.0	0.051	no
15	bam	9.4	0.033	0.045	13.1	63.0	0.054	no
5	granular	8.0	0.020	0.029	27.1	174.5	0.040	no
5	bam	7.9	0.020	0.029	28.7	184.3	0.042	no

* Area of steel, square inches of steel per inch width of PCC slab.
** Fail = no. of edge punchouts + no. of steel ruptures + no. of patches.
*** Based on 5 failures per mile criterion.
Design period: 20 years.
Climatic region: wet - freeze.
A value of 3.2 for J factor was used in the structural design equation.

1 in = 2.54 cm

Table 55 shows that the ARBP-CRSI designs with J=3.2 have many failures per mile even using the maximum amount of reinforcement. Compared to the structural design presented for the 1986 AASHTO Guide with 50 percent reliability level in table 50, the slab thickness for ARBP-CRSI designs is about 0.7 to 0.8 in (1.8 to 2.0 cm) less than that for the AASHTO designs. The results show that the ARBP-CRSI designs do not provide adequate structural designs for CRCP when J=3.2, which is recommended for the edge loading condition. The ARBP-CRSI procedure does not include climatic variables, such as the drainage coefficient in the structural design equation.

The ARBP-CRSI designs also show that the lighter traffic designs (5 million ESAL) produce more failures per mile for CRCP than the heavier traffic designs (15 million ESAL).

2. Reinforcement design: For a J-value of 3.2 and using #6 bars, the ARBP-CRSI procedure gives the value of minimum and maximum reinforcement for each design: 0.037 and 0.05 in^2/in (or 0.39 percent and 0.53 percent) for designs with 15 million ESAL traffic, and 0.023 and 0.033 in^2/in (or 0.29 percent and 0.42 percent) for designs with 5 million traffic. All the ARBP-CRSI designs have percent of steel values which are lower than the practical values ranged from 0.5 percent to 0.7 percent. The results show that area of reinforcement is very critical to the performance of the designs. Many failures per mile were predicted for ARBP-CRSI designs even with the maximum amount of reinforcement.

The required amount of reinforcement for the edge loading condition (i.e., J=3.2) to overcome the 5 failures per mile (3.1 failures per km) performance criterion was determined for each design situation by the Illinois predictive model. The ARBP-CRSI reinforcement designs were then compared to those areas of steel values obtained from the predictive model. The results given in table 55 show that the values of area of steel for the ARBP-CRSI reinforcement designs, with either the minimum or the maximum amount of reinforcement, are less than those obtained from the predictive model. The ARBP-CRSI designs do not seem to provide adequate amounts of reinforcement for CRCP according to these results.

The area of reinforcement using #5 bars is less than that using #6 bars because the smaller bars provide more bond area, thus more effectively reducing the crack width of pavements. However, the predictive model does not take the steel bond area into consideration.

3.11 **Illinois DOT** Procedure for CRCP

The Illinois Department of Transportation (IDOT) Design Manual was issued in 1982.[69] It contains the CRCP design procedure, which was first presented in the AASHO Road Test Conference held at St. Louis in 1962.[70,71] The structural design equation for CRCP is a modification of the AASHO Road Test equation. Also, IDOT Highway Standards are available for providing the necessary standard design for the reinforcement and other important CRCP components.[72]

3.11.1 Conceptual Evaluation.

Structural Design: The structural design equation used in the IDOT procedure was derived from the results on jointed concrete from the AASHO Road Test. The structural design of CRCP is accomplished through the use of tables and nomographs. Equations from which the tables and nomographs were derived are not given in the manual. The derivation, although not given in the manual, is identically the same as used in the AASHO Interim Guide.[36] However, a "Time-Traffic Exposure Factor" was incorporated into the original design nomographs in the IDOT procedure to adjust for long-term climatic effects. Slab thickness is increased about 10-15 percent. Also, from the thickness scale of the nomograph, the required slab thickness for CRCP is approximately 20 percent less than that for jointed concrete pavements to account for the perceived benefit of increased steel and omission of transverse joints.

The IDOT procedure assumes the material requirements, mixture designs, and construction procedures and controls are within the IDOT specifications. The inputs for the CRCP design contain the classification of roads, the value of Traffic Factor (ESAL in millions) and the Illinois Bearing Ratio (IBR, similar to CBR), or k-value, for subgrade soil support. The concrete properties are assumed equal to the AASHO Road Test and are built into the design nomographs. Traffic Factor (TF) is the projected number of total 18-kip ESAL in millions in the design lane for the 20-year design period. The Traffic Factor for the class I roads is calculated using the following equation:[1]

$$TF = 20 \left[\frac{(0.15\ P\ PV) + (116.07\ S\ SU) + (549.33\ M\ MU)}{1000000}\right] \qquad (30)$$

where P, S, M = percentage of PV, SU, and MU in the design lane
PV, SU, MU = structural design traffic (two-directional), expressed as the number of daily Passenger Vehicles, Single Units and Multiple Units

The IDOT procedure gives the provisions of the minimum structural design thicknesses for PCC slab and subbase for each class of pavements. The minimum slab thicknesses for the class I roads are currently 9 in (23 cm) for Interstate and other freeways and 8 in (20 cm) for all others. The minimum 4 in (10.2 cm) of stabilized granular subbase is required for all the Class I roads. This includes roads and streets designed as a facility with four- or more lanes, or as part of a future facility with four- or more lanes, and one-way streets with a structural design traffic greater than 3500 ADT.[69]

Joint Design - Typical sections for the longitudinal, expansion, construction and wide flange beam terminal joints are provided for CRCP in the IDOT Standard.[72] The Standards recommends that a 2-in (5 cm) expansion joint should be used instead of the wide flange beam terminal joint when the slab length is less than 1500 ft (457 m). Silicone joint sealant and polyethylene tape are recommended for use at the wide flange beam terminal joint. Anchor lugs are not recommended for the CRCP terminal system in the Standards.

Reinforcement Design - The IDOT Standards provides the tables and lap patterns for the reinforcement design for CRCP.[72] No equations are given in the manual with regard to the derivation of the tables for the required amount of reinforcement for CRCP. Both deformed bars and steel fabric reinforcement are used for CRCP in the Standards.

Drainage Design - The IDOT procedure provides general guidelines and typical sections for the pavement drainage design. General information for the design of underdrains is provided in the IDOT Manual and Standards.

Specific limitations of the IDOT procedure are summarized as follows:

1. General - The IDOT procedure employed the AASHO Road Test performance equation. Nearly all limitations previously presented for the AASHTO Guide for design of jointed concrete pavements and CRCP are applicable (see sections 3.2.1 and 3.3.2).

2. Joint Continuity Factor - The AASHO Road Test structural design equation which is employed by the IDOT procedure uses the Joint Continuity Factor of 2.2 for the design of CRCP. This results in a slab about 80 percent of that for jointed concrete. No known justification or verification of this value exists.

3. Joints - The IDOT Standards provides the typical sections for the expansion, construction, and wide flange beam terminal joints design. These joints design standards in the IDOT procedure are based on experience. No analytical design procedure is available for the designing of specific project conditions.

4. The beneficial effect of subbase support is basically ignored since the k-value used in design is that of the subgrade.

5. Reinforcement Design - The design of reinforcement in the IDOT procedure is accomplished by using the table which contains only the bar size and slab thickness as the inputs. The design procedure is so simplified that many important factors are not considered in this table, such as the effect of friction between slab and subbase, strength of concrete, etc. No recommendation is provided for steel anticorrosion design in freeze regions where large amounts of deicing salt are used each winter.

3.11.2 Analytical Evaluation.

A number of CRCP designs were generated for the wet-freeze climatic zone using the IDOT procedure. The inputs are shown in table 56. The various design factors included coarse-grained (CBR of 20) and fine-grained

Table 56.

Design inputs for the IDOT procedure for CRCP design.

Parameter	CRCP design
Road classification	Class I/Class II
Design period, year	20
Traffic Factor	5/15/30
Subgrade soil	fine-/coarse-grained
CBR for subgrade	3/20
Subbase type	4 in BAM / 4 in CAM

Climatic region: wet-freeze

1 in = 2.54 cm

(CBR of 3) subgrade soils, 4 in (10.2 cm) of bituminous aggregates material (BAM) and cement aggregate material (CAM), #6 deformed bars and 4x12 D-22xD-5 steel fabric reinforcement. The Traffic Factors (millions of ESAL's) for 20-year design were 5 (for Class II roads), 15 and 30 (both for Class I roads).

The CRCP predictive model developed from the Illinois database was used to estimate the extent of deterioration in terms of failures per mile. A reasonable value of 5 failures per mile (3.1 failures/km) was chosen as the limiting performance criterion value for design. The predictions for CRCP designs are shown in table 57.

1. Structural design: The required slab thickness for CRCP with coarse-grained subgrade soils (CBR=20) is about 0 to 1/2 in (0-1.3 cm) less than that with fine-grained (CBR=3). Both subgrade soils give nearly the same number of failures per mile. The required slab thickness for CRCP with BAM subbase is the same as that with CAM. However, the CAM results in many more predicted failures per mile. Table 57 shows that the designs with BAM subbases give acceptable performance.

The results show that heavier traffic results in more failures per mile even though the design slab thickness is also increased. The IDOT procedure provides better performance at lower traffic levels. In general, the results show that the IDOT designs provide sufficient structural adequacy for CRCP in wet-freeze zone with BAM subbase, but not with CAM.

2. Reinforcement design. For the deformed bar reinforcement, the IDOT procedure gives the amount of steel from 0.058 in^2/in (or 0.73 percent) for ESAL's of 5 million, to 0.071 in^2/in (or 0.68 percent) for ESALs of 30 million. The percent of steel of the CRCP designs are at the higher side of the normal value which range from 0.5 percent to 0.7 percent.[66] The area of reinforcement for #6 deformed bars is a little greater than that for 4x12 steel fabric. The steel fabric produces more failures per mile than do bars.

The required amount of reinforcement with BAM subbase to overcome the 5 failures per mile (3.1/km) performance criterion was calculated with the Illinois predictive model for each design situation. The amount of reinforcement designed using the IDOT procedure was then compared to the values calculated from the predictive model. Table 57 shows that the amount of reinforcement required according to the IDOT procedure is greater than that required by the predictive model except for the design with steel fabric and ESAL of 30 million. No feasible solution for the reinforcement design was obtained from the predictive model for CRCP with CAM that could overcome the 5 failures per mile criterion. The IDOT designs with BAM subbase generally provide an adequate amount of reinforcement for CRCP. Thus, through experience the Illinois DOT has developed thickness and reinforcement design procedures and standards that provide adequate performance.

Table 57. Summary of results for CRCP designs using IDOT procedure.

18-kip ESAL (million)	Subgrade IBR Value	Subbase Type	Slab Thickness (inch)	Reinforcement (Area of Steel) *	Fail ** Per Mile	Required Reinf. Determined by CRCP Model ***	Does the Design Provide Enough Reinforcement?
USE #6 BAR							
30	3	cam	10.5	0.071	605.6	- ****	-
30	3	bam	10.5	0.071	2.0	0.063	yes
30	20	cam	10.25	0.071	606.0	-	-
30	20	bam	10.25	0.071	2.4	0.064	yes
15	3	cam	9.5	0.064	157.6	-	-
15	3	bam	9.5	0.064	1.9	0.054	yes
15	20	cam	9	0.064	158.1	-	-
15	20	bam	9	0.064	2.4	0.056	yes
5	3	cam	8	0.058	19.0	-	-
5	3	bam	8	0.058	0.8	0.041	yes
5	20	cam	8	0.058	19.0	-	-
5	20	bam	8	0.058	0.8	0.041	yes
USE 4x12 FABRIC							
30	3	cam	10.5	0.070	612.3	-	-
30	3	bam	10.5	0.070	8.7	-	-
30	20	cam	10.25	0.070	612.7	-	-
30	20	bam	10.25	0.070	9.1	-	-
15	3	cam	9.5	0.065	159.0	-	-
15	3	bam	9.5	0.065	3.3	0.058	yes
15	20	cam	9	0.065	159.5	-	-
15	20	bam	9	0.065	3.9	0.060	yes
5	3	cam	8	0.055	19.4	-	-
5	3	bam	8	0.055	1.3	0.041	yes
5	20	cam	8	0.055	19.4	-	-
5	20	bam	8	0.055	1.3	0.041	yes

* Area of steel, square inches of steel per inch width of PCC slab.
** Fail = no. of edge punchouts + no. of steel ruptures + no. of patches.
*** Based on 5 failures per mile criterion.
**** No solution was obtained from the CRCP model with such input.
Design period: 20 years.
Climatic region: wet - freeze.
IDOT procedure requires a minimum design slab thickness of 8 in.

1 in = 2.54 cm

4.0 SIGNIFICANCE OF DESIGN FACTORS AND OVERALL EVALUATION OF MODELS AND METHODS

This section describes design factors which affect rigid pavement performance, and then rates the ability of the models and methods to directly consider the key design factors. The final recommendations based on these results are summarized in section 7.0.

4.1 Significance of Design Factors

A comprehensive list of rigid pavement design factors was prepared. The factors were categorized under the following main headings:

- PCC slab
- Base/subbase
- Subgrade
- Shoulder/edge support and curb/gutter
- Joints
- Slab moisture and thermal factors
- Drainage system
- Climate
- Traffic loadings
- Reliability of design
- Costs

A list of the key rigid pavement distresses that cause a great majority of the deterioration of each pavement type was also developed as follows:

	PAVEMENT TYPE		
Key Distresses	JPCP	JRCP	CRCP
Trans. joint deter.	X	X	-
Slab cracking	X	-	-
Crack deterioration	-	X	X
Trans. joint faulting	X	X	-
Pumping	X	X	X
Foundation movement	X	X	X
Punchouts	-	-	X

Other significant distress types such as "D" cracking and reactive aggregates are material problems which are considered directly through specifications and material testing.

To determine which design factors are of critical importance, a rating matrix was prepared as shown in table 58. The ratings were made by the project staff based upon extensive experience in rigid pavement performance. The ratings are defined below:

- High - design factor has strong effect on distress.
- Medium - design factor has some effect on distress.
- Low - design factor has very little effect on distress.
- None - design factor has no effect on distress.

Table 58. Significance of design factors on distress types in rigid pavements.

DESIGN FACTORS	MAJOR DISTRESS TYPES: JOINT DETERIORATION	CRACKING (JPCP) & CRACK DETERIORATION	FAULTING (JRCP & CRCP)	PUMPING	PUNCHOUTS (CRCP ONLY)	FOUNDATION MOVEMENT	LEVEL OF CONSIDERATION/ IMPORTANCE
PCC SLAB							
Thickness	Low	High	Medium	Medium	High	None	Must
Length	High	High	High	Medium	None	None	Must
Width	None	High	Low	Medium	None	None	Must
Stiffness (E)	Low	High	Low	Low	Medium	None	Must
Strength	Low	High	Low	Low	Medium	None	Must
Fatigue properties	None	High	None	None	High	None	Must
Durability	High	Low	None	None	None	None	Must
Reinforcement	None	High	None	None	High	None	Must
BASE/SUBBASE							
Thickness	None	Low	None	None	Medium	Low	Should
Stiffness (E)	None	High	Medium	Medium	High	Low	Must
Strength	None	Medium	Medium	Medium	High	Low	Must
Fatigue properties	None	Medium	None	None	Low	None	Should
Durability	None	Medium	Medium	Medium	Medium	None	Should
Erodability (loss of support)	Medium	High	High	High	High	None	Must
Drainability	Medium	High	High	High	High	None	Must
Friction between slab and base	None	High	Low	Medium	High	None	Must
Structural chara. of stabilized base as seperate layer	None	High	High	High	High	None	Must
SUBGRADE							
Stiffness (E)	None	Low	None	None	Medium	None	Should
Strength	None	Low	None	None	Low	None	None
Drainability	None	Medium	High	High	High	None	Must
Moisture sensitivity	None	Low	None	Low	Low	Medium	Should
Volume change potential	None	High	None	None	None	High	Must
Characterization as springs or elastic solid	None	Low	None	None	None	None	None

Table 58. Significance of design factors on distress types in rigid pavements (continued).

DESIGN FACTORS	MAJOR DISTRESS TYPES: JOINT DETERIORATION	CRACKING (JPCP) AND CRACK DETERIORATION	FAULTING (JRCP & CRCP)	PUMPING	PUNCHOUTS (CRCP ONLY)	FOUNDATION MOVEMENT	LEVEL OF CONSIDERATION/ IMPORTANCE
SHOULDER/EDGE SUPPORT/CURB AND GUTTER (Effect on Traffic Lane)							
Type of Shoulder							
PCC (Tied)	None	Medium	Medium	Medium	Medium	None	Should
Curb and gutter (Tied)	None	Medium	Medium	Medium	Medium	None	Should
Other	None	Medium	Medium	Medium	Medium	None	Should
PCC Shoulder Design (Distress on Shoulder)							
Thickness	None	High	Low	Medium	High	None	Must
Length	Medium	High	Low	None	None	None	Must
Width	None	Medium	None	None	High	None	Must
Stiffness (E)	Low	High	None	Low	Medium	None	Should
Strength	Low	High	Low	Low	High	None	Should
Fatigue properties	None	High	None	None	High	None	Must
Durability	High	Low	None	None	None	None	Must
Reinforcement	None	High	None	None	High	None	Must
Tie to traffic lane	None	Medium	Medium	Medium	High	None	Must
Drainage of Base	Medium	Low	High	High	High	None	Must
Curb and gutter Design							
Width	None	High	None	None	None	None	Must
Tie to traffic lane	None	High	None	None	None	None	Must
Thickness	None	Medium	None	None	None	None	Should
Other Shoulder Design							
Material type	None	Medium	Medium	Medium	None	None	Should
Width	None	None	None	None	None	None	None
Thickness surface	None	None	None	None	None	None	None
Erodability	None	Medium	Medium	High	None	None	Should

Figure 58. Significance of design factors on distress types in rigid pavements (continued).

DESIGN FACTORS	DISTRESSES: JOINT DETERIORATION	CRACKING (JPCP) AND CRACK DETERIORATION	FAULTING (JRCP & CRCP)	PUMPING	PUNCHOUTS (CRCP ONLY)	FOUNDATION MOVEMENT	LEVEL OF CONSIDERATION/ IMPORTANCE
JOINT DESIGN							
Load transfer							
Aggregate interlock	None	None	High	High	None	None	Must
Stiff base	None	Medium	High	High	None	None	Must
Mechanical device	Medium	High	High	High	None	None	Must
Moment transfer	None	Low	Low	Low	None	None	None
Corrosion	High	High	Medium	None	None	None	Must
Dowel diameter/spacing and length	High	High	High	High	None	None	Must
Joint spacing	High	High	High	Medium	None	None	Must
Sealant reservoir	High	None	Low	Medium	None	None	Must
Sealant properties	High	None	Low	Medium	None	None	Must
SLAB MOISTURE AND THERMAL FACTORS							
Drying shrinkage	None	High	None	None	High	None	Must
Thermal curling	None	High	None	None	None	None	Must
Moisture warping	None	Medium	None	None	None	None	Should
DRAINAGE SYSTEM							
Permeability	High	High	High	High	High	None	Must
Erosion potential	Low	High	High	High	High	None	Must
Type of drain	None	Medium	Medium	High	Medium	None	Must
Drain outlets	None	Medium	Medium	High	Medium	None	Must
Clogging	Medium	Medium	High	High	Medium	None	Must
CLIMATE							
Moisture, annual precip.	None	Medium	High	High	High	Medium	Must
Moisture, potential evaportranspiration	None	Medium	Medium	High	Medium	Low	Must
Temp., range yearly	High	Medium	High	Medium	Medium	Low	Must
Temp., annual mean	High	Medium	High	High	High	Medium	Must
Temp., freeze-thaw	High	None	None	None	None	Medium	Must

Table 58. Significance of design factors on distress types in rigid pavements (continued).

DESIGN FACTORS	MAJOR DISTRESS TYPES: JOINT DETERIORATION	CRACKING (JPCP) AND CRACK DETERIORATION	FAULTING (JRCP & CRCP)	PUMPING	PUNCHOUTS (CRCP ONLY)	FOUNDATION MOVEMENT	LEVEL OF CONSIDERATION/ IMPORTANCE
TRAFFIC							
Truck volume	Medium	High	High	High	High	None	Must
Axle type	Medium	High	Medium	Medium	Medium	None	Must
Axle loading	Medium	High	High	High	High	None	Must
Truck lane distribution in lane	Medium	High	High	High	High	None	Must
Tire pressure	None	Medium	Low	Low	Medium	None	Must
RELIABILITY OF DESIGN	None	High	Medium	Medium	High	None	Must

The last column of table 58 provides a judgement as to whether the design factor must, should, or need not be directly considered in rigid pavement design. Any design factor rated as having a "high" effect on any key distress was considered a "must" in design consideration. Those rated "medium" "should" be considered, while factors, having a rating of either "low" or "none" were designated as "none", meaning that they will not be directly considered in design.

4.2 Capability of Models/Methods to Consider Design Factors

4.2.1 Models

The models considered include finite element programs of slabs on grade, a computerized solution of the influence charts and a program based on mechanics of materials concepts. Each model's ability to consider each type of rigid pavement is shown in table 59. Each of the models were rated for their ability to consider the design factors that were judged "must" or "should" in section 4.1. The rating matrix is given in table 60. An explanation of the 0 to 10 ratings is given in the table. Table 61 shows the ratings for the applications of each model.

Results show that the finite element models ILLI-SLAB, JSLAB, WESLIQID, WESLAYER and RISC are the most promising programs for use in design. They are versatile enough to represent the actual material properties of rigid pavements and the effects of climate and loads on them. Of these programs, ILLI-SLAB was found to be the most verified, user friendly and computer efficient program. The JSLAB, however, was also recommended considering its curling analysis application in addition to abilities most similar to ILLI-SLAB. Required computer memory becomes excessive for finite element analysis programs, therefore, the efficient use of memory is a very important parameter in deciding what can be modeled by each program. Although the other programs have many desirable features the excessive computer memory required makes routine use impractical.

The finite element programs reinforced concrete pavements can only model indirectly. Some creative analysis will be required to structurally model JRCP and CRCP with ILLI-SLAB or JSLAB for traffic loadings. The CRCP-2 program models continuously reinforced pavements considering environmental conditions and material factors. It is the only program available for use in studying crack spacing and width aspects of design for CRCP, and is recommended for this purpose. However, CRCP-2 lacks the ability to model truck loading in any realistic manner. This could be handled using ILLI-SLAB.

Table 61 was developed to show the applications of the models for routine design, evaluation of designs, research activities and special complex analyses.

Table 59. Model's ability to consider type of rigid pavement.

MODELS	TYPE OF RIGID PAVEMENT		
	Jointed Plain	Jointed Reinforced	Continuously Reinforced
ILLI-SLAB	D	I	I
JCS1	D	N	N
H51	D	N	N
CRCP-2	N	N	D
JSLAB	D	I	I
WESLIQID	D	I	I
WESLAYER	D	I	I
RISC	D	I	I

NOTE: D = DIRECTLY CONSIDERED
I = INDIRECTLY CONSIDERED
N = NOT CONSIDERED

Table 60. Design model's ability to consider factors that affect rigid pavements.

DESIGN FACTORS	ILLI-SLAB	JCS-1	H51	CRCP-2	JSLAB	WESLIQID	WESLAYER	RISC
PCC SLAB								
Thickness	10	10	10	10	10	10	10	10
Length	10	7	0	0	10	10	10	10
Width	10	7	0	0	10	10	10	10
Stiffness (E)	10	5	7	7	10	10	10	7
Strength	0	10	0	10	0	0	0	10
Fatigue properties	0	5	0	0	0	0	0	5
Durability	0	0	0	0	0	0	0	0
Reinforcement	3	0	0	10	3	3	3	3
Smoothness/performance	0	5	0	0	0	0	0	3
BASE/SUBBASE								
Thickness	10	1	1	1	10	10	10	10
Stiffness (E)	10	0	0	0	10	10	10	10
Strength	0	0	0	0	0	0	0	0
Fatigue properties	0	0	0	0	0	0	0	0
Durability	0	0	0	0	0	0	0	0
Erodability (loss of support)	5	4	0	0	5	10	10	4
Drainability	0	0	0	0	0	0	0	1
Friction between slab and base	8	0	0	0	8	8	5	5
Structural chara. of stabilized base as seperate layer	9	0	0	0	8	8	8	7

MODELS: ILLI-SLAB, JCS-1, H51, CRCP-2, JSLAB, WESLIQID, WESLAYER, RISC

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 60. Design model's ability to consider factors that affect rigid pavements (continued).

DESIGN FACTORS	MODELS							
	ILLI-SLAB	JCS-1	H51	CRCP-2	JSLAB	WESLIQID	WESLAYER	RISC
SUBGRADE								
Stiffness (E)	10	8	8	8	9	9	9	7
Strength	0	0	0	0	0	0	0	0
Drainability	0	0	0	0	0	0	0	1
Moisture sensitivity	2	1	1	1	2	2	2	1
Volume change potential	0	0	0	0	0	0	0	0
Characterization as 1) springs or 2) elastic solid	10	5	8	6	7	7	7	7
SHOULDER/EDGE SUPPORT/CURB AND GUTTER								
Type of Shoulder								
PCC	10	10	1	1	10	8	6	8
Curb and gutter	10	10	1	0	10	8	6	8
Other	10	0	0	0	10	0	0	0
PCC Shoulder								
Thickness	10	8	4	4	10	10	6	8
Length	9	8	0	0	9	9	9	9
Width	10	9	0	0	10	10	10	10
Stiffness (E)	10	5	1	1	10	6	6	6
Strength	0	10	0	0	0	0	0	7
Fatigue properties	0	8	0	0	0	0	0	6
Durability	0	0	0	0	0	0	0	0
Reinforcement	3	0	0	10	3	3	3	3
Tie to traffic lane	9	7	0	0	9	10	9	6
Drainage	0	0	0	0	0	0	0	1

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 60. Design model's ability to consider factors that affect rigid pavements (continued).

DESIGN FACTORS	MODELS							
	ILLI-SLAB	JCS-1	H51	CRCP-2	JSLAB	WESLIQID	WESLAYE	RISC
SHOULDER/EDGE SUPPORT/CURB AND GUTTER (CONTINUED)								
Curb and gutter								
Width	10	8	0	0	10	10	10	10
Tie to traffic lane	9	7	0	0	9	10	9	6
Thickness	10	8	4	4	10	10	6	8
Other								
Surface material type	8	0	0	0	8	0	0	0
Width	10	0	0	0	10	0	0	0
Thickness surface	10	0	0	0	10	0	0	0
Erodability	5	0	0	0	5	0	0	0
Base type	10	1	0	0	10	5	5	1
JOINT DESIGN								
Load transfer								
Aggregate interlock	9	6	0	0	9	10	9	0
Stiff base	5	1	0	0	5	5	7	7
Mechanical device	9	1	0	0	10	8	8	8
Moment transfer	0	0	0	0	0	9	0	0
Corrosion	0	0	0	0	0	0	0	0
Dowel diameter/spacing and length	9	0	0	0	10	8	8	8
Joint spacing	9	5	0	0	9	9	5	8
Sealant reservoir	0	0	0	0	0	0	0	0
Sealant properties	0	0	0	0	0	0	0	0

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 60. Design model's ability to consider factors that affect rigid pavements (continued).

DESIGN FACTORS	MODELS							
	ILLI-SLAB	JCS-1	H51	CRCP-2	JSLAB	WESLIQID	WESLAYER	RISC
SLAB MOISTURE AND THERMAL FACTORS								
Drying shrinkage	0	0	0	8	0	0	0	0
Thermal curling	7	0	0	0	5	7	7	0
Moisture warping	0	0	0	0	3	0	0	0
DRAINAGE SYSTEM								
Permeability	0	0	0	0	0	0	0	0
Erosion potential	0	0	0	0	0	0	0	0
Type of drain	0	0	0	0	0	0	0	0
Drain outlets	0	0	0	0	0	0	0	0
Clogging	0	0	0	0	0	0	0	0
CLIMATE								
Moisture, annual precip.	0	0	0	0	0	0	0	0
Moisture, potential evaportranspiration	0	0	0	0	0	0	0	0
Temp., min monthly	0	0	0	2	0	0	0	0
Temp., max monthly	0	0	0	2	0	0	0	0
Temp., range yearly	0	0	0	0	0	0	0	0
Temp., annual mean	0	0	0	0	0	0	0	0
Temp., freeze-thaw	0	0	0	0	0	0	0	0

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 60. Design model's ability to consider factors that affect rigid pavements (continued).

DESIGN FACTORS	ILLI-SLAB	JCS-1	H51	CRCP-2	JSLAB	WESLIQID	WESLAYER	RISC
TRAFFIC								
Truck volume	0	7	0	0	0	0	0	7
Axle type	10	5	7	1	9	9	9	6
Axle loading	10	5	7	1	10	10	10	6
Truck lane distribution in lane	0	6	0	0	0	0	0	5
Tire pressure	10	0	10	5	10	10	10	3
CALCULATIONS								
Computer effiency	10	9	9	9	9	4	3	1
Theory verification	10	5	7	5	4	5	5	5
Ease of use	10	10	10	6	7	3	3	1

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 61. Applications of design models.

APPLICATIONS	MODELS							
	ILLI-SLAB	JCS-1	H51	CRCP-2	JSLAB	WESLIQID	WESLAYER	RISC
Routine Design	8	8	4	8	8	6	6	4
Evaluating Standard Pavement Designs	10	8	4	7	9	8	8	7
Research activities	10	7	4	7	9	8	8	7
Special Analysis and Complex Cases/Studies	10	5	1	6	9	8	8	7

APPLICATION CAPABILITY:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = VERY LIMITED
0 = NONE

4.2.2 Methods

Five design procedures were considered for jointed concrete pavements and four were considered for continuously reinforced concrete pavements. The JCS-1 program for jointed concrete shoulders and the RISC program also contain procedures for developing actual rigid pavement designs (they include a finite element model, and also life prediction models).

A rating matrix was prepared for design procedures. The rating scale is shown on table 62. The overall ratings are much lower than for the models in table 60 because the capabilities of the design methods are not as well developed and verified. Each design method has some strong areas and many weak areas for considering the key design factors.

Therefore, there is not a single design procedure that is capable of handling all or even a majority of the critical design factors for rigid pavement design. Portions of many of the design procedures could be extracted and improved design procedure must be developed.

Table 62. Design method's ability to consider factors that affect rigid pavements.

DESIGN FACTORS	Jointed Concrete Pavement					CRC Pavement			
	AASHTO	Zero-M	Cal DOT	PCA	RPS-3	AASHTO	RPS-3	ARBP-CRSI	IDOT
PCC SLAB									
Thickness	5	7	3	5	4	5	4	4	3
Length	0	8	0	0	0	0	0	0	0
Width	0	0	0	0	0	0	0	0	0
Stiffness (E)	5	5	0	0	5	5	5	5	0
Strength	5	7	0	6	5	5	5	5	0
Fatigue properties	0	7	0	5	0	0	0	0	0
Durability	SM	SM	SM	SM	SM	SM	SM	SM	SM
Reinforcement	5	0	0	5	5	5	3	5	3
Smoothness/performance	5	5	0	0	5	4	4	4	4
BASE/SUBBASE									
Thickness	5	5	4	5	5	5	5	5	3
Stiffness (E)	5	3	0	0	5	5	5	5	0
Strength	0	0	0	0	0	0	0	0	0
Fatigue properties	0	0	0	0	0	0	0	0	0
Durability	SM	SM	SM	SM	SM	SM	SM	SM	SM
Erodability	5	7	0	3	5	5	5	5	0
Drainability	0	7	9	0	0	0	0	0	0
Friction between slab and base	0	0	0	0	0	0	0	0	0
Structural chara. of stabilized base as seperate layer	0	0	0	0	0	0	0	0	0

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 62. Design method's ability to consider factors that affect rigid pavements (continued).

DESIGN FACTORS	Jointed Concrete Pavement					CRC Pavement			
	AASHTO	Zero-M	Cal DOT	PCA	RPS-3	AASHTO	RPS-3	ARBP-CRSI	IDOT
SUBGRADE									
Stiffness (E)	6	7	3	5	5	6	5	5	4
Strength	0	0	0	0	0	0	0	0	0
Drainability	0	3	0	0	0	0	0	0	0
Moisture sensitivity	6	3	5	0	0	0	0	0	0
Volume change potential	5	3	3	0	6	3	6	0	0
Characterization as									
1) springs or	Y	Y	N	Y	Y	Y	Y	Y	Y
2) elastic solid	N	N	N	N	N	N	N	N	N
Erodability	5	0	0	0	5	5	5	5	0
SHOULDER/EDGE SUPPORT/CURB AND GUTTER									
Type of Shoulder									
PCC	5	0	0	5	0	5	0	5	0
Curb and gutter	0	0	5	0	0	0	0	1	5
Other	5	6	5	0	0	5	0	0	5
PCC Shoulder									
Thickness	0	5	0	0	0	0	0	5	0
Length	0	0	0	0	0	0	0	0	0
Width	0	0	0	0	0	0	0	0	0
Stiffness (E)	0	0	0	0	0	0	0	0	0
Strength	0	0	0	0	0	0	0	0	0
Fatigue properties	0	0	0	0	0	0	0	0	0
Durability	SM	SM	SM	SM	SM	SM	SM	SM	SM
Reinforcement	0	0	0	0	0	0	0	0	0
Tie to traffic lane	5	7	0	0	0	5	0	5	0
Drainage	2	5	0	0	0	2	0	1	0

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

Table 62. Design method's ability to consider factors that affect rigid pavements (continued).

DESIGN FACTORS	Jointed Concrete Pavement					CRC Pavement			
	AASHTO	Zero-M	Cal DOT	PCA	RPS-3	AASHTO	RPS-3	ARBP-CRSI	IDOT
TRAFFIC									
Truck volume	10	10	9	10	7	10	7	9	9
Axle type	10	9	9	10	5	10	5	9	7
Axle loading	10	10	5	10	5	10	5	9	5
Truck distribution									
1) between lane	5	5	5	10	5	5	5	5	5
2) in lane lateral	0	8	0	8	0	0	0	0	0
Tire pressure	1	0	0	0	0	0	0	0	0
RELIABILITY OF DESIGN									
Source of variation	6	5	0	5	7	6	7	3	0
REHABILITATION	3	0	0	0	5	1	5	0	0
COSTS									
Construction	9	5	0	0	10	9	10	3	0
Maintenance	9	0	0	0	10	9	10	3	0
Rehabilitation	5	0	0	0	9	5	7	0	0
User delay	7	0	0	0	8	7	8	0	0
User other	5	0	0	0	6	5	6	0	0

CONSIDERATION OF DESIGN PARAMETER:
10 = EXCELLENT
9 = VERY GOOD
7 = GOOD
5 = FAIR
1 = POOR
0 = NONE

5.0 DESIGN OF POTENTIAL EXPERIMENTAL PROJECTS

The objectives of this section are to develop a set of new rigid pavement designs to be constructed as inservice pavements, and to test their feasibility and effectiveness. Ideally, these experimental projects will be constructed in each climatic zone to determine their applicability in a wide range of environments.

This section presents the unique features considered in the experimental projects and a set of new designs developed for the nine climatic zones. The performance monitoring work plan recommended for this experimental project is that of the "Strategic Highway Research Program (SHRP) for Long-Term Pavement Performance (LTPP)" studies.(131)

Both jointed plain (JPCP) and reinforced (JRCP) concrete pavements were considered in the design of experimental projects.

5.1 Unique Design Features

The unique rigid pavement designs developed for the potential experimental projects were based on designs from the U.S. and foreign countries, analyses by the research staff, and research studies and experimental projects. These unique design features include the following:

- Trapezoidal slab cross section.
- Widened outer truck lane.
- Tied PCC shoulders.
- Permeable base layer.
- Longitudinal drainage pipes.
- Precoated dowels.
- Shorter joint spacings (for JRCP).

All of these features have been constructed either as experimental sections or as normal construction. Their performance has not been well documented, however, and some very potentially useful combinations of these features have not been constructed.

Compared to many conventional designs, these unique design features provide the following major advantages:

- Edge support.
- Reduction of slab curling, critical edge stresses, and deflections.
- Rapid subdrainage immediately beneath the slab.
- Adequate transverse joint load transfer.
- Reduction of JRCP joint opening/closing movement.
- Reduction of corrosion of dowel bars.
- Cost effectiveness.

The following sections describe each unique rigid pavement design feature.

5.1.1 Trapezoidal Cross Sections

A trapezoidal cross section varies linearly in thickness across all lanes carrying traffic in one direction. Slab thickness has been shown to be one of the most important design variables.[42] The advantage of a trapezoidal cross section across two or more traffic lanes is that it allows slab thickness to vary according to the traffic loading across the lanes. The trapezoidal cross section would provide a thickened outer edge of the truck lane, either for increased reliability for the same quantity of PCC as in a uniform section, or decreased cost and the same reliability (thinner section overall).

A typical variation in truck traffic across lanes is shown in the table 63. This variation has resulted in many badly deteriorated pavements in the outer lane and almost no deterioration in the inner lane. The California Department of Transportation (Caltrans) design procedure recommends the use of a tapered cross section in the passing (inner) lane to avoid steps in the structural section on multilane facilities (where the traffic load between adjacent lanes indicates two different thickness of PCC) with a constant thickness of base.[49] Figure 20 shows the trapezoidal cross section used by Caltrans.

France has used the trapezoidal cross section for several multiple lane highways with a linearly varied PCC thickness since 1976. Figure 21 shows the thickness combinations of the pavement structure according to different design traffic levels and soil conditions and an example of the trapezoidal cross section. The PCC slab thickness for the case of $T_1S_3S_4$, in the illustrated example, is tapered linearly from 11.0 in (28 cm) at the extremity of the slab on the heavy traffic lane side, to 8.7 in (22 cm) at the other extremity. This design allows the thickness of the slab for different traffic lanes to be designed for its traffic level. Figure 22 shows a photograph of a freeway near Paris that has a trapezoidal cross sections and a widened outer traffic lane that has performed very well for over 10 years. This pavement carries 45,00 ADT with 20 percent trucks. Edge drainage was provided. Very little pumping, faulting or cracking was observed by the author. However, one short section did not have edge drains, and pumping was evident.

The slab thickness for a trapezoidal two-lane cross section developed for the experimental projects is tapered from t_1 at the outer edge of the outer lane to t_2 at the inner edge of the inner lane, where the t_1 and t_2 are determined as follows:

t_1 = design thickness for traffic in outer lane edge plus 1 in (2.54 cm)
t_2 = design thickness for traffic in inner lane edge (8 in minimum (20.3 cm))

The transverse cross section is shown in appendix B.

5.1.2 Widened Truck Lanes

The widened lane concept for rigid pavements is an attempt to minimize the load-associated distresses at the corner and the lane/shoulder joint. The traffic lane is widened by 2 to 3 ft with striping and rumble strips to discourage traffic encroachment of this area. This feature causes a considerable reduction in deflections under wheel load along the longitudinal joint and particularly at the slab corner. Truck loading becomes practically like an interior load in terms of deflection and stress.

Several States and foreign countries are now building widened traffic lanes. Minnesota has constructed several widened (15 ft (4.6 m)) outside lane pavements, but the actual benefits are not yet clear. However, they have performed well and indicate potential benefits in extending the life of JRCP.[76] Figure 23 shows a photograph of Minnesota T.H.15 JRCP with a widened outer traffic lane.

In 1973 France began widening lanes by 2.5 ft (0.75 m), combined with the trapezoidal cross section. The original truck lane was 11.5 ft (3.5 m) wide, i.e., total width of the truck lane is 14 ft (4.25 m). (see figures 21 and 22)

West Germany widened lanes by 1.6 ft (0.5 m) beginning in the early 1970's. The original standard lane width was 12.3 ft (3.75 m), thus the total width of each lane is approximately 14 ft (4.25 m). Figures 24 and 25 illustrate West Germany's widened-lane structure and construction. The performance of this widened-lane feature was reportedly very good. Pavement sections showed no major distresses after 8 years of heavy traffic (survey by author).

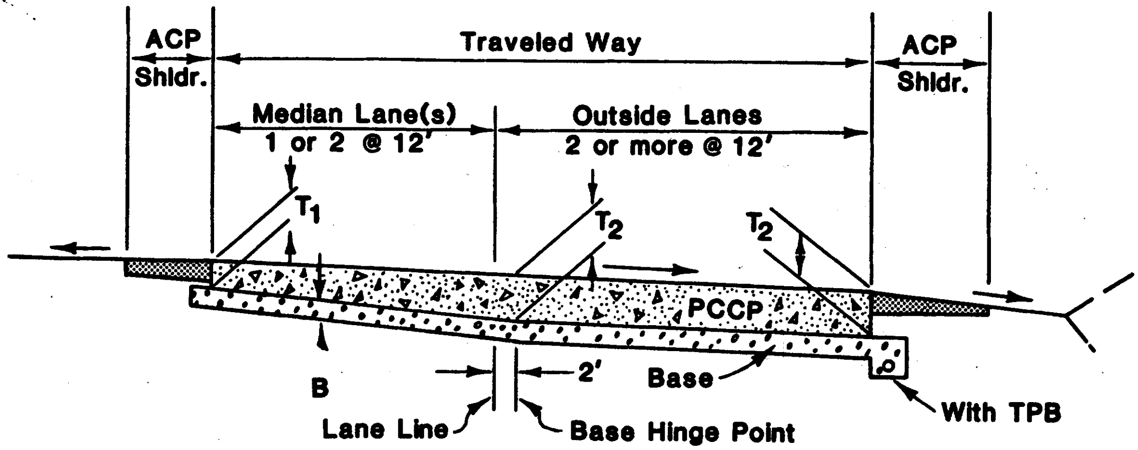
Table 63. Truck distribution for multiple-Lane controlled access highways

One-Way ADT	2 Lanes (One-Direction)		3+ Lanes (One-Direction)		
	Inner	Outer	Inner*	Center	Outer
10,000	19**	81	7	25	68
20,000	25	75	7	30	63
40,000	31	69	8	35	57
80,000	--	--	8	41	51

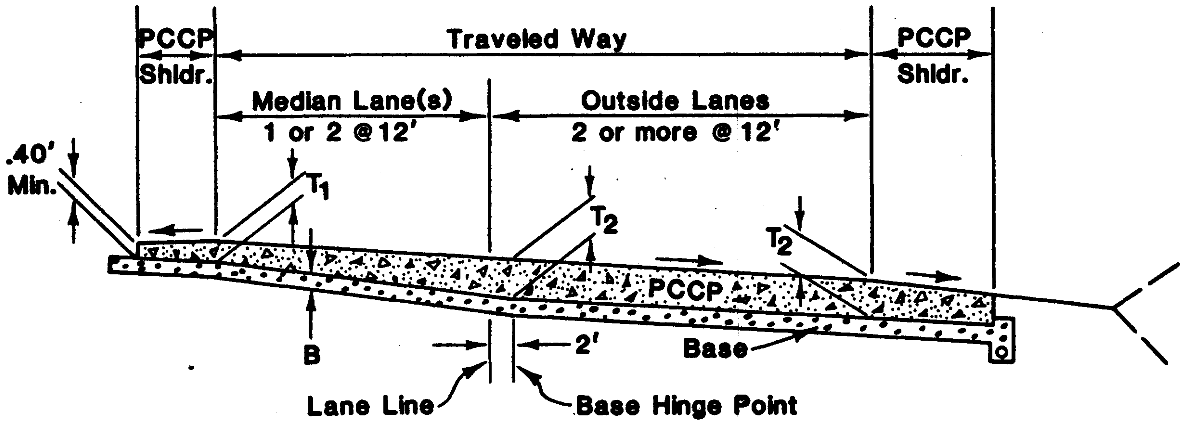
Source: NCHRP Project 1-19 (NCHRP Report 277).

* Combined inner lanes (one or more).
** Percentage of all trucks in one direction.

Tapered Cross Section
For 3 or More Lanes One Direction



(a) **Asphalt Concrete Shoulders**



(b) **Concrete Shoulders**

T_1 = PCC Thickness For Median Lane TI
T_2 = PCC Thickness for Outside Lane TI
B = Constant Base Thickness for Outside Lane TI for the Entire Width

Figure 20. Typical trapezoidal cross section used by Caltrans: (a) with AC shoulders; (b) with PCC shoulders. [49]

(maximum weight per axle:13t)

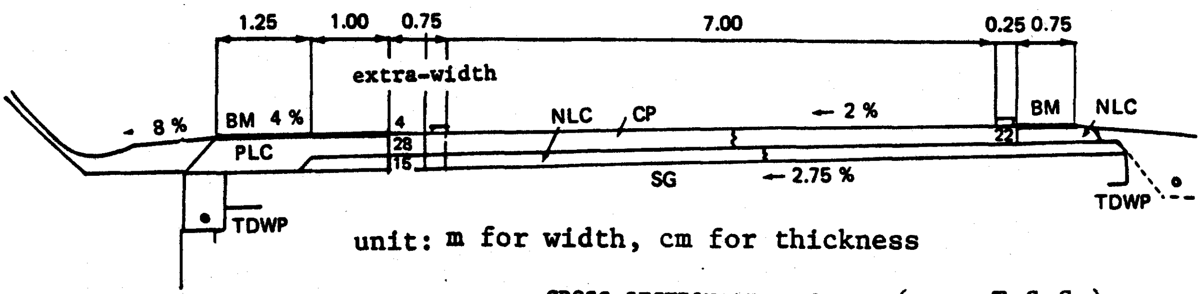
Concrete (BC). Sand-gravel-cement (GC). Untreated sand-gravel (GNT)

TRAFFIC / SOIL	T_1	T_2	T_3	T_4
S_1 S_2	27cm BC 15cm GC	25cm BC 15cm GC	22cm BC 15cm GC	20cm BC 15cm GC
S_3 S_4	25cm BC 15cm GC	23cm BC 15cm GC	20cm BC 15cm GNT (1)	18cm BC 10cm GNT (1)

REMARKS ON THE CROSS-SECTIONAL PROFILE

For dual carriageway roads it is recommended that the thickness of the concrete slab should be varied linearly:

- from 28cm, at the extremity of the slab on the heavy traffic lane side, to 22cm, at the other extremity in the case of T_1 S_3 S_4;
- from 26cm to 20cm in the case of T_1 S_3 S_4;
- from 26cm to 22cm in the case of T_2 S_1 S_2;
- from 24cm to 20cm in the case of T_2 S_3 S_4;
- from 23cm to 20cm in the case of T_3 S_1 S_2;
- from 21cm to 18cm in the case of T_3 S_3 S_4.



CROSS SECTION AFTER 1976 (case $T_1S_3S_4$)

CP = Concrete pavement

CTB = Cement treated base

NLC = Normal lean concrete

PLC = Porous lean concrete

TDWP = Trench drain with pipe

BM = Bituminous mix

SG = Subgrade

Note: 1.0 in = 2.54 cm
1.0 ft = 0.305 m

Figure 21. Typical trapezoidal and widened-lane cross section design in France.[81,82,83]

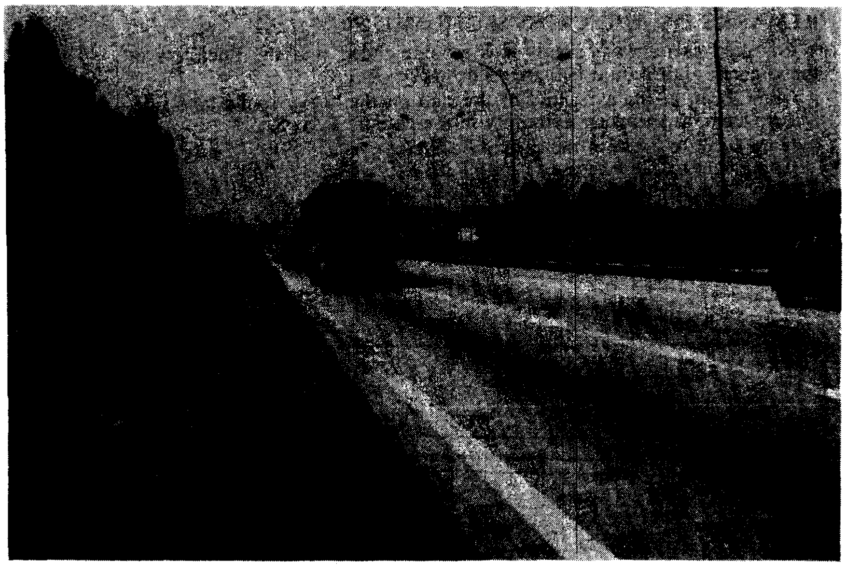


Figure 22 Photograph of 10-year old freeway near Paris that has a trapezoidal cross section and a widened outer traffic lane (10.2 to 11.4 in [26-29 cm] slab over two traffic lanes), no dowels, edge drain, 1986.

To determine the necessary width, a comprehensive fatigue analysis was conducted using the finite element program and Miner's fatigue damage hypothesis in the Zero-Maintenance study across a traffic lane.[46] Figure 26 shows the illustration of the mean distribution distance, D, from the slab edge to the outside of the truck wheels. Some interesting findings of the study were:

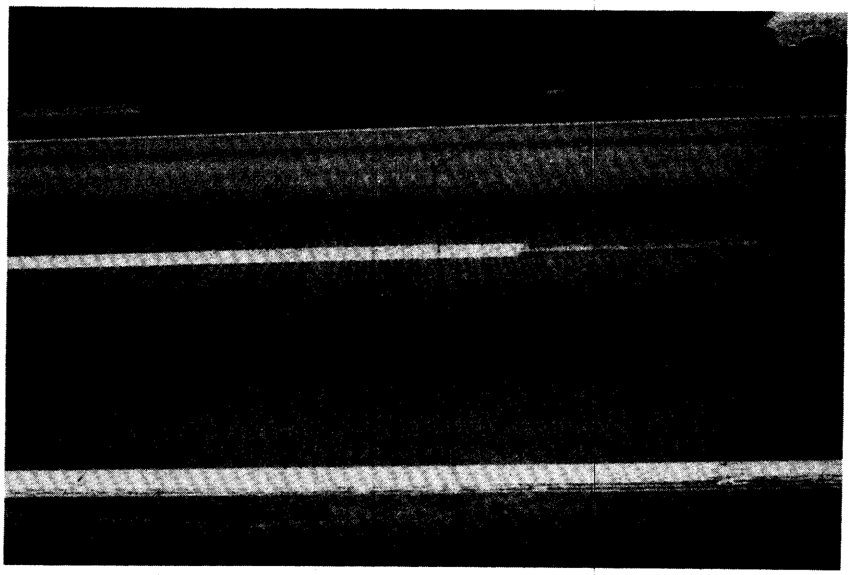
- Transverse cracks initiate at the outer slab edge and work their way across the slab. The critical fatigue damage point is at the slab edge.

- The highest stress occurs when the load is at the slab edge (lateral placement of trucks, D = 0 in) and decreases as the load is moved inward (D = 6, 18 and 30 in (15,46, and 76 cm)) (see figure 27).

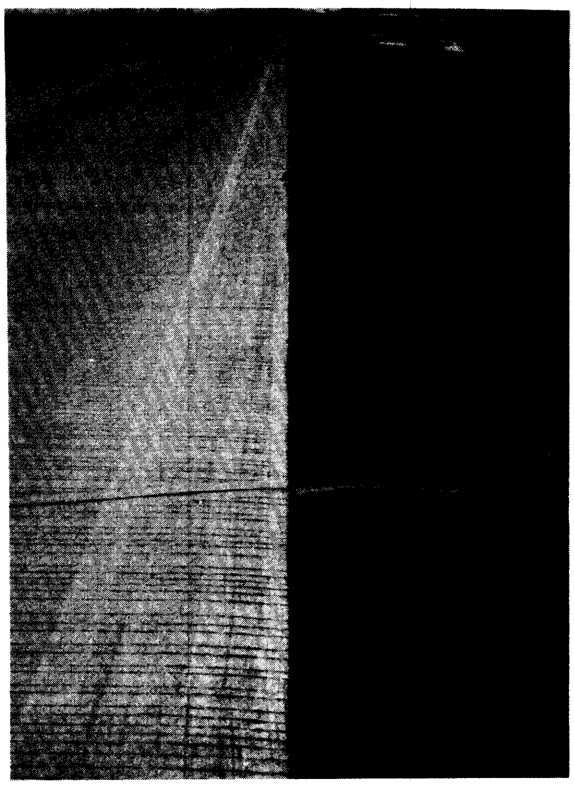
- As shown in figure 28, for an 8-in slab, when the mean lateral placement (D) is less than 36 in (91 cm), the critical fatigue damage point is at D = 0 or the slab edge, and the amount of fatigue damage at slab edge decreases as the lateral placement increases. When the mean lateral placement is more than 42 in (106 cm), the critical fatigue damage point is moved inward near the wheel load. The effect of the reduction of the critical fatigue damage by increasing the lateral placement of trucks is even more significant for thicker slabs. The mean lateral placement of trucks, D, is typically 18 in (46 cm) from the slab edge. This indicates that an additional width of 24 in (42-18=24) (61 cm) would remove the outer edge from being the critical fatigue location.

- The mid-slab edge position has a much higher fatigue damage than the transverse joint position when both are subjected to the same traffic, i.e., transverse cracking would theoretically be expected to occur long before longitudinal cracking (see figure 28 to figure 33). Field observations support this result.[42]

The concrete bearing stress was computed for doweled transverse joints for the widened lane using the ILLI-SLAB finite element program. The input parameters are presented in table 64. The single-axle wheel load was first placed on the corner at a transverse joint, and then an extra width was added to the slab. Three values of extra width, 0, 15 and 31 in (0,38,79 cm), were used. Table 65 gives the pavement responses, such as maximum deflection, maximum stresses and distribution factors, for each widened-lane case.



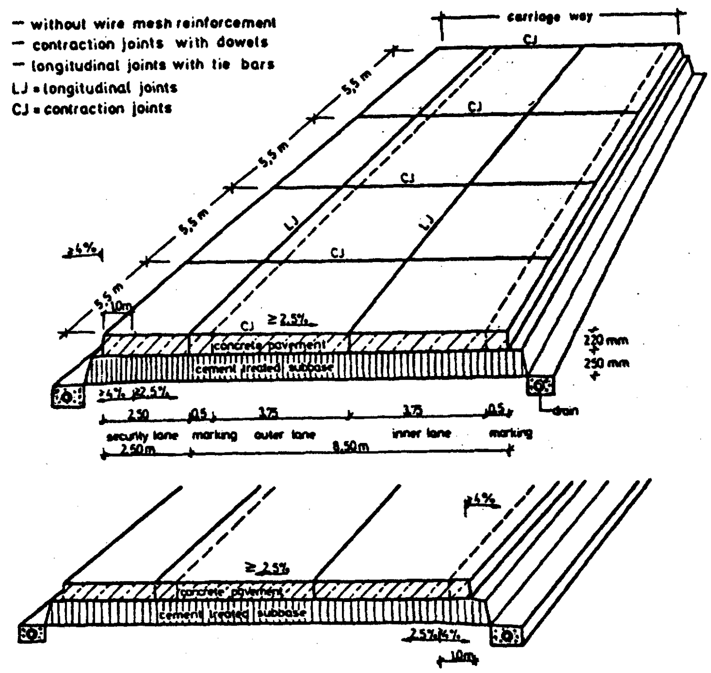
(a) Transverse joint



(b) Widened lane

Figure 23 Photographs of Minnesota T.H.15 JRCP with widened outer traffic lane.

Concrete Full Depth Construction



1.0 in = 25.4 mm; 1.0 ft = 0.305 m

Figure 24. Typical widened-lane (1.64 ft or 0.5 m) cross section design in West Germany (subbase is bonded to PCC slabb; recently grooves have been cut in subbase below longitudinal and transverse joints to control cracking).

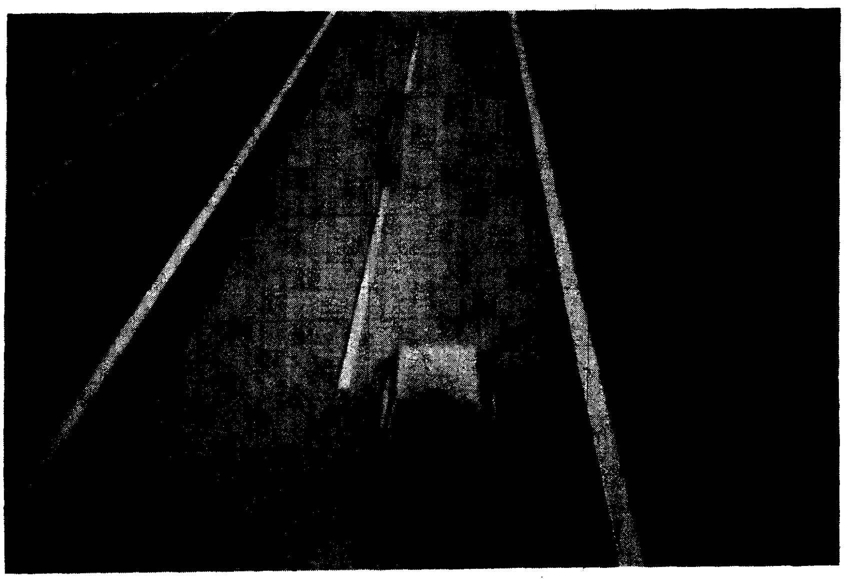


Figure 25. Photograph of a widened-lane cross section in West Germany, 1986.

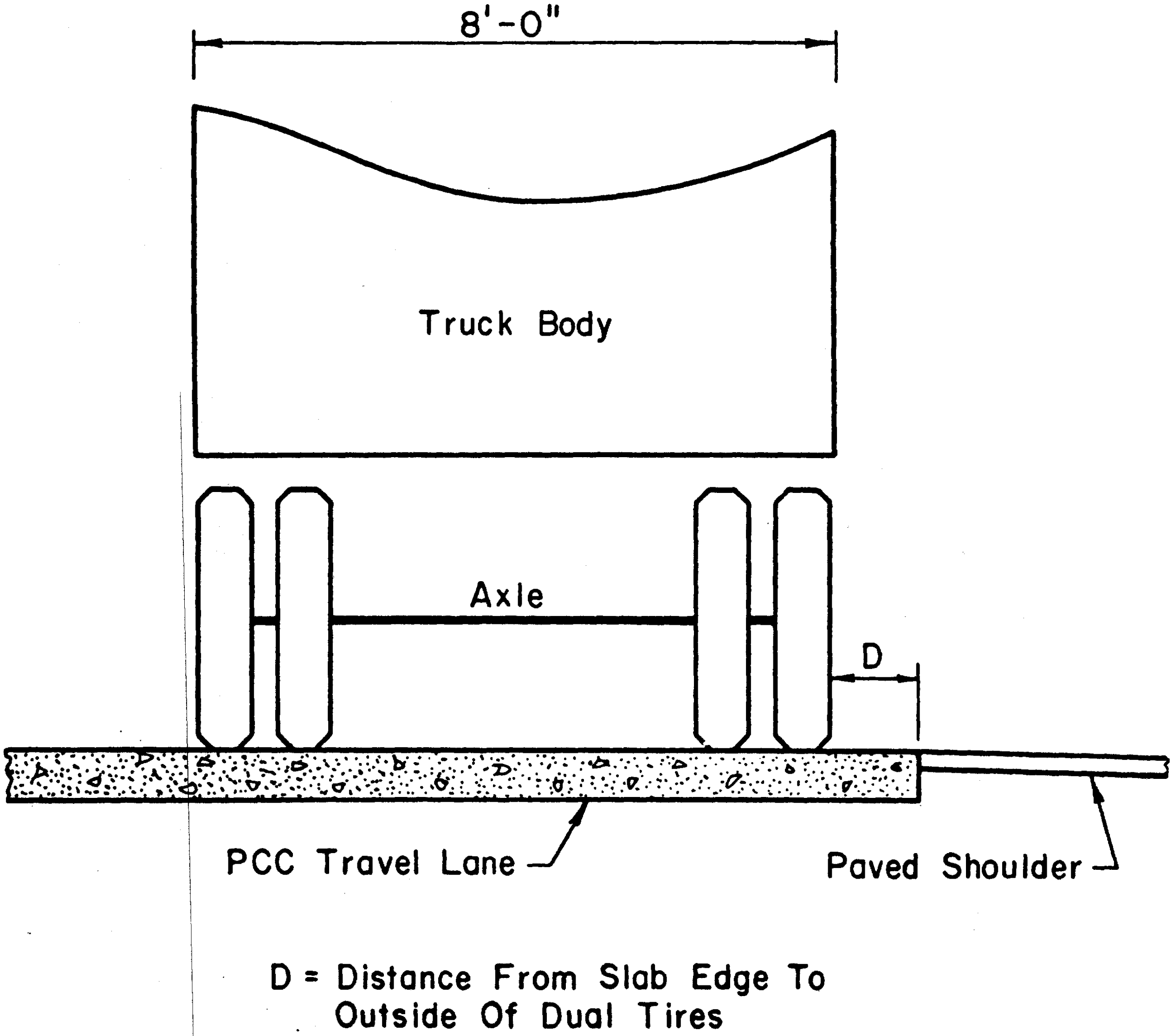


Figure 26. Illustration of the mean distance from slab edge to outside of dual tires. [46]

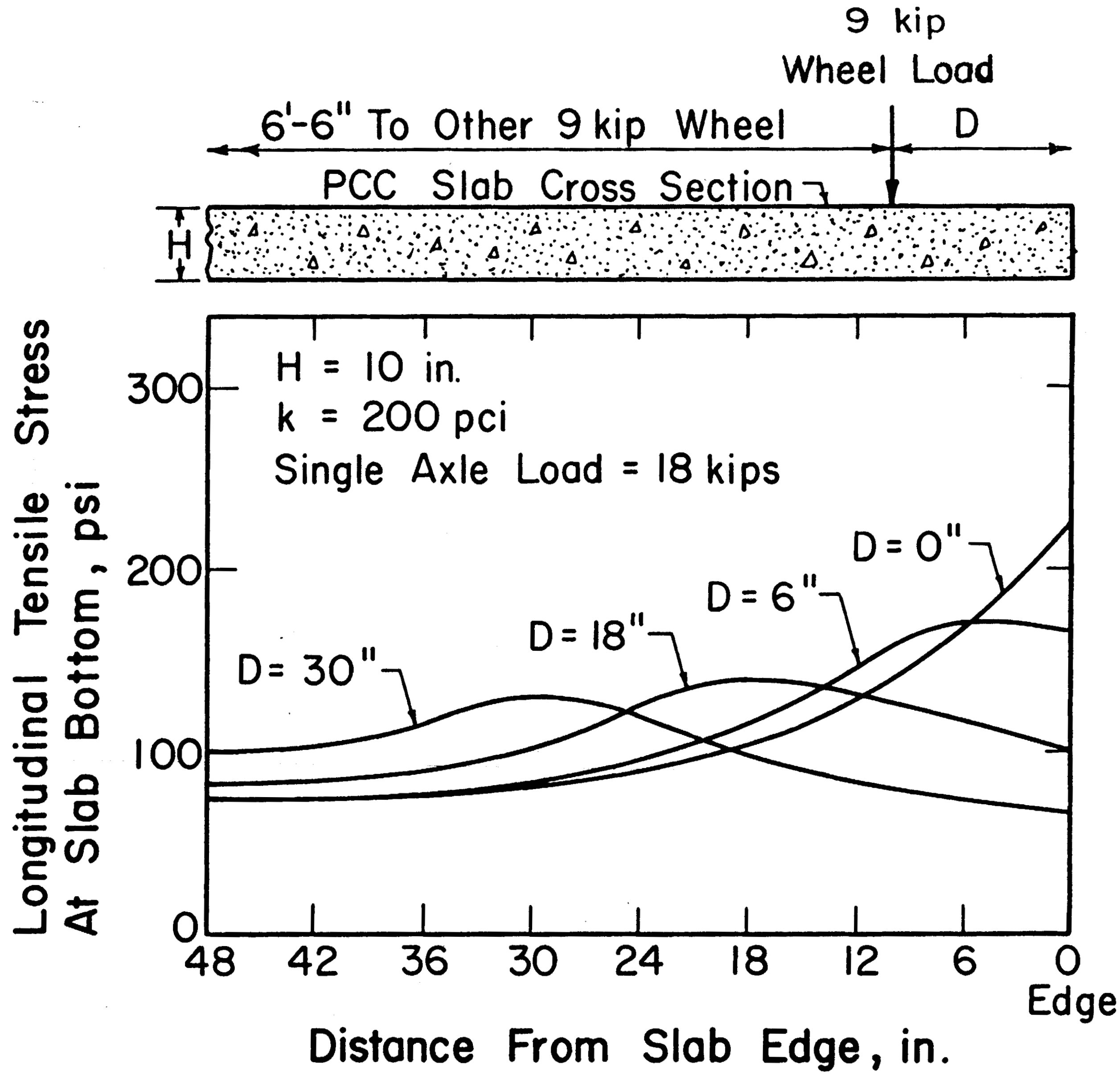


Figure 27. Computed tensile stresses across bottom of PCC slab at midpoint between transverse joints for various transverse positions of axle load. [46]

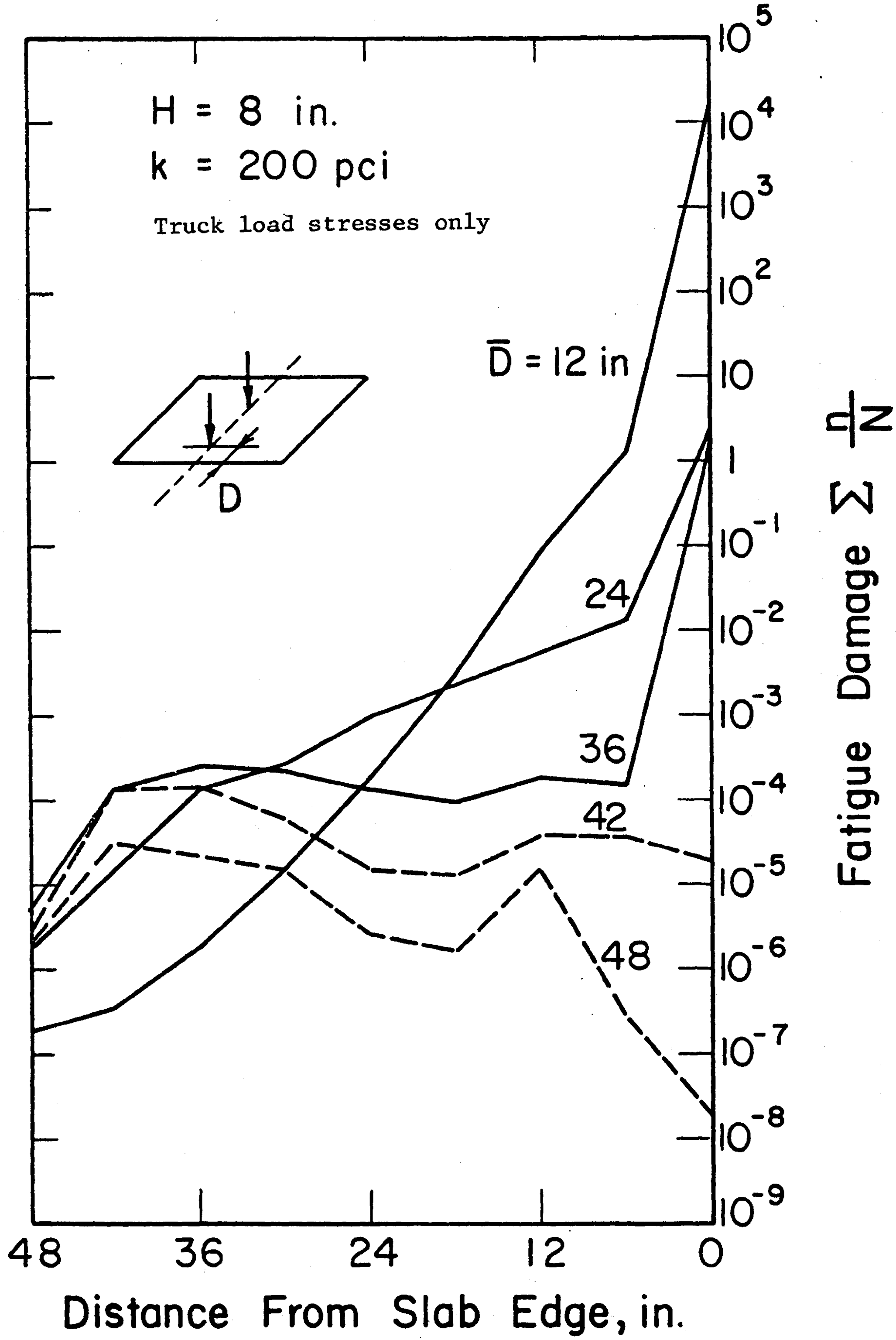


Figure 28. Computed fatigue damage across slab due to lateral distribution of trucks on lane--at midpoint between transverse joints. (8-in slab) [46]

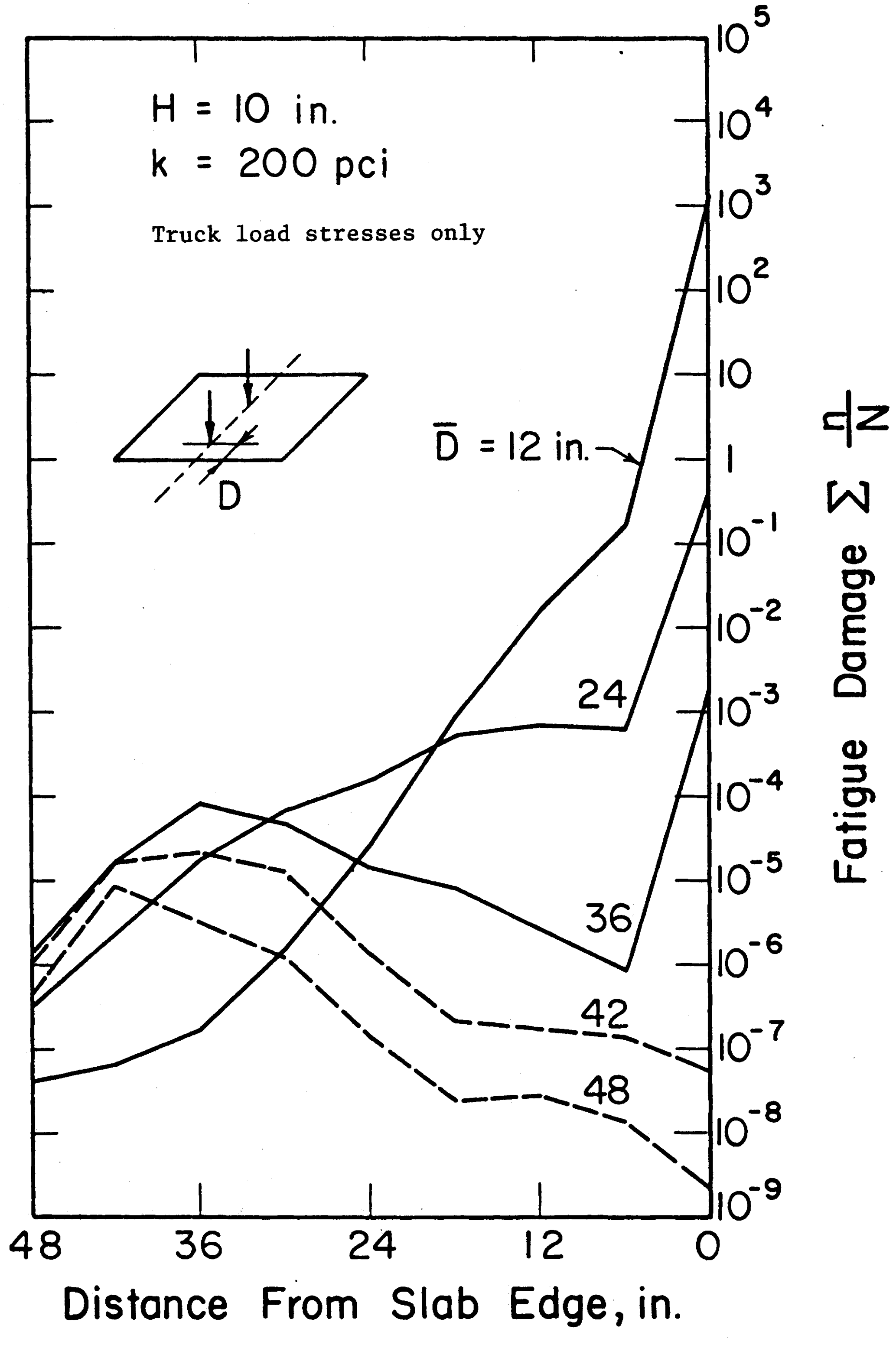


Figure 29. Computed fatigue damage across slab due to lateral distribution of trucks in lane--at midpoint between transverse joints. (10-in slab) [46]

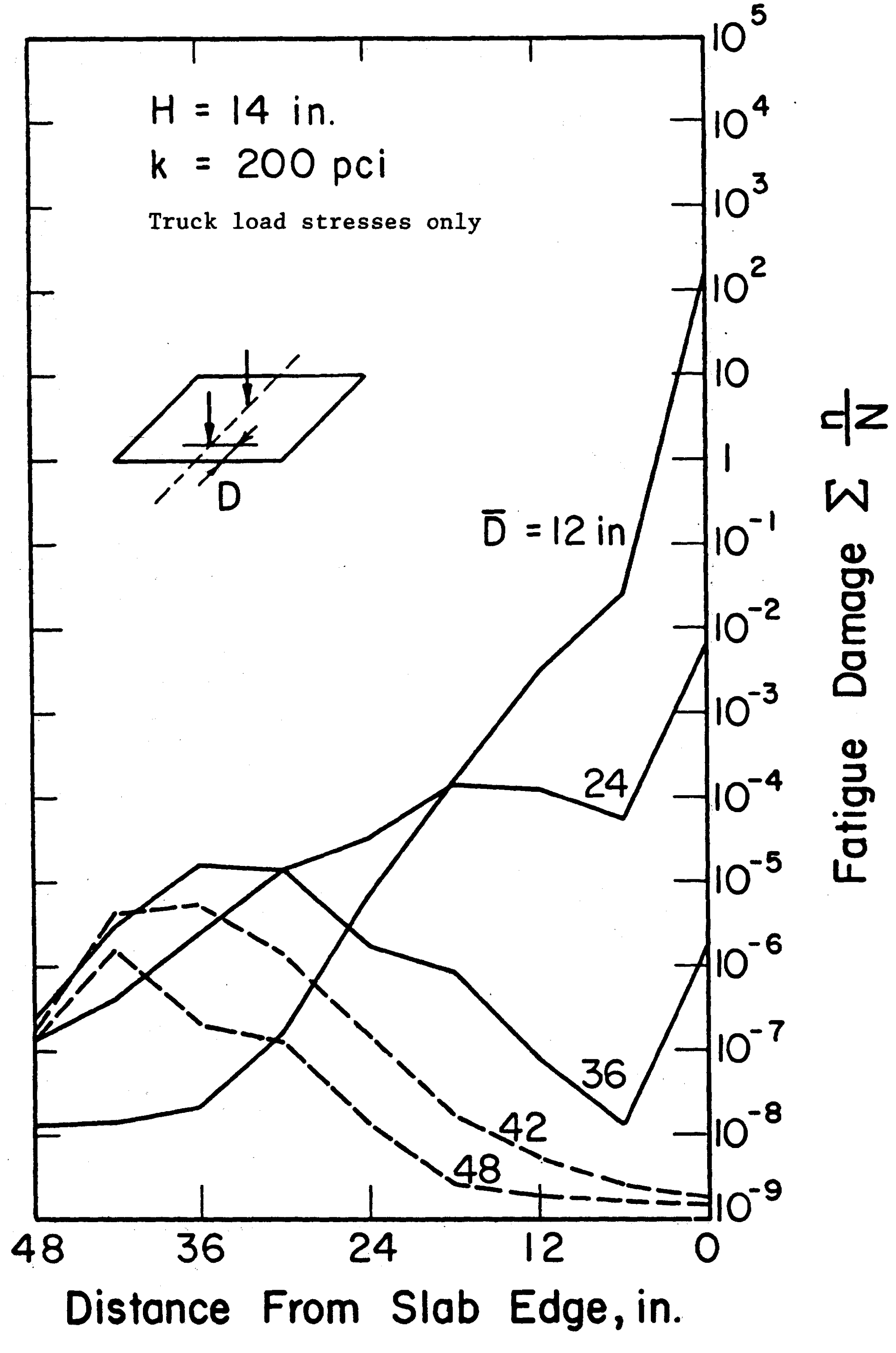


Figure 30. Computed fatigue damage across slab due to lateral distribution of trucks in lane--at midpoint between transverse joints. (14-in slab) [46]

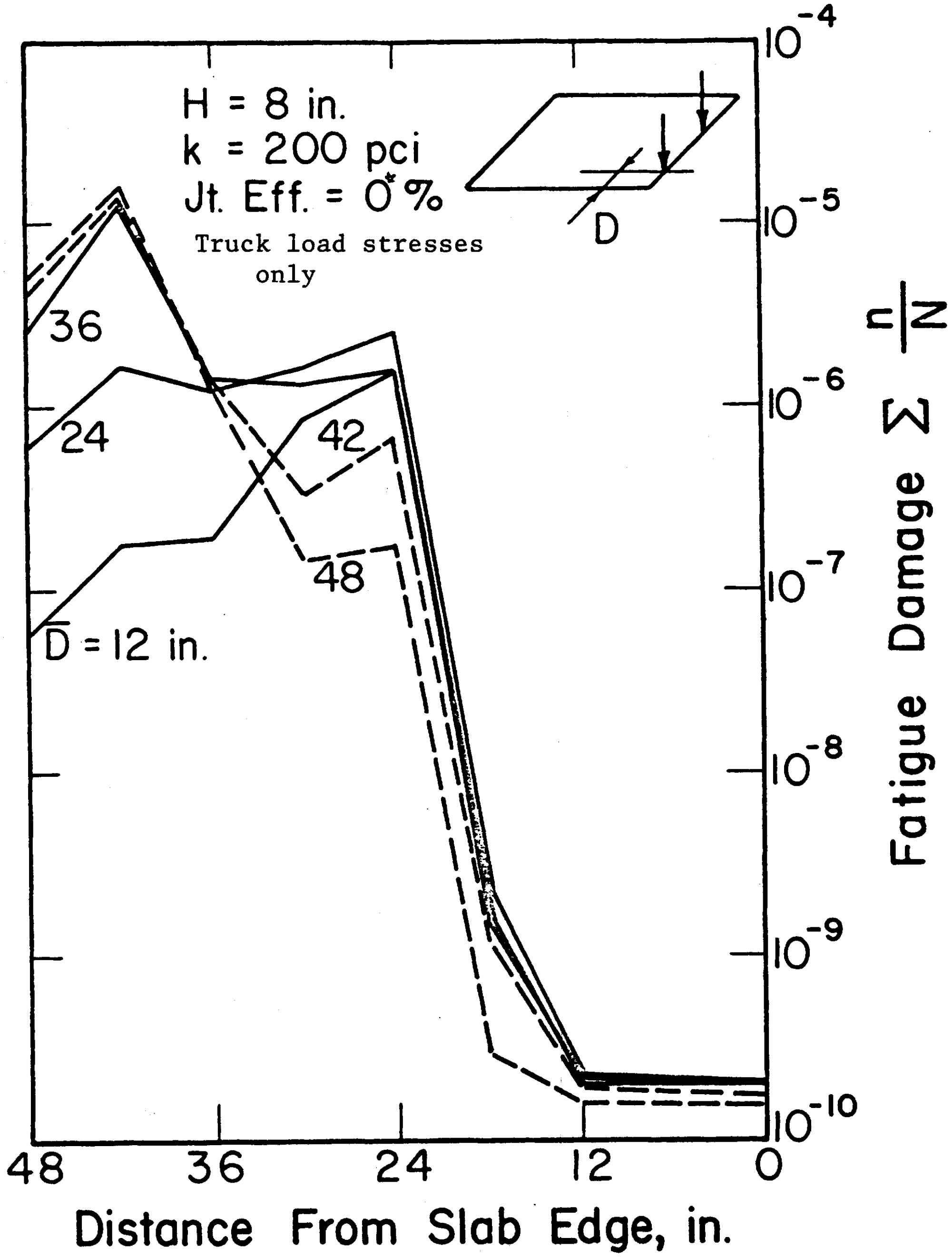


Figure 31. Computed fatigue damage across slab due to lateral distribution of trucks in lane--at transverse joint (8-in slab) (H = slab thickness, k = modulus of foundation, D = distrance from edge of slab measured toward center of slab). [46]

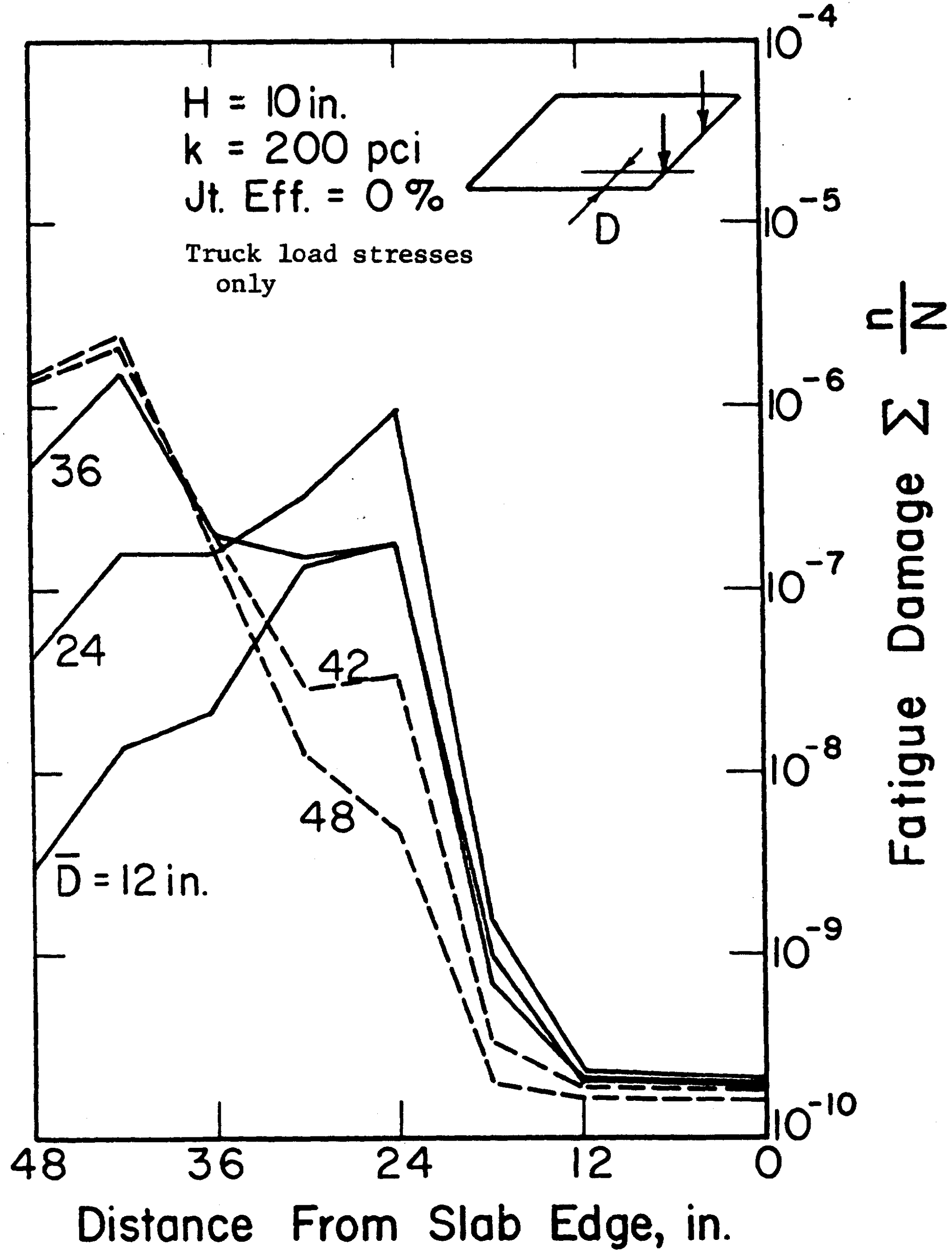


Figure 32. Computed fatigue damage across slab due to lateral distribution of trucks on lane--at transverse joints. (10-in slab) [46]

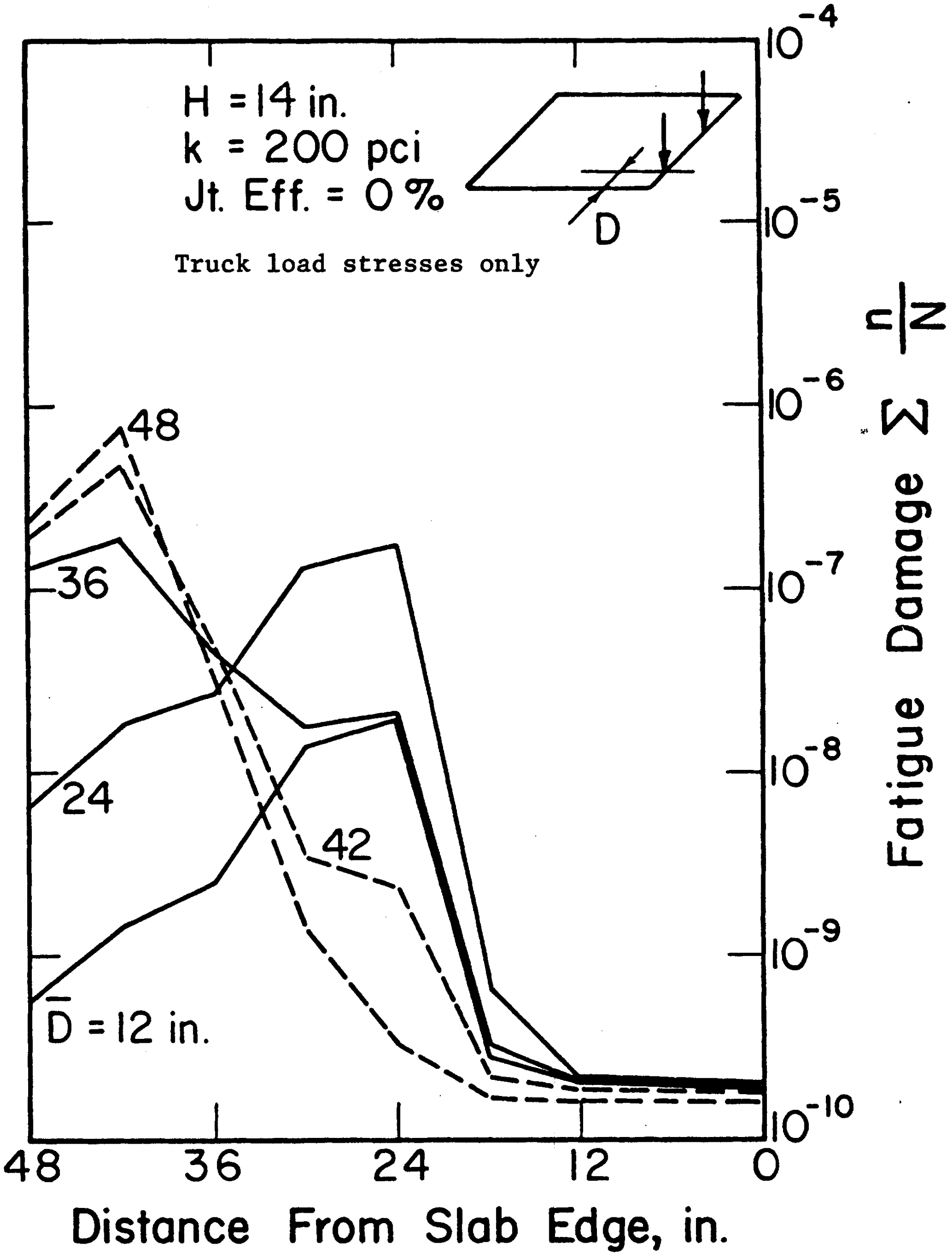


Figure 33. Computed fatigue damage across slab due to lateral distribution of trucks in lane--at transverse joint. (14-in slab) [46]

Table 64. Parameter inputs for ILLI-SLAB program for one-layer pavement

TYPE OF PAVEMENT	JPCP	
SURFACE LAYER		
PCC SLAB THICKNESS	10	in
POISSON'S RATIO	0.15	
MODULUS OF ELASTICITY	4000000	psi
SUBGRADE		
SUBGRADE MODEL	WINKLER	
SUBGRADE MODULUS	50	pci
DOWEL AND JOINT PARAMETERS		
JOINT OPENING	0.2	in
MODULUS OF DOWEL SUPPORT	1500000	pci
MODULUS OF ELASTICITY OF DOWEL BARS	29000000	psi
POISSON'S RATIO OF DOWEL BARS	0.30	
TRANSVERSE JOINT		
DOWEL BAR DIAMETER	0.75	in
DOWEL BAR SPACING	12.0	in
DOWEL CONCRETE INTERACTION (DCI) BY FRIBERG'S ANALYSIS	581123	lb/in
LONGITUDINAL JOINT	N/A	
LOADING		
TYPE OF AXLE	Single axle	
TYPE OF WHEEL	Single wheel	
GROSS WEIGHT OF AXLE	18000	lbs
TIRE PRESSURE	90.0	psi
AREA OF (SQUARE SHAPE) TIRE PRINT	10x10	in^2

1 in = 2.54 cm
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3
1 kip = 454 kg

Table 65. Pavement response with varying widened slab widths using ILLI-SLAB program

Run No.	Loading Condition	Extra Slab Width* (in)	Maximum Deflection (in)	Maximum Stress** (psi)	Distribution Factor*** (%)
1	Corner	0.0	54.06	110	21.53
2	Edge	15.0	41.01	152	14.82
3	Edge	31.0	31.95	193	12.55

* The distance from slab edge to the outside of the truck wheel equal to the same value of the extra (widened) slab width in each case.

** The maximum stress is determined by the calculated principle tensile stress at the bottom of the slab.

*** The Distribution Factor is defined in terms of the percentage of transferred load contributing to the maximum concrete bearing stress. The values of the transferred load calculated from ILLISLAB program for all three cases are approximately equal to 7690 lbs (or 43% of 18 kips applied load).

1 in = 2.54 cm
1 kip = 454 kg
1 psi = 0.07031 kg/cm^2

The transferred load calculated from the ILLI-SLAB program for all three cases were approximately equal 7,690 lbs (3491 kg) (or 43 percent of 18 kips applied load), therefore, the value of maximum bearing stress depends solely on the "distribution factor" for the same pavement structure. The distribution factor, which reflects the magnitude of the transferred load contributing to the maximum bearing stress, is defined as a percentage of the transferred load. Some major findings from the analysis are as follows:

1. The maximum corner deflections were significantly decreased when the extra width was added. This is because an axle loading changed from a corner load to a edge load when the loading was applied at a transverse joint.

2. The maximum tensile stress in the slab increased with the width. This increases the potential of slab cracking. However, the maximum stress is less critical than the maximum deflection for the corner loading condition.

3. The distribution factor in the widened-lane case, for example, a 10-in (25.4 cm) slab widened by 31 in (79 cm) (widened width, is approximately 12.6 percent compared to 21.5 percent for the standard slab. This means that the maximum concrete bearing stress was reduced approximately 41 percent when the slab was widened from 0 to 31 in.

This result implies that the size of dowel bars can be designed more economically without increasing the bearing stress between dowel and concrete when widened-lanes are being used. This has been done in West Germany where the use of 1-in (2.5 cm) diameter dowel bars, with 12 in (30 cm) spacing, has not resulted in faulting under heavy loads.

As trucks become wider, this may shift in wheel loads closer to the lane/shoulder joint. This increases the advantages of the widened traffic lane.

The proposed design for the widened lane consists of a 2.0 ft (0.61 m) extra width plus the original width of the truck lane (or a total width of 14 ft)(4.3 m). The cross section of the widened-lane design developed for the experimental projects is shown in appendix B. The type of shoulder used with a widened truck lane can be either tied PCC or AC. The use of tied PCC shoulder is highly recommended for improved performance, however.

Proper longitudinal joint design and construction (early sawing before microcracking to a depth of at least one third of the slab thickness) is recommended. Curling and warping stresses become significant for widened traffic lanes and may cause increased probability of longitudinal cracking. Minnesota had a problem with 15 ft (4.6 m) lanes and now builds 13 ft (4 m) left and 14 ft (4.3 m) right lane on one-way roadways. It is reported that the wider the slab, the greater the longitudinal cracking (12.2 to 15.5 ft or 3.65 to 4.65 m).[87]

5.1.3 Tied PCC Shoulders

Tied PCC shoulders provide low-maintenance performance in addition to the beneficial effect of increased slab edge and corner support. The PCC shoulders tied to the traffic lane also result in a tight lane/shoulder joint that greatly reduces moisture infiltration into the pavement section.

A concrete traffic lane having an asphalt concrete shoulder provides a material inconsistency that makes it nearly impossible to seal the joint against moisture infiltration. The difference in thermal properties of PCC and AC makes it very difficult to obtain a sealant that can perform properly and bond to both materials. NCHRP Project 1-19 and another field study of shoulder performance have shown evidence that AC shoulders built in most states were deteriorated along the lane/shoulder joint in the form of an open joint, alligator cracking and/or pumping blowholes.[42,75] A lane/shoulder joint without proper sealant or with the shoulder pulled away from the traffic lane allows moisture to freely infiltrate into the pavement structure.[75] This often results in pumping and softening of the underlying bases. Full-depth AC shoulders generally give better performance. However, they require frequent longitudinal joint maintenance and often exhibit some separation and cracking at the longitudinal lane/shoulder joint in freeze areas.

West Germany's 8.2-ft (2.5 m) PCC shoulders tied onto their widened truck lanes provide a temporary traffic lane for emergency use (see also figure 24). Tied PCC shoulders have been observed to give over 20 years of almost maintenance-free performance in Illinois, and equal performance in other States over shorter time periods. Recommendations from highway agency engineers indicate a preference for tied PCC shoulders when the main traffic lanes were PCC. The long-term effectiveness of edge support by tied PCC shoulders has not been proven through field performance for all types of rigid pavements. Two studies are cited that show the potential effect of tied concrete shoulders.

1. The oldest section of retrofit PCC shoulders (uniform 6 in (15 cm) thickness) in the U.S., on Route 116 in Illinois, showed the following cracking and faulting after 21 years (the shoulder was placed soon after initial construction). The traffic lane pavement was 10 in (25 cm) thick JRCP with 100 ft (30 m) joint spacing.[73]

Traffic Lane	ESAL	Faulting, in	Deteriorated Cracks, ft/mile
Outer (tied PCC sh.)	2,759,000	0.05	317
Inner (AC sh.)	287,000	0.11	412

The inner lane has much greater cracking and faulting than the outer lane which was tied to the PCC shoulder, despite having about one-tenth the traffic.

2. Two sections of CRCP outer traffic lanes had retrofit PCC shoulders (6 in (15 cm) thick, 10 (3 m) to 100 ft (30 m) joint spacing) placed soon after construction over a portion of the projects. These were surveyed after 9 and 10 years of performance.[13] The following results were obtained.

Project	Age	PCC Shoulder	Edge Punchouts/mi.	Wide Cracks/mi.
I-80	9	No	6.0	7.9
		Yes	1.1	5.0
I-74	10	No	4.0	1.6
		Yes	0.0	0.0

Those portions of the projects having tied PCC shoulders showed much better performance than those sections without PCC shoulders. Also, the I-80 sections had anchor bolts that had begun to pull out resulting in a poor load transfer between the shoulder and traffic lane.

An investigation was conducted using the ILLI-SLAB program to simulate the edge beam (a beam structure similar to PCC shoulder tied to truck lane to increase edge support) effects. It demonstrated that the edge beam concept can substantially decrease critical edge and corner deflections and stresses in pavements when voids are present beneath the slab.[74]

A field study conducted in Minnesota to evaluate the effect of tied PCC shoulders also showed that the pavement structural response is improved for pavements using a tied PCC shoulder as compared to pavements without one.[76] Curling and warping stresses were not considered, however.

The addition of tied PCC shoulders could reduce the slab thickness required to sustain the same traffic loads. The AASHTO and PCA design procedures have incorporated the optional use of tied PCC shoulders into their design considerations.

The effectiveness of the edge support depends upon the reduction in deflections and stresses in the traveled lane slab. This reduction depends on the adequacy of the tie system and the width and thickness of the PCC shoulder. The design of a tied PCC shoulder becomes the most important design consideration to ensure the long-term effectiveness of edge support. Recommendations are provided in references 13 and 14 for this tie.

5.1.4 Permeable Base Layer and Longitudinal Drainage Pipes

Many field studies have revealed that much of the deterioration in all types of rigid pavements is caused by exposure to heavy truck loadings when the pavement structure is in a saturated condition. In the past few years, pavement engineers have started looking at improved subdrainage through the use of permeable bases placed directly beneath the PCC slab to reduce this deterioration. Some experimental projects have shown significant benefits with permeable base layers (such as the Clare, Michigan project observed recently by the authors, where both faulting and joint spalling was greatly reduced over that of bathtub type bases).

A project which studied the pavement subdrainage system was conducted in New Jersey. The project was located in frost penetrated area and wet freeze climate. Major findings and conclusions obtained from the study were summarized as follows:[85]

- Generally, moisture is constantly present at a depth of about 6 (15 cm) in below the lowest bound layer of a pavement, and in amounts that bring the base material close to its saturation point. The type of surfacing had little effect on the moisture conditions immediately below a pavement appeared in this study. It is recommended that the top 4 in of the uppermost unbound layer of the pavement should be replaced by a suitable drainage layer which is drained by longitudinal edge drains.

- The drainage layer should satisfy three basic requirements: it must be open enough to drain water in a reasonable length of time yet with low enough flow rates to prevent internal erosion; it must be dense enough to support traffic loads; and it must possess filtration characteristics compatible with base and subbase materials.

- Two types of drainage layer materials, bituminous-stabilized open-graded and nonstabilized open-graded, were developed through laboratory tests in the study. Each could be used with any kind of pavement, however the stabilized material was considered most appropriate for flexible pavement construction and the nonstabilized material best for use with rigid pavements.

- The Army Corps of Engineers Waterway Experiment Station (WES) evaluated the relative strength of the open-graded mixtures. The conclusion was that, provided there was at least 6 in of overburden for confinement, these materials would perform structurally on a par with the New Jersey DOT's Type 5A stone base. Type 5A is a dense-graded, nonstabilized stone base roughly equivalent to the dense-graded stone base of the AASHO Road Test.

- The Army Corps of Engineers Cold Region Research Experimentation Laboratory (CRREL) performed a series of studies to determine if an open-graded layer would cause increased frost penetration. The basic conclusion was that open-graded drainage layers will probably have very little effect on frost penetration under pavements in New Jersey.

- In addition to the gradation specification, a test for density, permeability and gradation stability should be incorporated into the specifications for drainage layer materials to secure adequate base, subbase and subgrade materials for a highway.

- Extensive full-scale experimentation is required to verify the research theories and laboratory findings for subdrainage design.

- Effective subdrainage for infiltrating surface water is only possible if there is effective surface drainage.

Typical open-graded permeable bases consist of high quality coarsely open-graded crushed aggregate, or open-graded crushed aggregate treated with asphalt or portland cement. The use of asphalt or Portland cement as a binder is determined basically on the basis of economic consideration and materials availability.

Open-graded permeable bases provide a highly permeable drainage layer within the pavement structural section. A treated permeable layer is generally considered an integral part of the structural section and is to fulfill all or part of the support function normally required of the base layer. If the open-graded layer is not stabilized (or stabilized with asphalt), a question is raised in designing the slab thickness because of the low support properties of the untreated open-graded materials.

A subdrainage analysis and design for a permeable (open-graded) base layer and longitudinal pipe system is provided in appendix C. The subdrainage design recommended by the California Department of Transportation for a permeable base is shown in figure 34. However, because of high deflections, dowels maybe be needed to ensure positive load transfer and long-term (30 to 40 years) performance or until long-term performance studies can determine if and under what conditions they are not needed.

The construction of a drainage layer requires consideration of the lateral extension of the base and the provision of transverse interceptor drains.[49] The California design for transverse interceptor drains is shown in figure 35.

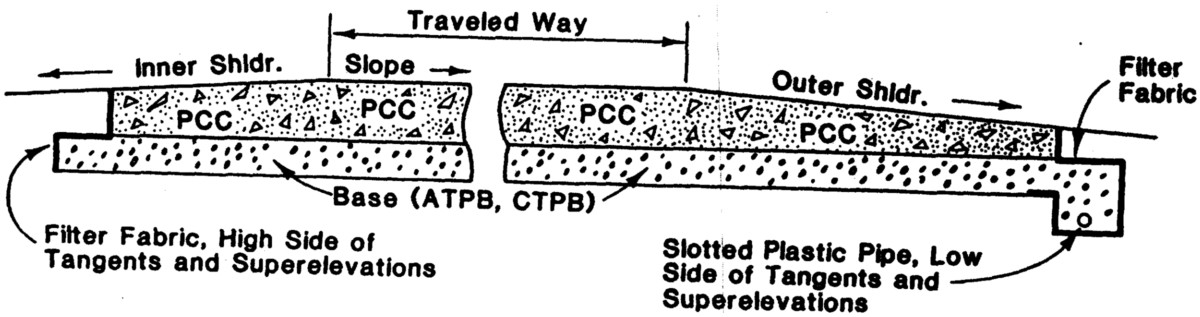
There are two types of outlet systems for the permeable layer. One is side drainage, and the other is longitudinal drainage pipes beneath the shoulders. A longitudinal drain pipe with laterals is recommended for most designs, due to the filling up of the side slope drainage with silt and grass. This may eventually prevent the permeable base from draining water.

The type of shoulders will affect the design of location of the longitudinal drainage pipe. For an AC shoulder, the pipes will be put beneath the inner side of the AC shoulder due to the consideration of rapid drainage of the lane/shoulder joint infiltration of moisture. For tied PCC shoulders, considered to be better for preventing infiltration of water into the lane/shoulder joint, the location of drainage pipe can be designed at the location beneath the outer side of the PCC shoulder. See the illustrations in Figure 34. The subdrainage pipes are usually placed beneath the outside edge of the tied PCC shoulder since the tied lane/shoulder joint is thought as eliminating excessive moisture infiltration.

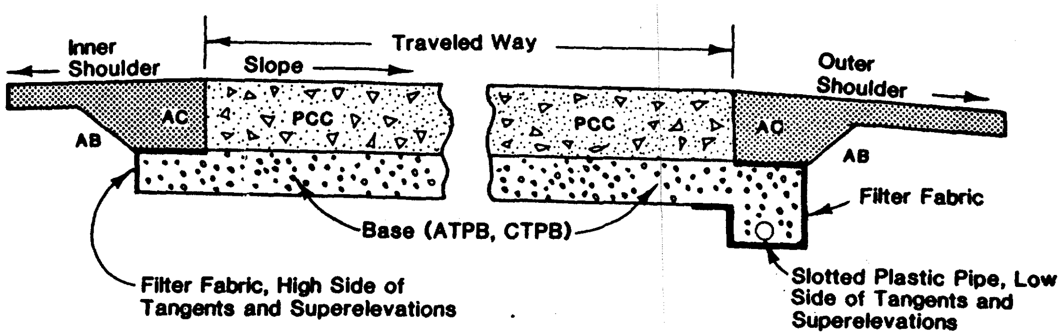
5.1.5 Precoated Dowels

The use of round steel dowels at transverse joints is a proven effective method of increasing load transfer and reducing joint faulting. However, the corrosion of these steel dowels is a common problem caused primarily by deicing salts. Uncoated steel dowels can become badly corroded in as little as 5 years. Several investigators have demonstrated that it may lead to premature pavement distresses including the following:

- Lockup of joints due to high concrete to dowel slip resistance.
- Rupture of reinforcement at nearby transverse cracks due to high tensile stresses.
- Joint spalling due to corrosion pressure buildup.



Concrete Shoulders - Permeable Base



Asphalt Concrete Shoulders - Permeable Base

Figure 34. California Department of Transportation design of permeable base and collector pipes. [49]

Cross Drain Interceptor Details
Longitudinal Section
For Use with Treated Permeable Base (TPB)

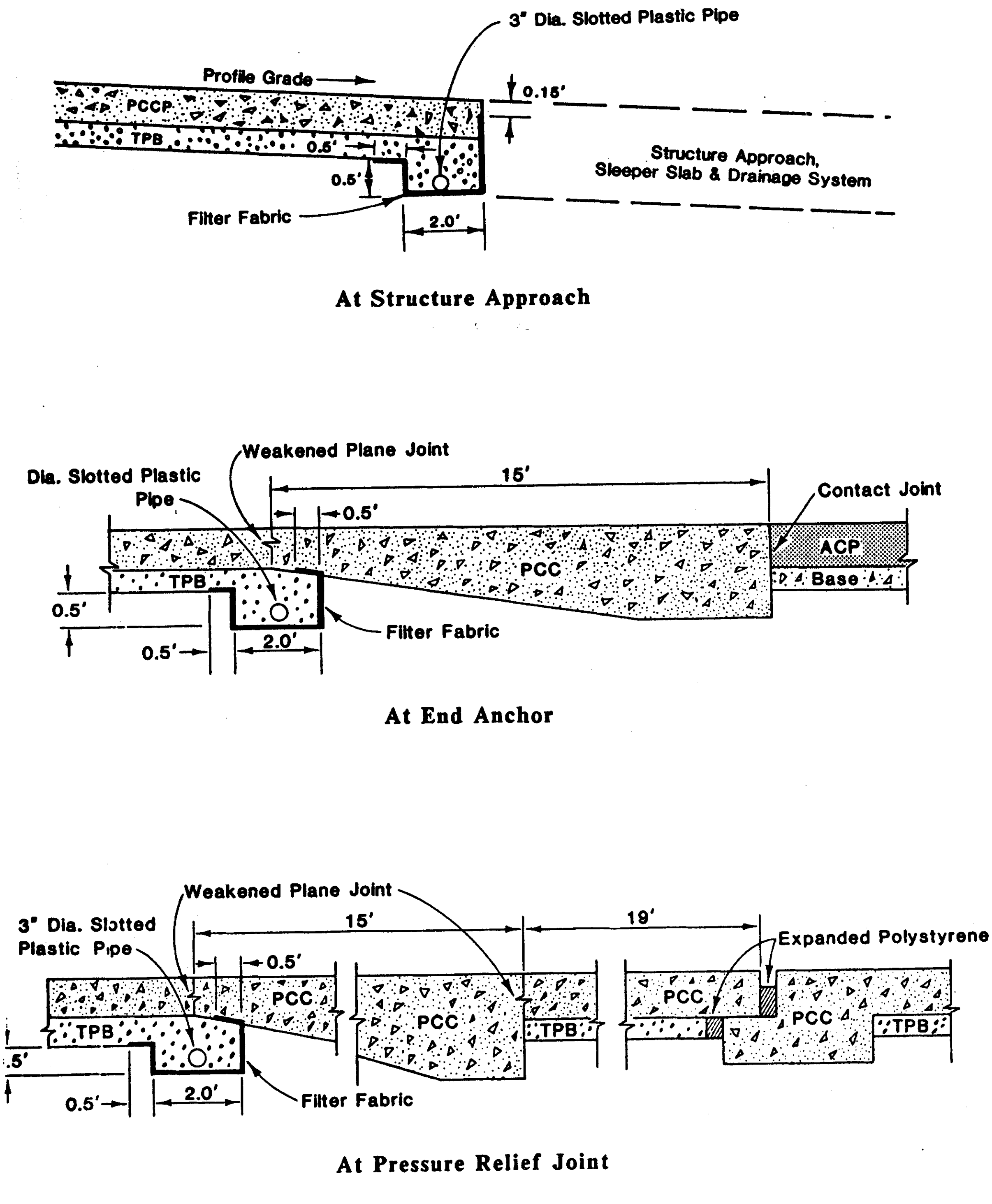


Figure 35. California cross drain interceptor design. [49]

The permanent coating of dowels, therefore, has finally become recognized as an important means of preventing corrosion of dowels. Several types of precoated dowels are available, such as epoxy coated dowels, but without long-term performance data.[79] The only long-term proven performance coatings are stainless steel and Monel. Stainless steel and Monel clad dowels have been used successfully for many years by New Jersey, New York and Michigan. [78,79,80] Dowel corrosion and subsequent lockup problems were eliminated through the use of these dowels. Figure 36 gives the specifications for the Monel dowels used in New Jersey. It is recommended that stainless steel or Monel coatings extend over the entire dowel to control any joint spalling due to dowel corrosion and maintain load transfer durability.

The use of precoated corrosion proof dowel bars is recommended in all designs in the experimental project to eliminate the factor of possible deterioration. Corrosion resistant epoxy coated tie bars should be used also where corrosion is a problem.

5.1.6 Shorter Joint Spacing (JRCP)

The field studies performed under NCHRP Project 1-19 showed that the shorter the joint spacing, the less the transverse joint deterioration per mile.[42] The effect of longer joint spacing of JRCP was very severe, particularly in the wet-freeze climates. A spacing of 40 ft (12.2 m) (currently recommended by many agencies) results in more severely deteriorated joints per mile than any other spacing. The data indicate that a joint spacing of approximately 27 ft (8.2 m) may produce the best long-term joint performance in JRCP from a joint deterioration standpoint.

The NCHRP Project 1-19 joint deterioration model for JRCP indicates that Minnesota's 27-ft JRCP results in much lower joint deterioration per mile than the 40-ft joint spacing.[42]

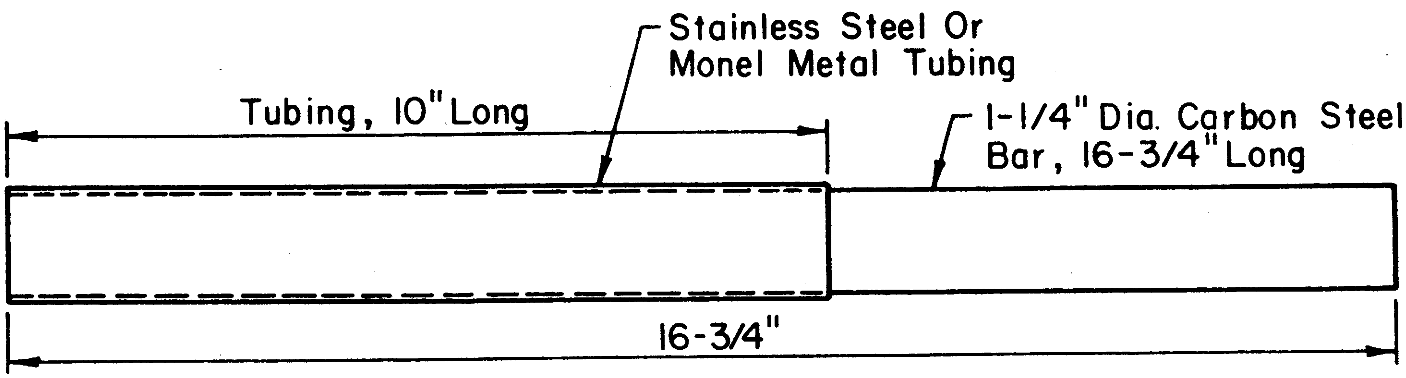
The same study also shows that JPCP, having a much shorter joint spacing, has much less joint deterioration. Thus, it appears that the reduced opening and closing resulting from a shorter joint spacing results in less infiltration and buildup of compression stresses.

5.2 Development of New Rigid Pavement Designs

A set of new and unique rigid pavement designs were developed to be tested in field experimental projects.

The factorial design approach was used to develop the experimental project. Some unique design features were identified as variables in developing the experimental project. A full factorial design containing a minimum of four different test sections (plus some replicate sections and conventional design sections) is recommended.

The set of new unique designs could be utilized in the Specific Pavement Studies (SPS) program under the "New Concepts for Rigid Pavements" experiment of the Long-Term Pavement Performance (LTPP) studies. The research and performance monitoring plan for the experimental projects would be identical to the Strategic Highway Research Program (SHRP) for the LTPP studies, except that additional instrumentation is recommended (such as



The dowels shall consist of either (a) 1-1/4" diameter solid stainless steel bars, (b) 1-1/4" diameter carbon steel bars encased in stainless steel or Monel metal, or (c) 1-1/4" diameter carbon steel bars that have been impregnated with chromium throughout their exposed surface. The stainless steel shall contain not less than 12 percent chromium. If encased in stainless steel or Monel metal, the thickness of the stainless steel or Monel shall be not less than .01 inches, and the tightness of fit shall be such as to preclude the occurrence of corrosion between the stainless steel or Monel and the underlying carbon steel. If rendered corrosion-resistant by impregnation with chromium, the layer of metal which has been so impregnated shall have (a) an average thickness of not less than .009 inch, (b) at no point a thickness of less than .008 inch, and (c) an average chromium content of not less than 20 percent, by weight. The Contractor shall furnish the Engineer with a certification showing that the means employed for rendering the dowels corrosion-resistant complies with the foregoing specification.

The dowels shall not vary in straightness throughout their length in excess of 1/32". The sliding portion of the dowel shall be of uniform cross section, free from burrs, projections, and any other irregularities that would interfere with free movement in the concrete.

Figure 36 Corrosion proof dowel specified by New Jersey.[78]

drainage outflow measurements). The individual cross sections of the rigid pavement design experiments are documented in a two-page format shown in appendix B.

5.2.1 Factorial Design Approach

The factorial design approach is used to determine the independent effect of selected variables, and any interactions between variables. Clearly, factorial designs are the most effective and efficient approach for the development of rigid pavement experimental projects.

A full two-level n-factor factorial design (i.e., 2^n factorial design) requires 2^n experimental sections. Since this number can be fairly large, it is possible to perform only a fraction of a full 2^n factorial design, i.e., the partial factorial design. It is much more efficient to perform a partial factorial design, yet obtain most of the desired information, e.g., the main effects and two-factor interactions, instead of a full factorial design.

The major deficiency of a partial factorial design is that one or more of the effects may become confounded (inseparable) with other effects. Therefore, it is not possible to estimate the individual effect separately from other effects. The following table gives a list of some full or partial two-level factorial designs without main effects or two-factor interactions confounded:

Type of Factorial	No. of Tests Required	No. of Factors Considered	Main Effects Confounded?
2^2 (Full)	4	2	No
2^3 (Full)	8	3	No
2^4 (Full)	16	4	No
2^{5-1} (Partial)	16	5	No
2^5 (Full)	32	5	No

From the table shown above, it is obvious that a half-fraction of a 2^5 factorial design is an efficient factorial design. It takes five variables into account and requires only 16 test sections instead of 32 for a full factorial. Figure 37 gives a 2^{5-1} partial factorial design example with illustration of its variables high/low levels and test section design.

5.2.2 Set of New Rigid Pavement Designs

Two alternative options were developed for the experimental rigid pavement designs project. They are described as follows:

<u>Option A.</u>

<u>Description</u>:

Develop a 2^{5-1} partial factorial designed experiment containing a minimum of 16 test sections. It is possible to directly consider the main effects and two-way interaction effects of five important variables for new rigid pavement designs. The controlled variables are considered and their high/low levels are listed as follows:

2^{5-1} Partial Factorial Designs for JPCP

Variable	High Level (+)	Low Level (-)
1. Dowel Diameter	1.5 ins.	1.0 ins.
2. Transverse Joint Spacing	14 ft.	21 ft.
3. Base Type *	Stablized	Granular
4. Edge Support	Tied PCC Shoulder	AC Shoulder
5. Slab Thickness	12 ins.	9 ins.

* permeable base

(a) Variable high/low levels

Slab Thickness	Edge Support	Base Type	Dowel Diameter + / Trans. Jt. Spacing +	Dowel Diameter + / Trans. Jt. Spacing -	Dowel Diameter - / Trans. Jt. Spacing +	Dowel Diameter - / Trans. Jt. Spacing -
+	+	+	X			X
		-		X	X	
	-	+		X	X	
		-	X			X
-	+			X	X	
			X			X
	-		X			X
				X	X	

X represents a test section.

(b) Test layout

Figure 37. Illustrated example of factorial design: (a) variable high/low levels; (b) test layout.

1. JPCP:

Controlled Variable	High Level (+)	Low Level (-)
1) Slab thickness uniformity	8-12 in Tapered	10 in Uniform
2) Edge support-PCC shoulder	Widened Truck Lane	None
3) Subdrainage	Permeable Drainage Layer	None
4) Dowel diameter	1.5 in	0.0 in
5) Transverse joint spacing	12 ft	18 ft

2. JRCP:

Controlled Variable	High Level (+)	Low Level (-)
1) Slab thickness uniformity	8-12 in Tapered	10 in Uniform
2) Edge support-PCC shoulder	Widened Truck Lane	None
3) Subdrainage	Permeable Drainage Layer	None
4) Dowel diameter	1.5 in	1.0 in
5) Transverse joint spacing	27 ft	40 ft

Test Section Layout:

The high/low levels of these controlled variables set for the 16 sections are layed out as follows:

Controlled Variable	Test Section No. 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1) Dowel diameter	+	-	+	-	+	-	+	-	+	-	+	-	+	-	+	-
2) Transverse joint spacing	+	+	-	-	+	+	-	-	+	+	-	-	+	+	-	-
3) Subdrainage	+	+	+	+	-	-	-	-	+	+	+	+	-	-	-	-
4) Edge support	+	+	+	+	+	+	+	+	-	-	-	-	-	-	-	-
5) Slab thickness	+	-	-	+	-	+	+	-	-	+	+	-	+	-	-	+

These 16 test sections would be constructed consecutively along the same roadway to achieve uniform soil support, traffic loading and climatic conditions. The recommended length for both test section and replicate test section is 1500 ft (457 m) plus a 500 ft (152 m) transition area. The order of the layout of these test sections in the test field must be determined randomly. A few replicates are highly desirable (at least four replicate test sections randomly chosen from any of the 16 designs) and any desired conventional design sections used by the State may also be constructed along the test site at random locations. A control section using the State's standard design must be included for comparative purposes.

Option B.

Description:

Develop a 2^2 full factorial designed experiment containing a minimum of four new unique jointed concrete pavement designs. It is recommended that one such experimental project be constructed in each of the climatic zones in the United States. However, the main objective is to obtain results in each State where the experiment is constructed, to assist them in improving their rigid pavement designs. The full factorial design recommended is shown below with only two major factors which are varied, and all other factors held constant. An individual State could vary additional factors if desired, but these two should be investigated as a minimum to make it possible to analyze the effect of climatic zones across the United States.

If possible, the next variable to be included is load transfer (with or without dowel bars). There is evidence to indicate faulting is minimal with a permeable base and no dowels, and this aspect deserves testing for JPCP. The designs of the individual sections are based on providing positive transverse joint load transfer, reducing JRCP joint movement, and providing positive subdrainage, edge support, reduction in slab curling by shorter slabs and reduction in critical edge stresses and deflections.

Test Section Layout:

The combinations of these variables set for the four test sections are layed out as follows:

	Test Section No.			
Unique Features	1	2	3	4
Main Factors:				
1) Tapered cross section	-	-	+	+
2) Widened truck lane	-	+	-	+
Constant Factors:				
Tied PCC shoulder	+	+	+	+
Asphalt/cement-treated open-graded permeable base	+	+	+	+
Longitudinal drainage pipe	+	+	+	+
Precoated dowel bars	+	+	+	+
27 ft shorter joint spacing (JRCP)	+	+	+	+
15 ft joint spacing (JPCP)	+	+	+	+

These four test sections will be constructed consecutively (in random order) along the same roadway to achieve uniform soil support, traffic loading and climatic condition. The recommended length for both test section and replicate test section is 1500 ft (457 m) plus a 500 ft (152 m) transition area. At least two replicate sections selected randomly and any desired conventional design sections used by the State may also be constructed along the test site at random locations. A control section using the State's standard design must be included for comparative purposes.

Other variables that are considered, but are not shown on the above tables for both alternatives, are the climatic zone and time. When the experimental projects are constructed in different climatic zones, the temperature, moisture and freeze-thaw factors will vary and may impact on performance. Data will be collected over time and traffic to develop performance curves.

Comparison on these two options:

Option A - It is a factorial type designed experiment that directly considers the main effects and two-way interactions of slab thickness uniformity, shoulder type, subbase/drainage, dowels and transverse joint spacing. A one-half (2^{5-1}) partial factorial design, would require 16 test sections (with replicate sections which would add at least 4 sections), plus the construction of any conventional sections that may be desired. This would allow the determination of all of five main effects and ten two-way interactions and would provide a wealth of information for improved design. These test sections would be constructed consecutively along a given highway in each of the climatic zones. This would require a project length of 2000 ft x 20 / 5280 = 7.6 miles (12.2 km) over fairly uniform terrain. It would be desirable to do this in the four main climatic zones.

Option B - Most of the unique design features could be constructed simultaneously in this design. However, due to its small-sized factorial designed experiment, it would consider only two variables. A 2^2 full factorial design would require 4 test sections plus at least 2 replicate sections and any conventional design sections. Sets of these sections could be constructed in all or some of the nine climatic zones. The required length of project is approximately 2000 ft x 6 / 5280 = 2.3 miles (3.7 km) over a uniform terrain.

Option A is the best choice given that there are no restrictions because the results will be widely useful and direct comparisons between the effects of all five design features can be made for the same climate, traffic and soils. The big disadvantage of this design, however, is the required project length and the cost of construction. Option B is much less expensive and some very important results would be achieved in terms of evaluation of its unique rigid pavement features. Furthermore, many more potential project sites could be found in participating States.

After considering the specific limited objectives of this experimental project, Option B is recommended for the experimental rigid pavement designs. Each of the individual cross sections in Option B is detailed in a two-page format in appendix B.

5.2.3 Design Recommendations

The following gives some brief recommendations and references for the design of rigid pavement components.

- Climatic Effects: The experimental project should ideally be constructed in each of the nine climatic zones in the United States. The climatic effects contributing to the distresses of rigid pavement systems are typically in terms of frost damage (low temperature problems), freeze-thaw, thermal curling problems and moisture damage (moisture-accelerated distress). The following are some major concerns for the design of some rigid pavement components due to the climatic effects:

  1. Recommendation for transverse and longitudinal joint dimension design:

     A) Low-modulus silicone sealant or compression seals are is recommended for use in transverse joint sealing in all climatic zones.

     B) Recommendations for width of silicone sealant versus transverse joint spacing are given in figure 38.

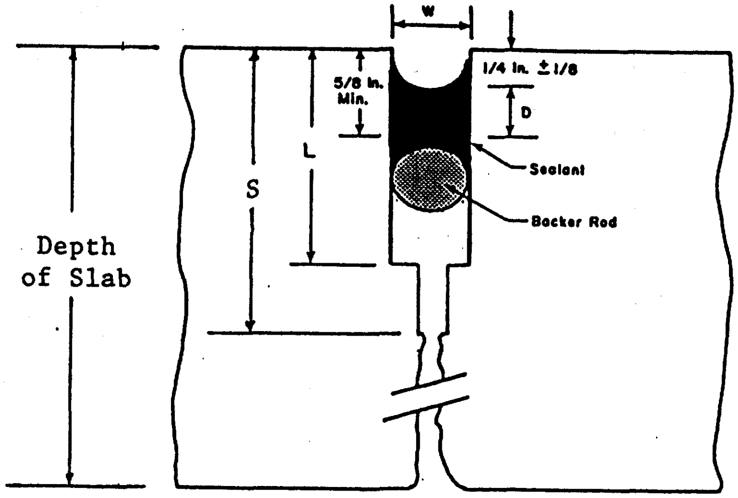
     C) A minimum width of 0.25 in (0.6 cm) and depth of 1/3 slab thickness is recommended for longitudinal joint saw cuts.

  2. The depth of drainage pipe to be installed should be below the frost penetration line to protect the pipe from freezing up and to provide effective collection of water flow from beneath the slab and base.

  3. In deep freeze climates, measures to prevent frost heaving should be taken. The necessary frost protection in the deep freeze climate may be achieved by subgrade excavation and compaction to make it as uniform as possible.

  4. In areas with a very hot climate, there are general problems in hot weather manufacturing, placing and curing of concrete (e.g., shrinkage cracking of concrete). These problems could be limited by better construction process control and curing techniques and/or temperature restrictions.

  5. In all areas and particularly with large concentrations of moisture, extensive care must be given to drainage design.



S = Depth of initial saw cut (1/4 slab depth)

Joint Spacing (ft)	D (in)	L (in)	W (in) Climatic Zones I-A	II-A	III-A	I-B	II-B	III-B	I-C	II-C	III-C
< 15	.25	1.25	.25*	.25	.25	.25	.25	.25	.25	.25	.25
16 - 20	.25	1.25	.25	.25	.25	.25	.25	.25	.25	.25	.25
21 - 25	.25	1.50	.375	.375	.375	.25	.25	.25	.25	.25	.25
26 - 30	.25	1.50	.375	.375	.375	.375	.375	.375	.25	.25	.25

* Recommended silicone joint sealant width in inch.

Figure 38. Width of silicone sealant reservoir recommendations over nine climatic zones.

- **Slab Thickness Design:** The design of the slab thickness for the experimental project must be equal in all climatic zones to make it possible to determine climatic effects.

- **Trapezoidal Cross Section:** See appendix B for dimensions.

- **Widened Truck Lane Design:** See appendix B for dimensions.

- **Tied PCC Shoulders Design:** See appendix B for dimensions.

- **Permeable Base/Drainage Design:** Some guidelines for the design of subdrainage is provided in appendix C. The type of base used must be the same for all experimental sections.

- **Dowel Design:** Dowel design guidelines for the proper use of dowels in jointed concrete pavements is included in section 6.

5.2.4 Research and Performance Evaluation Work Plan

The continuous follow-up performance monitoring of the experimental projects is vital to the study. Only uniform data collection will permit meaingful evaluation of the data. The nationwide pavement data collection procedures developed by the Strategic Highway Research Program (SHRP) for the Long-Term Pavement Performance (LTPP) studies is recommended. The data to be collected are categorized as follows:

- Inventory data.
- Monitoring data.
- Traffic data.
- Environmental data.
- Maintenance data.
- Rehabilitation data.

The data obtained from the experimental projects in each state where they are constructed would be used to:

- Document the performance of each unique design.

- Determine the effect of widened lanes and trapezoidal cross sections in comparison to conventional designs.

- Compare cost-effectiveness of each unique design versus the conventional.

- Analyze the climatic effects if similar experimental projects are constructed in different climatic zones.

5.3 Project Description Form (PDF)

A project description form (PDF) was prepared. It contains three sections including administrative, technical, and research. The PDF should be completed by each highway agency interested in participating in field evaluation of the new rigid pavement design experimental project. The major functions of a PDF are to:

- Provide a standard format to evaluate all submissions from interested States on an equal basis.
- Serve as a comprehensive check list of pertinent information required in each of three sectins, i.e., administrative, technical and research, with minimal follow-up clarifications required.

The PDF requests each interested individual highway agency to provide sufficient key information including the following:

- Name, address and telephone number of principal contacts for follow-up negotiations.
- Information on the type of technical assistance the participating agency may require to perform the performance monitoring requirements.
- Information necessary to properly describe potential sites and environment suitable to evaluate the appropriate designs.
- Information to initiate technical discussions between the participating agency and the sponsoring agency as to by whom and how the follow-up performance monitoring is to be performed.
- Any questions or comments that may be raised about the experimental projects.

In order to plan the work for construction and follow-up monitoring for the experimental projects, a complete PDF should be submitted for final review and acceptance.

The PDF is included in appendix D.

6.0 DOWEL DESIGN TO PREVENT TRANSVERSE JOINT FAULTING

Smooth round dowel bars have been used widely as a mechnical load transfer device in jointed concrete pavements for a long time. The major functions of dowel bars are to prevent faulting, keep corner deflections and slab stresses low, reduce pumping and prevent corner breaks due to loss of a base support. Nonetheless, adequate guidelines have not been established for the design of dowels, based on a mechanistic approach. The main objective of this section is to develop such guidelines for the proper use of dowels in jointed rigid pavement. This was achieved by identifying the most important dowel design factors, and incorporating these into faulting predictive models generated using a mechanistic-empirical approach.

A comprehensive review of the major issues involved in the design of dowels is presented first. This resulted in the formulation of a new method for determining the maximum concrete bearing stress, which combines the theoretical rigor of the Friberg analysis, as well as the results of more recent finite element studies.[93,99] Faulting predictive models are developed using the COPES database for both doweled and undoweled jointed concrete pavements.

6.1 Analytical Methods for Doweled Joints

The first rational procedure for the design of doweled joints in concrete pavements was presented by Westergaard in 1928.[102] This crude but ingenious method enabled engineers to base such decisions as number and spacing of dowels used on theoretical principles, assuming that the load was applied midway between two dowels, and that the deflected shape of the load side of the joint coincided at all points with the basin formed by the unloaded slab. Thus, all dowels were assumed to be perfectly rigid. The background for this method consisted entirely of Westergaard's earlier analytical studies of the one-slab problem, yet two important new conclusions were reached:[98]

- Only the two, or at most four, dowels nearest to the load need be considered as active, since the contribution of more remote bars is negligible.
- Dowels are only effective in reducing the bending stress developed in the loaded slab if they are spaced closely enough ($\leq$ 2 ft (0.6 m)).

These two issues remained the prominent foci of the debate that followed in the next several decades, even to the present day. The Arlington tests provided the first documented opportunity for a field study of dowel performance.[103] This corroborated Westergaard's conclusions, suggesting that a dowel spacing even closer than 2 ft (0.6 m) may be necessary. As indicated by theory, increasing the stiffness of the dowels will enhance their efficiency, but this may also cause a detrimental increase of restraint to longitudinal warping or curling. Thus, dowels that are too stiff may cause more distress in the pavement slab than would result from their complete omission.[104]

Westergaard's method was employed to determine dowel reactions in their independent investigations of the stress condition existing in and around the steel bars.[105,106] Their analytical treatments, however, were based upon another method presented by Timoshenko and Lessels that considers the dowel as an infinite beam encased in an elastic medium.[107] This approach is sensitive to a parameter that is difficult to determine with any degree of accuracy, i.e., the modulus of dowel support, K. Not withstanding early warnings that such calculations "should be taken as significant qualitatively rather than quantitatively" [104], this procedure had been used exclusively in such studies until the introduction of the finite element method in the 1970's.

Credit for the prominence of the Timoshenko analysis is generally given to the theoretical and experimental expositions published in the late 1930's [105,93]. A set of design equations were presented for evaluating dowel deflections, moments and stresses, provided the (shear) force transferred in the dowel could be determined [93]. Thus, the maximum concrete bearing stress, σ_{max}, is given by the formula:

$$\sigma_{max} = K \delta_o \tag{30}$$

where:

K = modulus of dowel support; and
δ_o = deflection of the dowel with respect to the concrete at the face of the joint.

Deflection, δ_o, may be evaluated from:

$$\delta_o = P_t(2 + \beta z)/4\beta^3 E_s I \tag{31}$$

in which:

P_t = shear force acting on the dowel, transferred across the joint;
z = width of joint opening;
E_s = modulus of elasticity of the dowel bar;
I = moment of inertia of dowel bar cross-section,
= $0.25\ \pi\ (d/2)^4$ for round bars, diameter d; and
β = relative stiffness of the dowel-concrete system,
= $\{K\ d\ /\ (4\ E_s\ I)\}^{0.25}$.

In deriving these equations, use was made once again of Westergaard's early theoretical works, leading Friberg to conclude that dowels at distances greater than 1.8 times the radius of relative stiffness of the slab-foundation system, ℓ, from the point of application of the external load were inactive, and did not influence the moment at the load point.[99] Furthermore, the effective dowel shear was assumed to decrease linearly with distance from the point of loading.

Friberg's assumption of a value of 1,000,000 pci (27680 kg/cm^3) for the modulus of dowel reaction, K, for all sizes of dowels elicited considerable discussion. Thus, Grinter postulated that K ranged between 300,000 and 1,500,000 pci (8304 and 41520 kg/cm^3), but also anticipated a "maximum variation of a hundred-fold" in the value of this parameter [106]. Less attention was paid to Friberg's assertion of an effective length of 1.8ℓ, despite the fact that this would not be in accordance with Westergaard's own conclusions.[98] For a typical-value of 36 in (0.9 m) and dowel spacing of 2 ft (0.6 m), Westergaard's assumption that only the two dowels closest to the load are active, would correspond to an effective length of only 1.0ℓ. Other data presented also supported a shorter effective length.[107]

A potential for a real breakthrough in analytical methods for doweled joints was created in the late 1970's with the introduction of the finite element method. Although the capabilities of this versatile numerical tool are far from exhausted even to this day, several very important observations have already been made. Tabatabaie, et al. were among the first to present a finite element model of the doweled joint, and concluded that "only the dowels within a distance 1.0ℓ from the center of the load are effective in transferring the major part of the load."[99,111] It was proposed that a linear approximation to the dowel shear force diagram be used, beginning with a maximum under the load and diminishing to 0 at a distance of 1.0ℓ from this point.

Finite element studies also led to the conclusion that the dowel diameter, d, and concrete modulus of elasticity, E, have a very significant effect on the maximum dowel deflection and concrete bearing stress. Slab thickness, h, and subgrade modulus, k, play a much lesser role. Earlier laboratory investigations by Marcus and Teller and Cashell had also pointed out the same effects.[112,113] The following relationship for the maximum concrete bearing stress, σ_{max}, was developed, "based on the results of two- and three-dimensional" finite element analyses:[111]

$$\sigma_{max} = \alpha \frac{(800 + 0.068E)}{d^{4/3}} (1 + 0.355\, z) \quad s\, P \qquad (32)$$

where:

- E= concrete modulus of elasticity, ksi;
- d= dowel diameter, in;
- z= width of joint opening, in;
- s= dowel spacing, in;
- P= applied wheel load, kips; and
- α= load location coefficient,
  - 0.0091 for edge load;
  - 0.0116 for protected corner load;
  - 0.0163 for unprotected corner load.

6.2 Proposed Method for Maximum Bearing Stress Determination

Recent research has provided additional evidence supporting the conclusion reached by Tabatabaie and others, that the effective length over which dowels are active in load transfer is considerably shorter than was assumed by Friberg. Joints designed assuming that all dowels within 1.8 ℓ from the applied load are effective have exhibited unacceptable performance (e.g. substantial faulting [42]), indicating that a more conservative approach is necessary. A comparison was, therefore, conducted between Tabatabaie's formula (which assumes an effective length of 1.0ℓ) and Friberg's original equation. This is shown in figure 39, for a typical rigid pavement section under a single 9-kip edge load. The assumed percentage of load transferred across the joint ranged from 0 to the maximum value of 50 percent, and the maximum bearing stress was calculated using Friberg's equation, assuming the effective length was 1.8ℓ. As expected, this yields a straight line, whose slope is entirely dependent on the assumed effective length. Therefore, decreasing this length to 1.0ℓ results in a second straight line, located above the 1.8ℓ-line and having a steeper slope. Clearly, a more conservative estimate of the maximum bearing stress is obtained.

The actual percentage of load transferred across the doweled joint may be estimated by comparison of Friberg's predictions to the calculated maximum bearing stress according to Tabatabaie, <u>et al</u>.[111] This is also shown in Figure 39, for the three loading locations considered, i.e. loading at an edge, a protected corner and an unprotected corner. It is apparent that a high assumed percentage of load transferred (close to 50 percent) leads to a conservative estimate of the maximum bearing stress. A value of 45 percent was adopted in this study.

It is interesting to note that both the Friberg and Tabatabaie formulae for the determination of maximum bearing stress, can be rewritten as:

$$\sigma_{max} = A(\text{pavement}) * B(\text{load}) \qquad (33)$$

The first term, A, is entirely determined by the pavement system characteristics, while the second term, B, is the transferred load. It would be reasonable to expect that at least the A-term should be the same according to the two equations. This, however, is not true, as illustrated by the following calculations for the case considered above:

(i) <u>Tabatabaie</u>:

$$A = \frac{800 + 0.068E}{1000\ d^{4/3}} (1 + 0.355\ z) \qquad (34)$$

$$= 1.6849 \text{ (units unclear)}$$

(ii) <u>Friberg</u>:

$$A = \frac{K(2 + \beta z)}{4 \beta^3 E_s I} \tag{35}$$

$$= 2.5812 \text{ in.}^{-2}$$

It is noted that the discrepancy in this case is of the order of 35 percent. Friberg's term is considered superior, however, since it is theoretically based, and dimensionally consistent. Parameters K and E do not enter Tabatabaie's formula. Dowel support was not explicitly prescribed in his three-dimensional finite element analysis, while a value of 1,500,000 pci (41520 kg/cm^3) was assumed in his two-dimensional analysis. A value of E_s of 29,000,000 psi (2,039,000 kg/cm^2) was assumed by Tabatabaie.

Turning now to the load B-term in these equations, it is possible to express this as follows:

$$B = P_t = P * \%TL * f_d \tag{36}$$

where:

P	=	applied wheel load;
%TL	=	percent transferred load across joint; and
f_d	=	a dimensionless distribution factor indicating how much of the transferred load acts on any given (usually on the critical) dowel bar.

Note that the distribution factor, f_d, does not depend on the amount of load transferred but is entirely the consequence of the assumptions regarding the effective length and the linear diminution of the dowel shear forces with distance from the applied load.

A recently modified version of ILLI-SLAB was used to compute the vertical shear at each dowel bar across the slab for three different load positions (a single edge load, a single corner load, and a single axle load located at the corner). The distribution factor, f_d, was computed by dividing shear at each dowel by the total shear across the traffic lane. The variation of the distribution factor under an edge, corner and single axle load is shown in Figure 40 for the pavement section considered above.[121]

From observation, the distance until there is practically no vertical shear on a dowel (e.g., the effective length) for unprotected corner loading is of the order of 1.0 ℓ. The effective length for an edge loading is also of the order of 1.0 ℓ, not the 1.8 ℓ assumed by Friberg.

The results show that the corner loading position produces the greatest dowel shear load. Therefore, this is the critical dowel that should be considered in design. These results may be used directly in the computation of a more accurate dowel bearing stress for this dowel by placing a single wheel load at the corner and using an effective length of 1.0 ℓ to determine the number of effective dowel bars.

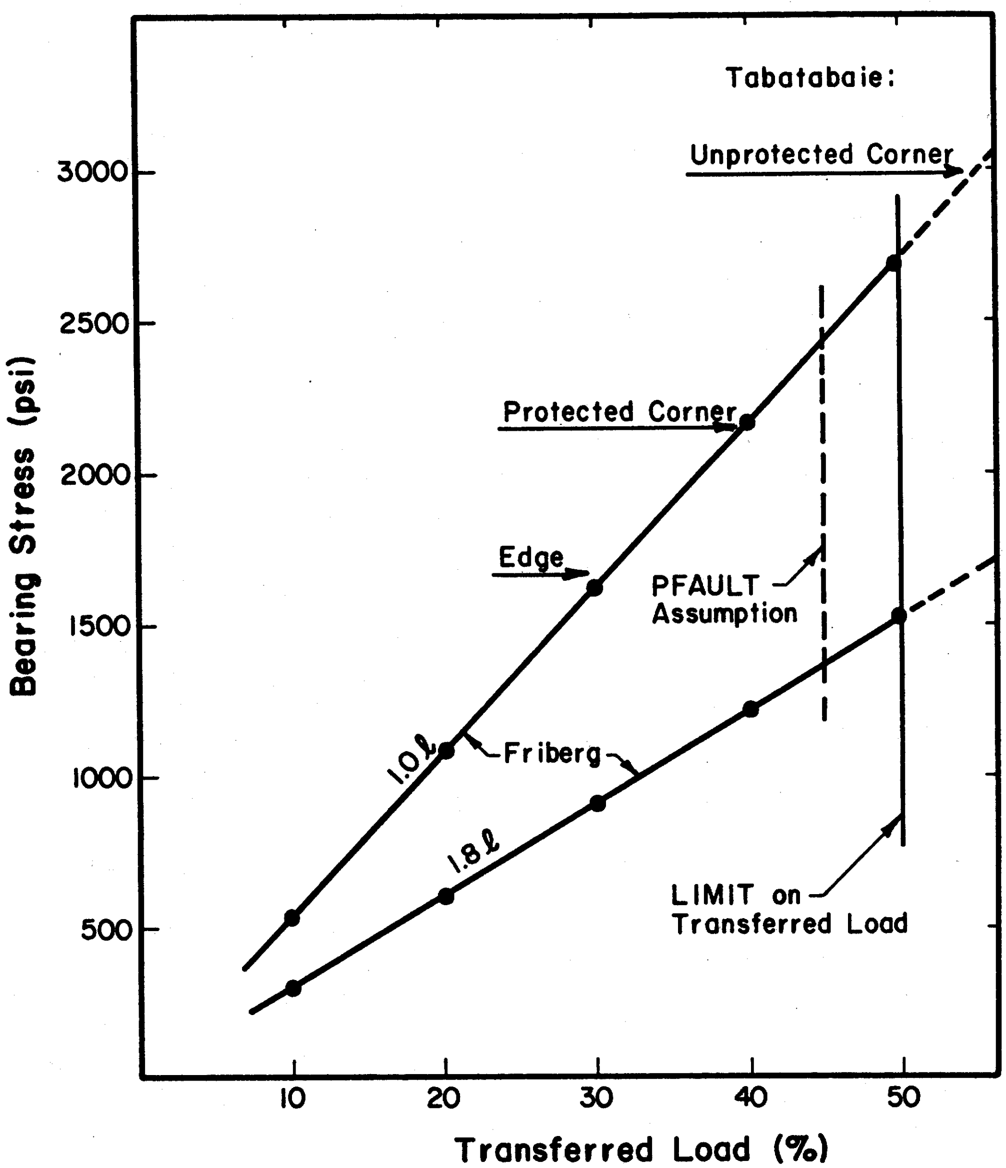
Using this procedure to determine the f_d-value for the load B-term, and Friberg's pavement A-term, the maximum bearing stress may be determined in a manner that accounts both for the location of the load and for the rest of the parameters entering Friberg's theoretical development.

Results obtained during this investigation using ILLI-SLAB indicates that the %TL is only slightly affected by load location, and is generally about 42 $\pm$ 1 percent. The assumed value of 45 percent is, therefore, slightly conservative, as desired.

The sensitivity of the proposed method for determining the maximum bearing stress was investigated for a wide range of the parameters involved. The results are shown in figures 41 through 44. Dowel diameter clearly emerged as the most important of the variables considered. As expected, bearing stress decreases as dowel diameter increases. The reduction in bearing stress is particularly dramatic for dowels with diameters less than 1.5 to 2 in (3.8 to 5.1 cm). The sensitivity of the maximum bearing stress to the rest of the parameters entering its determination is considerably smaller. It was observed, however, that the solution is more sensitive to those variables related to the dowel bars used (i.e., dowel diameter, spacing and modulus of support) than to the pavement system characteristics (e.g., slab modulus and thickness, subgrade modulus, and joint width).

6.3 Development of Mechanistic-Empirical Faulting Predictive Models

The amount of transverse joint faulting for rigid pavements can be predicted by developing mechanistic-empirical models from a database containing in service pavement data. The NCHRP Project 1-19 (COPES) database was used in this study, along with several mechanistically derived variables.[42] Both linear and nonlinear multiple regression techniques from the SPSS software package were used for the statistical analysis effort.[43] Three mechanistic variables, i.e., concrete dowel bearing stress, joint opening and corner deflection, were computed and used as independent variables. This results in a more realistic and accurate prediction of transverse joint faulting in both doweled and undoweled pavements. Thus, guidelines for dowel design based on a mechanistic-empirical approach may also be formulated.



dowel diameter = 0.75 in
dowel spacing = 12 in
modulus of dowel support
= 1.5×10^6 pci
Young's modulus, steel
= 29×10^6 psi

slab thickness = 10 in
Young's modulus, slab = 4×10^6 psi
Poisson's ratio, slab = 0.15
radius of relative stiffness = 51 in
joint opening = 0.2 in
wheel load = 9000lbs

Figure 39. Bearing stress according to Friberg and Tabatabaie.

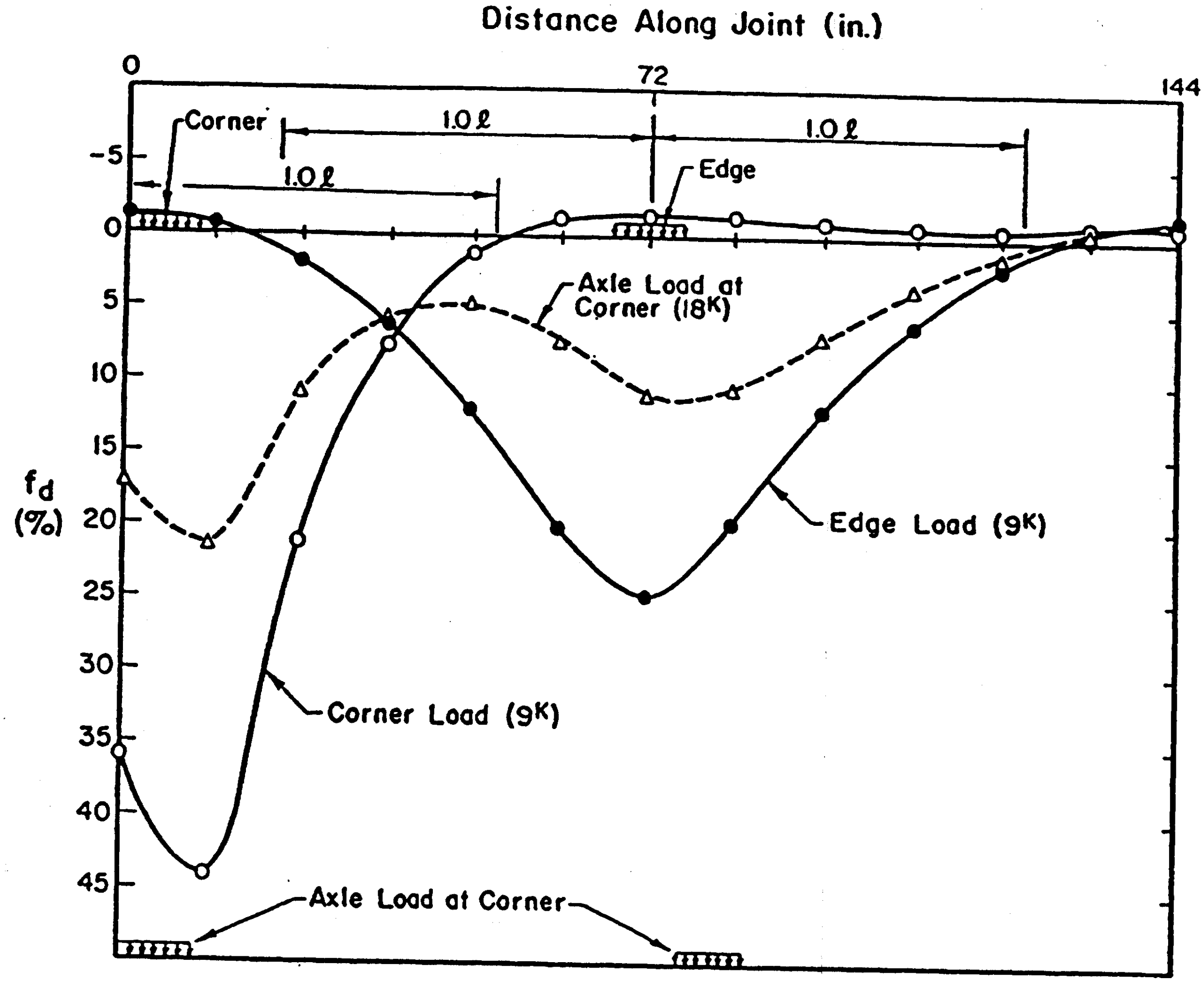


Figure 40. Distribution factor, f_d, from ILLI-SLAB.

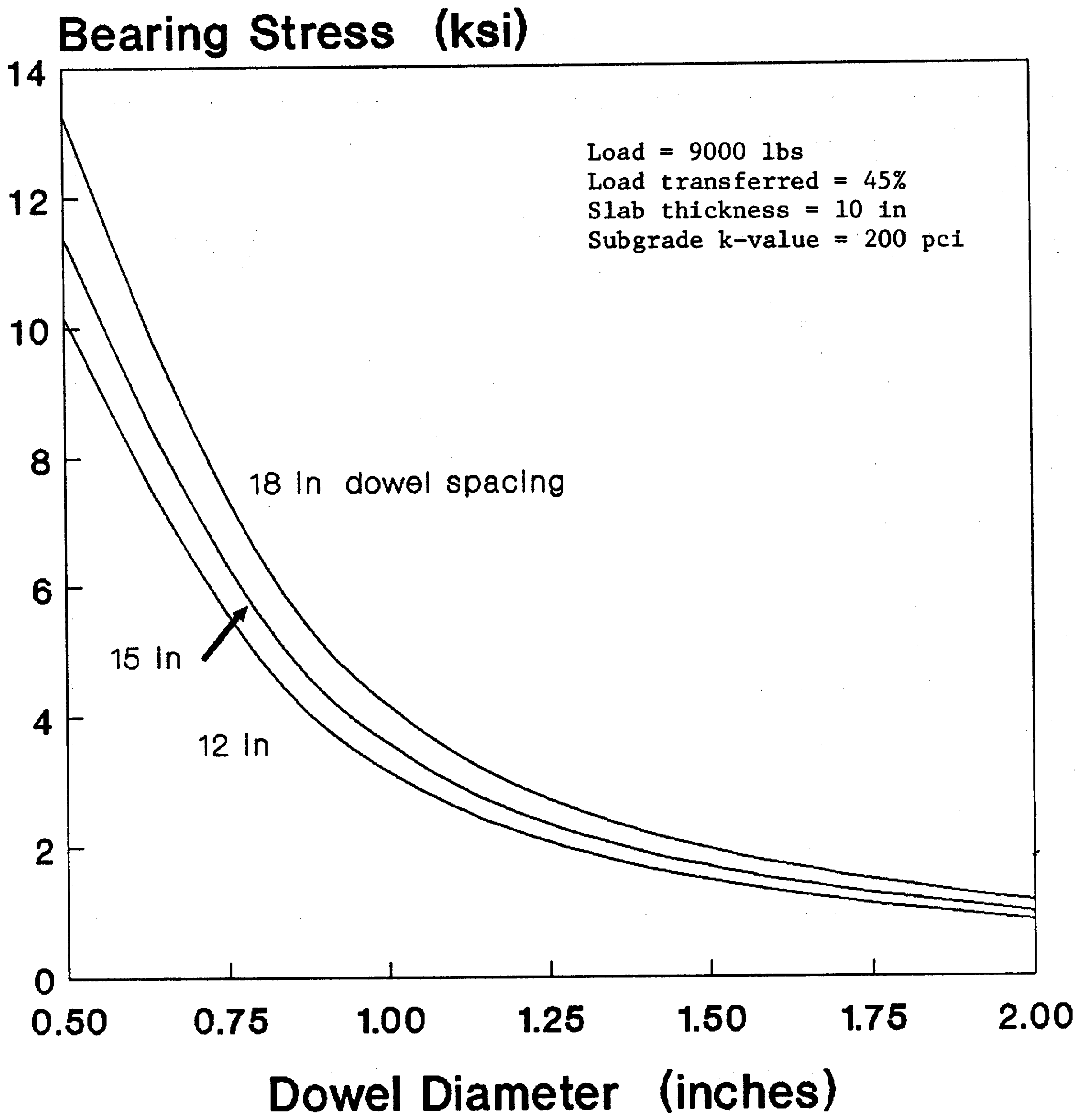


Figure 41. Sensitivity of bearing stress vs. dowel diameter with varying dowel spacing.

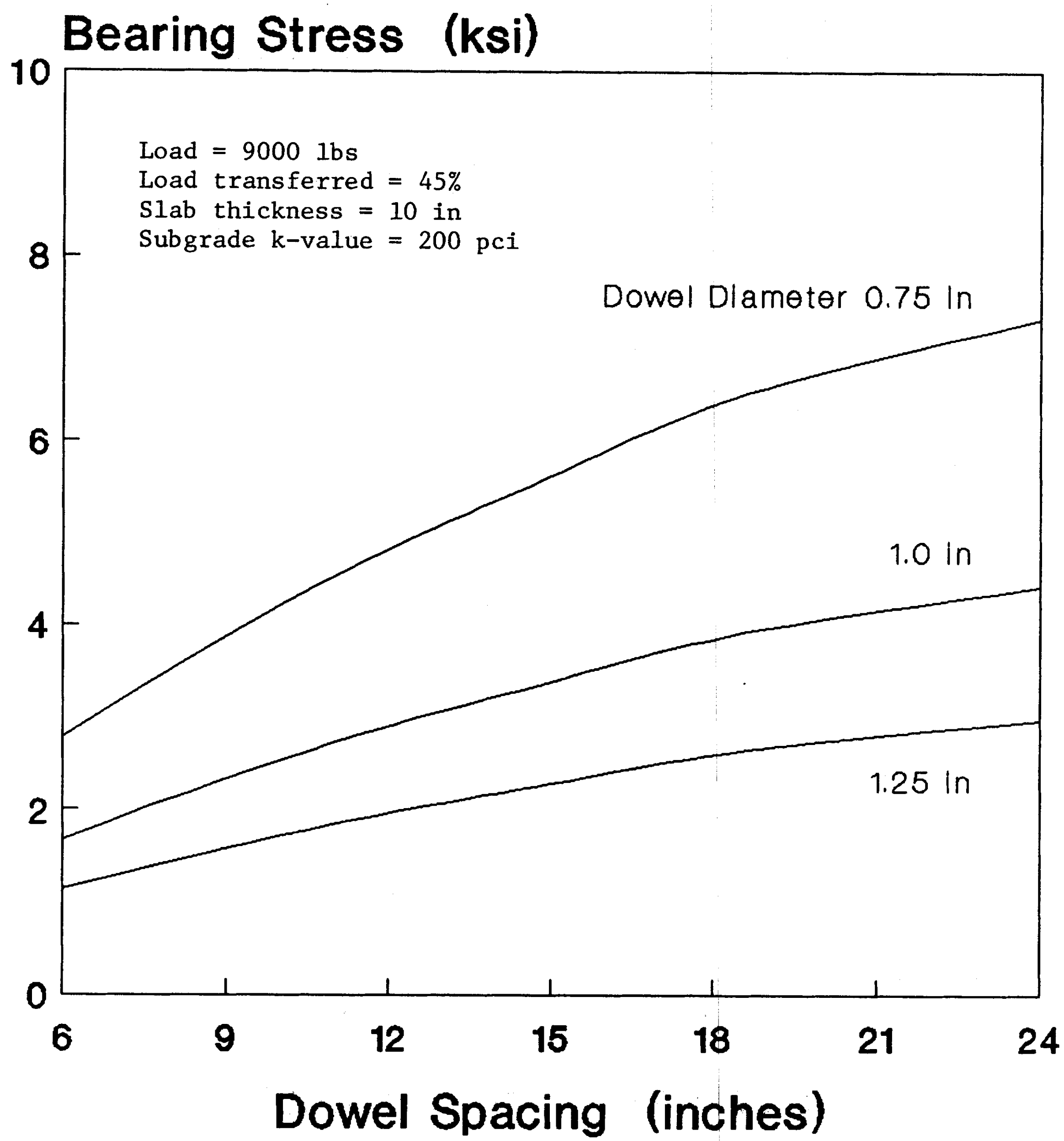


Figure 42. Sensitivity of bearing stress vs. dowel spacing with varying dowel diameter.

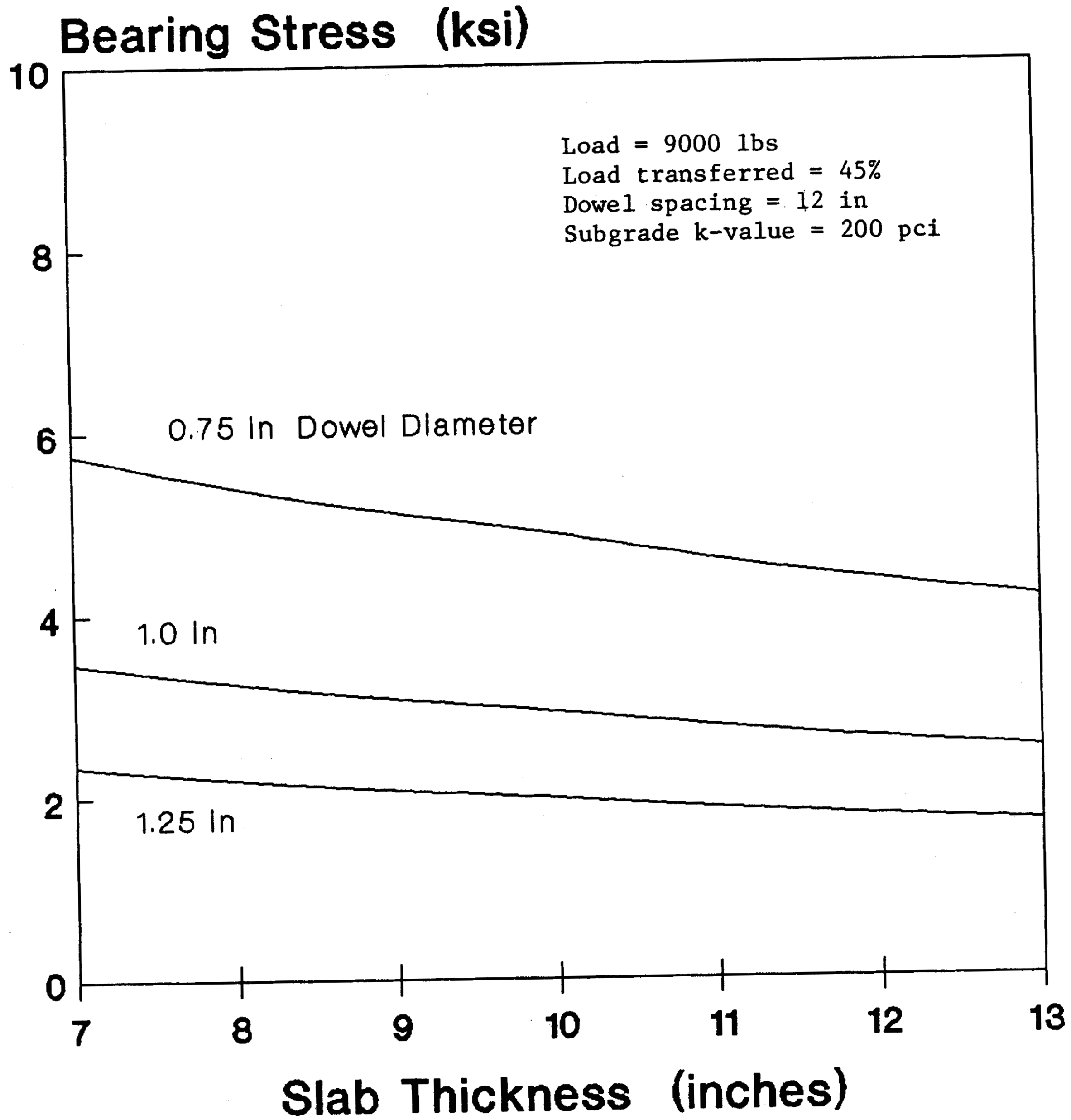


Figure 43. Sensitivity of bearing stress vs. slab thickness with varying dowel diameter.

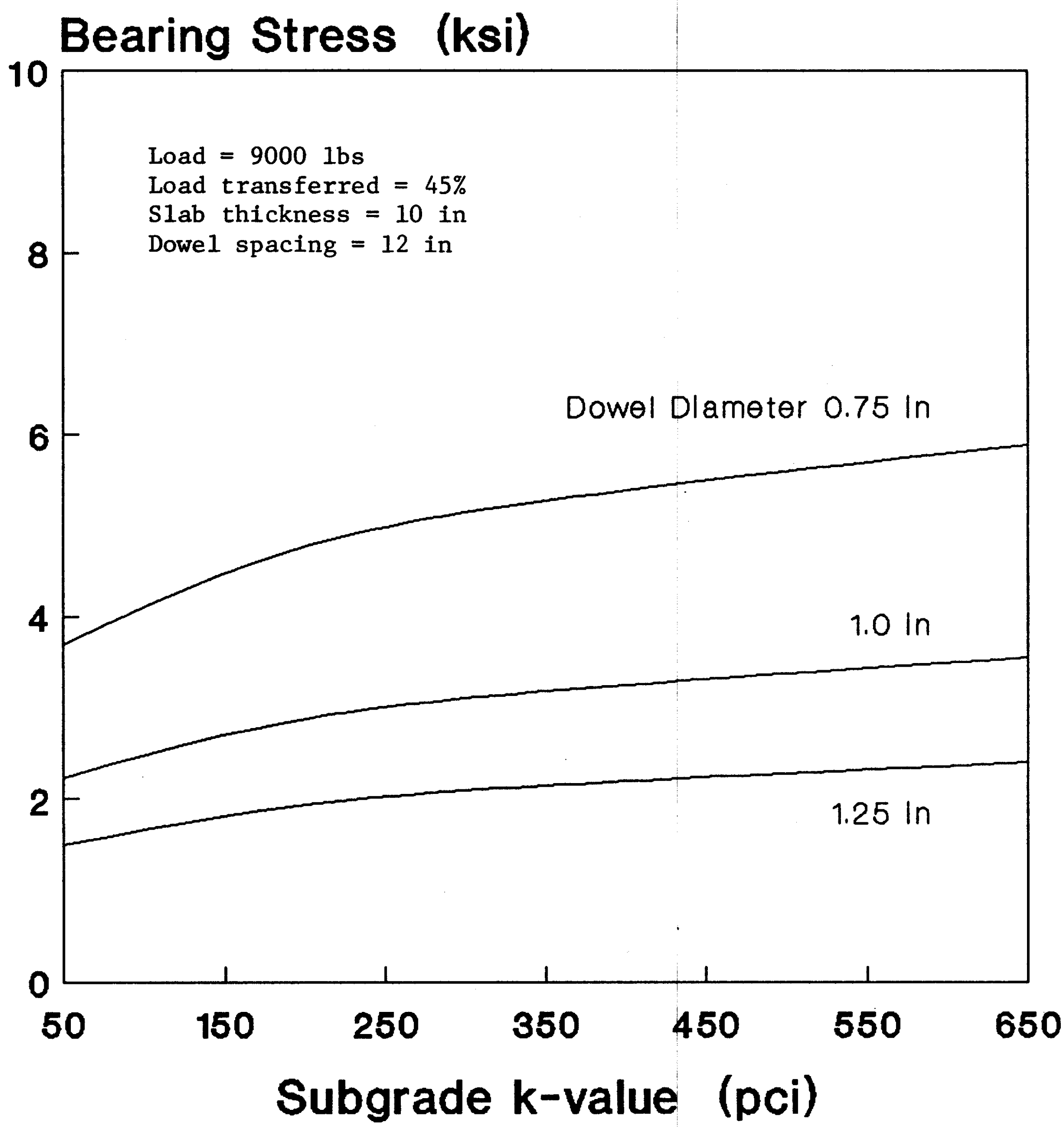


Figure 44. Sensitivity of bearing stress vs. subgrade k-value with varying dowel diameter.

6.3.1 Predicted Faulting for Doweled Pavements

The COPES database contains doweled pavement sections from the following four States: Illinois, Louisiana, Minnesota and Nebraska. The doweled pavement sections in the database consist of two types of jointed concrete pavements, i.e., plain and reinforced. Multiple linear regression was utilized to determine which independent variables significantly affected actual faulting. A knowledge of the faulting versus traffic loading relationship and nonlinear regression was then used to develop a model for predicting faulting in doweled pavements, following the same procedure as was used to generate the COPES predictive equations.[42]

The maximum concrete bearing stress was computed using the new proposed method of calculation for an 18-kip single-axle load acting at the corner of each pavement section considered. The transferred load was assumed to be 45 percent of the applied (18-kip) axle load.

Joint opening and closing is primarily the function of the following factors: transverse joint spacing, type of base material (friction factor), temperature range, concrete thermal coefficient and PCC slab drying shrinkage coefficient. Variation in joint opening from joint to joint in a given pavement is high, with a coefficient of variation of 40 percent computed on one project.[114] Despite the many complexities involved, mean joint opening over a yearly or daily time period can be computed approximately using the following expression:[46]

$$\text{OPENING} = CL\,[\,\alpha\,\Delta T + e\,] \tag{38}$$

where:

OPENING = transverse joint opening caused by temperature change ΔT and drying shrinkage of PCC;

α = thermal coefficient of contraction of PCC (generally 9 - 10.8 x 10^{-6} /°C);

e = drying shrinkage coefficient of PCC (approximately 0.5 - 2.5 x 10^{-4} strain);

L = joint spacing, in;

ΔT = temperature range (maximum mean daily air temperature in July minus minimum mean daily air temperature in January), °C; and

C = adjustment factor due to subbase/slab frictional restraint (0.65 for stabilized subbase, 0.80 for granular subbase), determined experimentally.

The size of joint opening used in the concrete bearing stress computation, however, was computed using the half value of mean temperature range for each pavement section to consider a more realistic average condition of joint opening. All other variables used in the regression equation, such as traffic, slab thickness, joint spacing and effective subgrade k-value, etc., were directly input from the NCHRP 1-19 database.

The extended AASHO Road Test sections on I-80 (1962-1974) were initially selected for analysis to check the significant parameters relating to faulting in doweled jointed concrete pavements. This database includes a range of key variables, such as: cumulative 18-kip ESAL from 1.03 to 12.27 million per truck lane; slab thickness from 8 to 12.5 in (10.3 to 31.8 cm); dowel diameter from 1.0 to 1.63 in (2.5 to 4.1 cm); and transverse joint spacing from 15 to 100 ft (4.6 to 30.48 m). The following non-linear model was developed based on data from 50 extended AASHO Road Test sections:

$$FAULT = ESAL^{0.2692} [-2.995 + 0.00779\ BSTRESS^{0.4527} + 2.766\ JSPACE^{0.00676}] \quad (39)$$

where:

ESAL = cumulative 18-kip ESAL in design life (applied over 16 years)
BSTRESS = maximum concrete bearing stress by new method, psi
JSPACE = transverse joint spacing, ft

Statistics: $R^2 = 0.612$
SEE = 0.041 in
n = 50

This model indicates that cumulative 18-kip ESAL, concrete bearing stress and joint spacing all have strong effect on the transverse joint faulting. It was, therefore, used as an initial model to develop a model using the complete nationwide database for doweled pavements. The final faulting predictive model established for doweled pavements is as follows:

$$FAULT = ESAL^{0.5398} [2.128 + 0.00296\ BSTRESS^{0.4584} + 0.000493\ JSPACE^{0.9993} - 2.066\ KVALUE^{0.0136}] \quad (40)$$

where:

KVALUE = effective modulus of subgrade reaction, pci

Statistics: $R^2 = 0.513$
SEE = 0.054 in
n = 268

(Note: See Appendix E for an updated version of this equation based on the inclusion of additional field data in the data base).

A plot of actual versus predicted faulting for the AASHTO and nationwide models is shown in figures 45 and 46, respectively.

Several climatic variables (e.g., precipitation and freezing index) were then introduced into the model generating process, but surprisingly they did not show any statistical significance. Therefore, they were not included in the model. The following is a list of the major deficiencies of this model:

- Perhaps due to the limited number of climatic zones in the database, climatic variables were not directly included in the model.

- A variety of other situations existed, in which there was not sufficient range of some of the variables (e.g., permeable base type, subgrade type, edge support and subdrainage).

Therefore, the user must be alert not to use this model to predict faulting by undue extrapolation beyond the data range used in its generation. This is particularly true for open graded drainable bases.

A sensitivity analysis of the four input variables was performed on the model. All independent variables are correlated logically with faulting as shown below:

Variable Change	Effect on Faulting
Increase traffic (ESAL)	Increase
Use of stabilized base (rather than granular base)	Decrease
Increase temperature range (TRANGE)	Increase
Increase joint spacing	Increase
Increase joint opening	Increase
Increase slab thickness	Decrease
Increase dowel diameter	Decrease
Increase concrete bearing stress	Increase
Increase k-value	Decrease

An example of the effect of these parameters is shown in table 66. A set of standard conditions were used in obtaining these plots, and one variable was varied at a time. Plots illustrating the sensitivity of the variables in the model are shown in figures 47 to 51. Among the four parameters in the final model, the sensitivity analysis shows that concrete bearing stress and dowel diameter have the most significant impact on joint faulting. The sensitivity analysis for the model shows that for a certain pavement design condition (e.g., certain joint spacing, slab thickness, effective subgrade k-value, etc.) the size of dowels is critical to control transverse joint faulting. However, dowels are costly and it may be cost effective to utilize other design features (e.g., shorter joint spacing, thicker slabs, permeable base/subbases) to control faulting more economically under certain climatic and traffic conditions.

6.3.2 Predicted Faulting for Undoweled Pavements

Load transfer in undoweled pavements is accomplished by aggregate interlock. The degree of load transfer by aggregate interlock is affected by the size of the joint opening (joints that open more fault more), the effective k-value at the top of the base, the thickness of slab, the coarseness and angularity of aggregates used in the PCC and the number of load repetitions.

The slab corner deflection is another important factor contributing to transverse joint faulting. As joint opening increases, aggregate interlock will lose its ability to transfer load efficiently. This causes a greater corner deflection, and results in greater pumping and faulting.[42] The maximum deflection under corner loading was computed using the following Westergaard equation:[98]

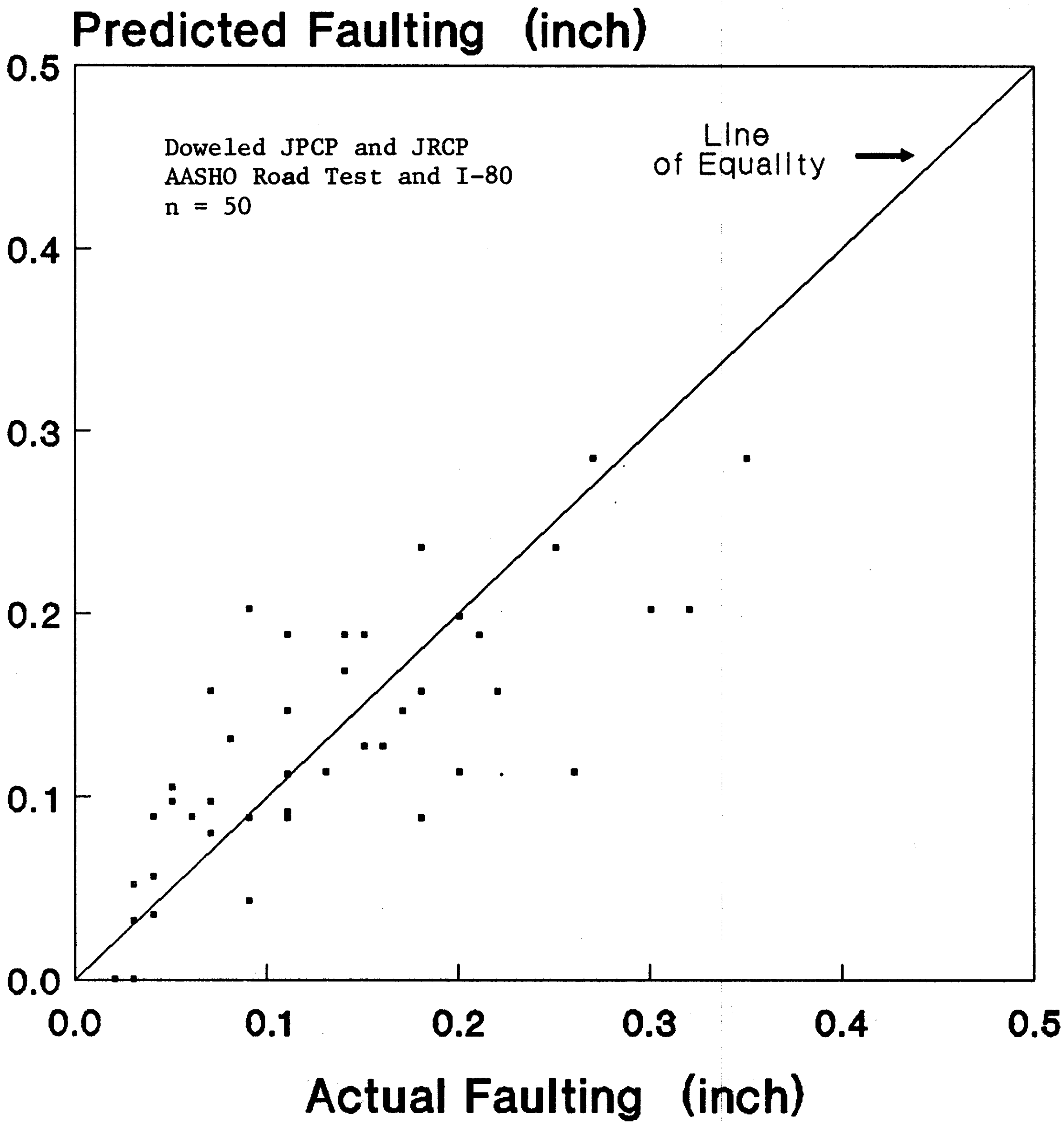


Figure 45. Predicted vs. actual faulting for model using AASHO Road Test sections for doweled pavements (PFAULT version 1.0).

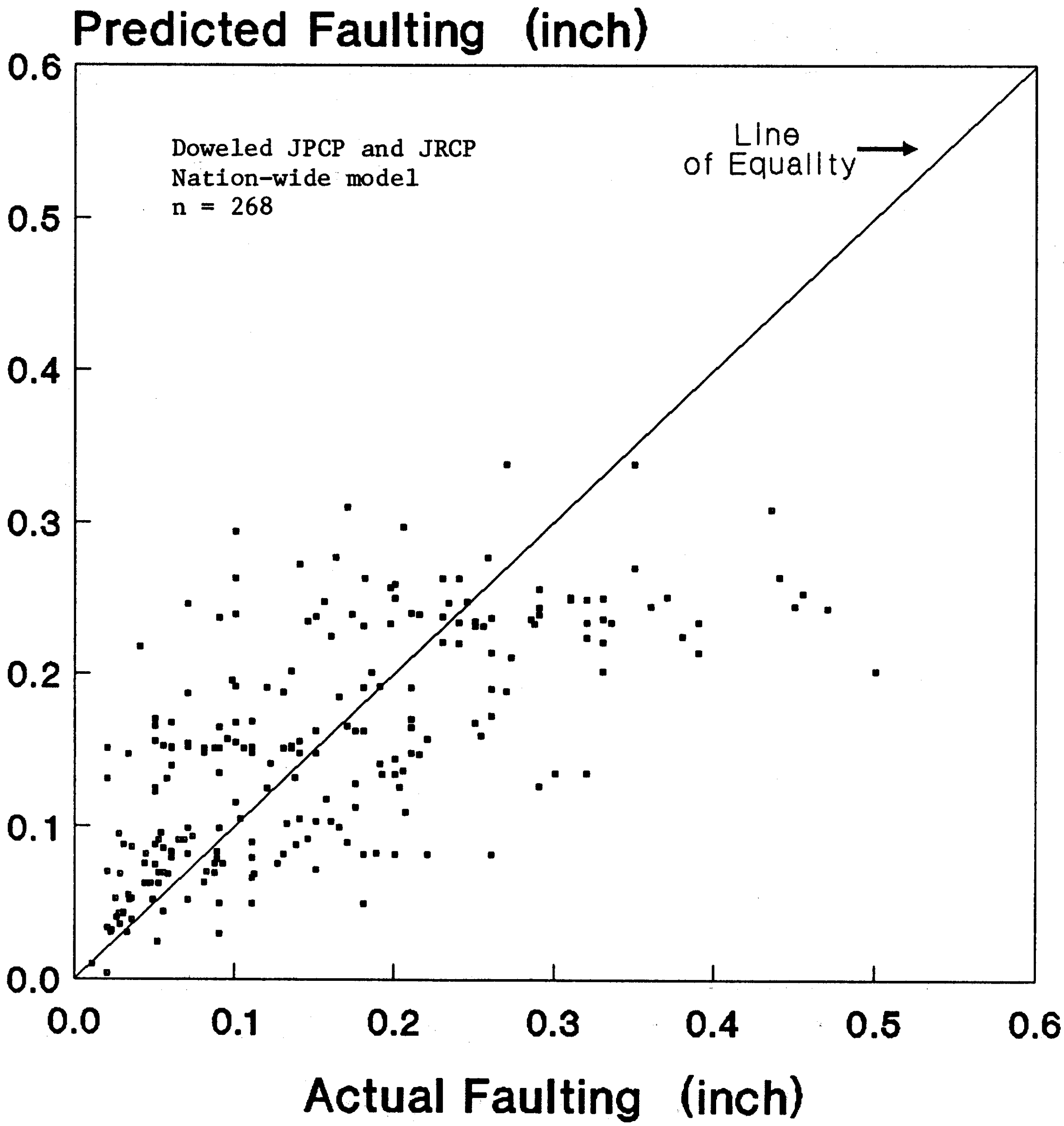


Figure 46. Predicted vs. actual faulting for nationwide model for doweled pavements (PFAULT version 1.0).

Table 66.

Example sensitivity analysis of nationwide faulting predictive model for doweled pavements, Version 1.0.

Design Parameter	Change in Parameter	Change in Faulting (in)
Traffic (18-kip ESAL), million	10 to 20	0.10 to 0.15
Dowel Diameter, in	1.0 to 1.25	0.10 to 0.04
Transverse Joint Spacing, ft	15 to 40	0.10 to 0.15
Slab Thickness, in	10 to 12	0.10 to 0.08
Effective Subgrade k-value, pci	200 to 400	0.10 to 0.05

Note: 1. Concrete bearing stress is a function of dowel diameter, transverse joint spacing, slab thickness and subgrade k-value, etc.

2. A set of standard condition was as follows:
   Traffic (18-kip ESAL) = 10 million
   Dowel bar diameter = 1 in
   Transverse joint spacing = 15 ft
   Effective subgrade k-value = 200 pci
   Slab thickness = 10 in
   Temperature range = 30 ^{o}C

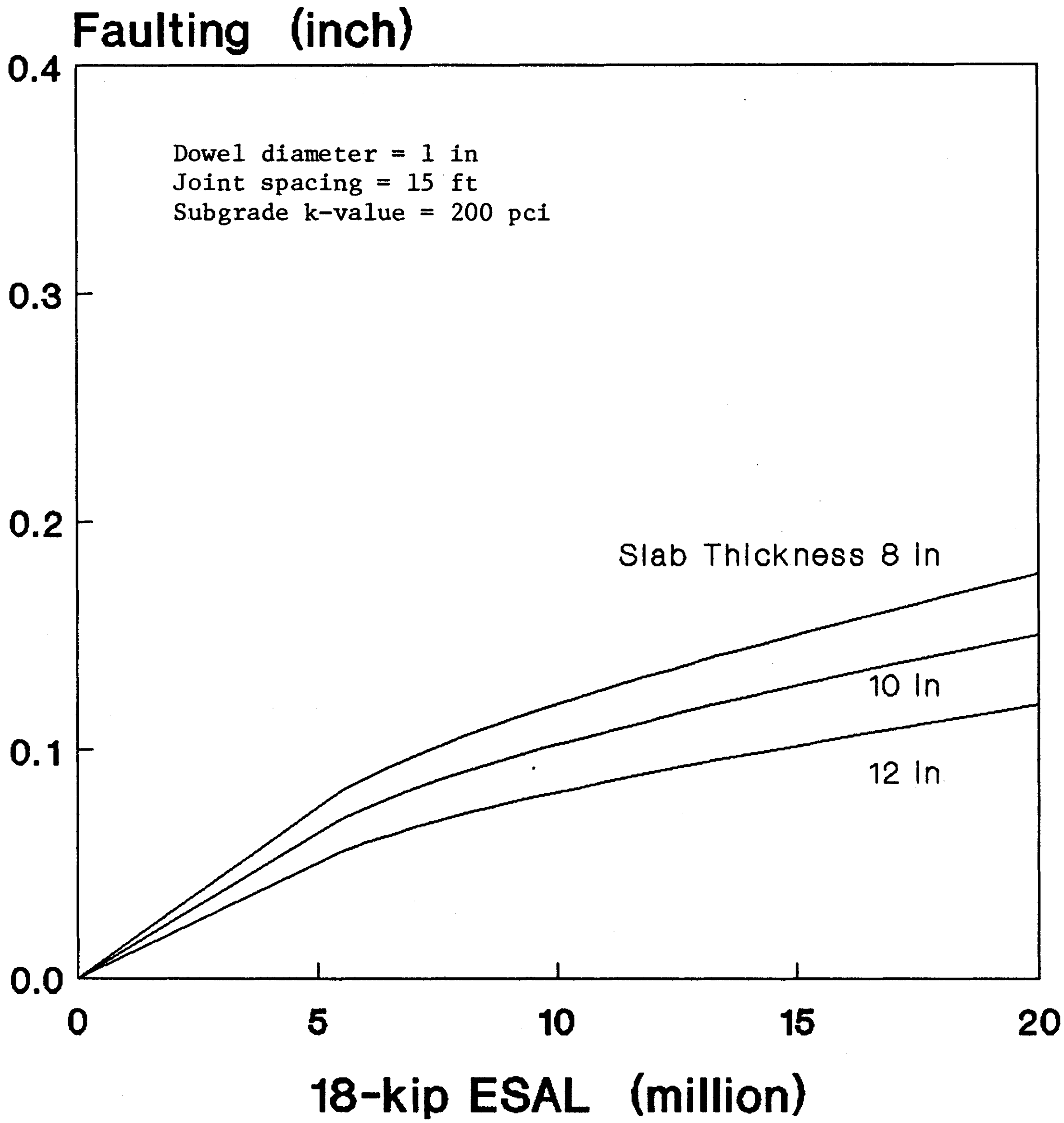


Figure 47. Sensitivity of faulting vs. cumulative 18-kip ESAL with varying slab thickness (PFAULT version 1.0).

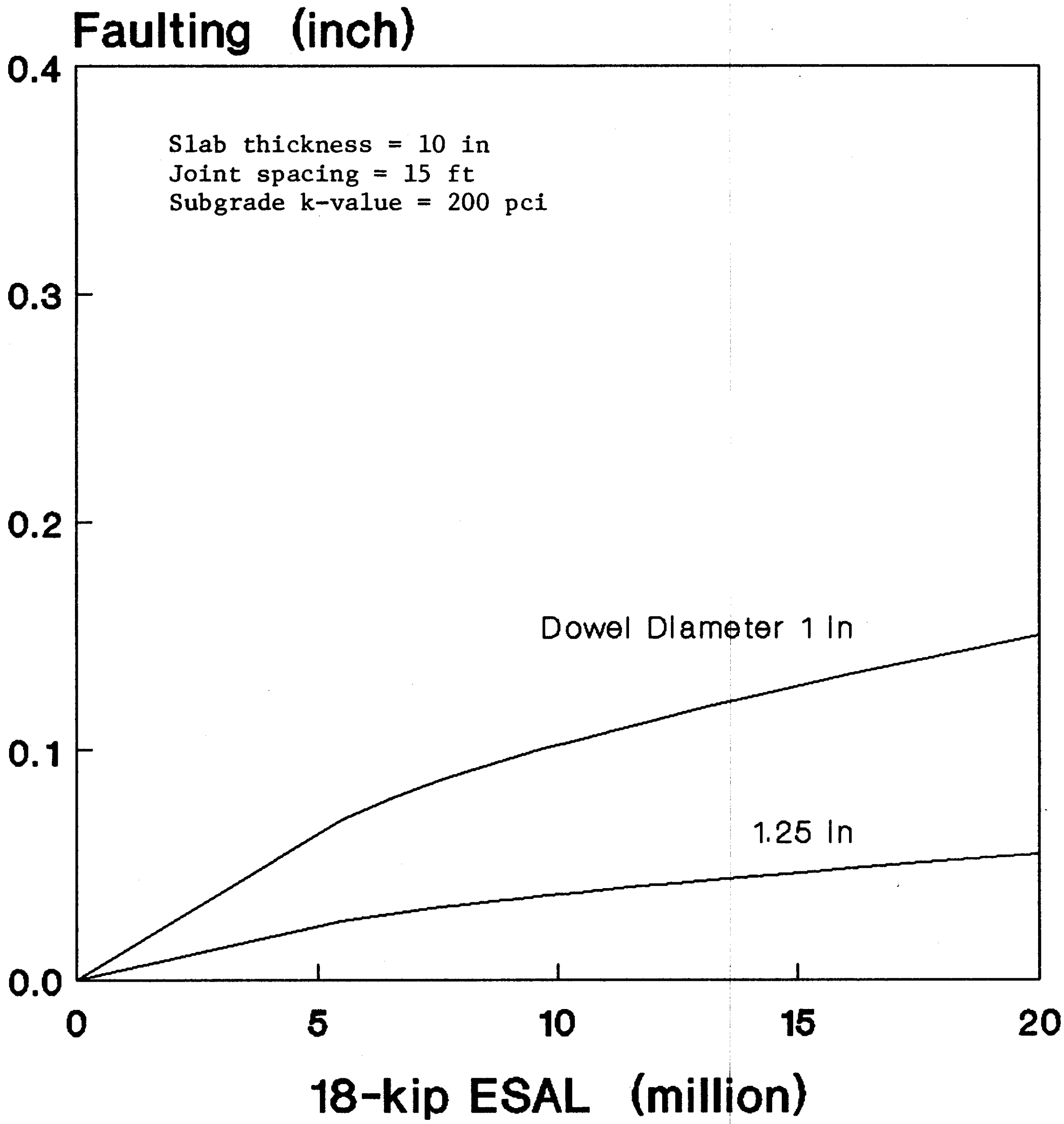


Figure 48. Sensitivity of faulting vs. cumulative 18-kip ESAL with varying dowel diameter (PFAULT version 1.0).

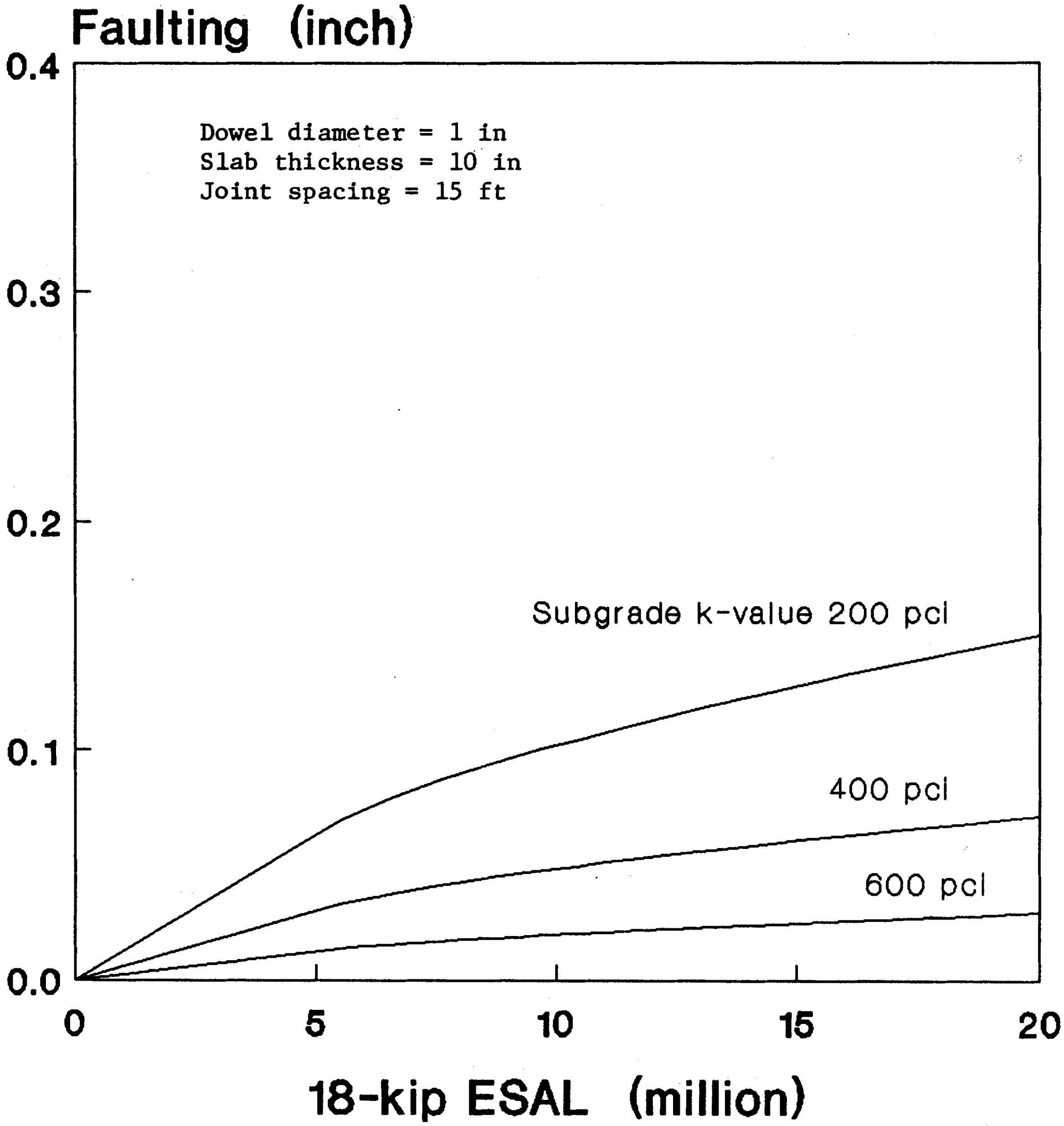


Figure 49. Sensitivity of faulting vs. cumulative 18-kip ESAL with varying subgrade k-value (PFAULT version 1.0).

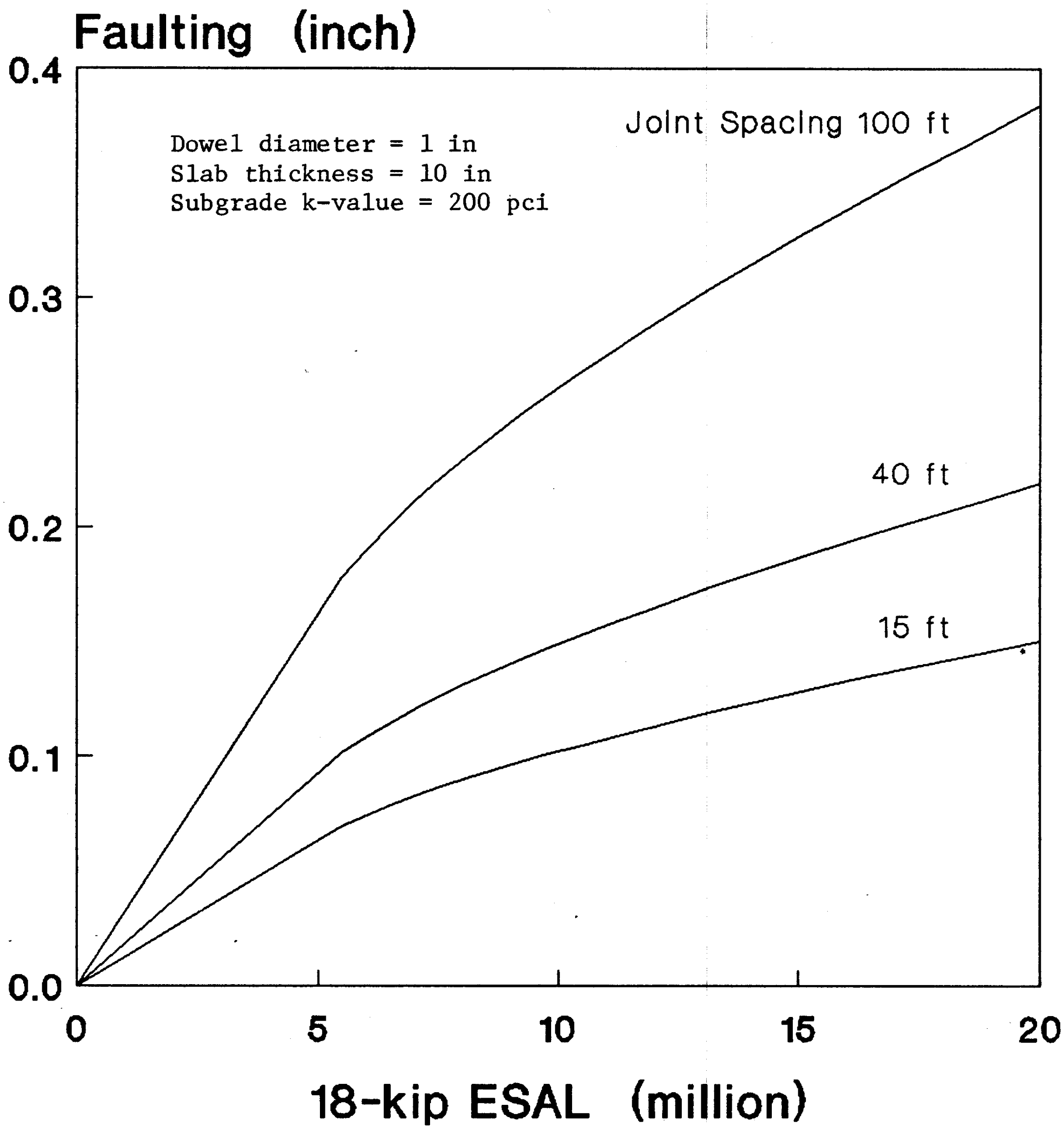


Figure 50. Sensitivity of faulting vs. cumulative 18-kip ESAL with varying joint spacing (PFAULT version 1.0).

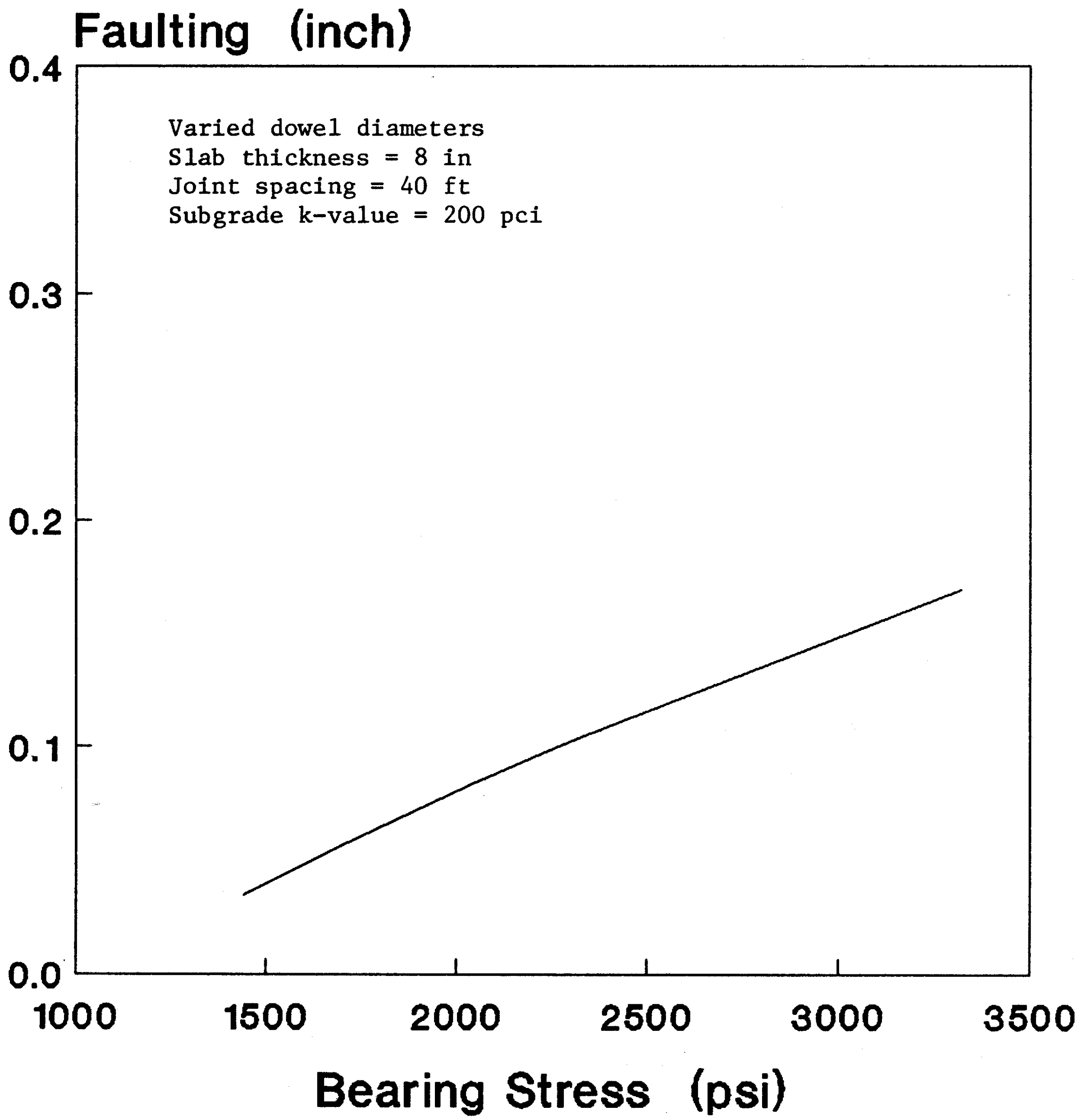


Figure 51. Faulting vs. bearing stress (40 ft joint spacing) (PFAULT version 1.0).

$$DEFL = \frac{P}{k\ell^2} [1.1 - 0.88 (\sqrt{2} \frac{a}{\ell})] \quad (41)$$

where:

DEFL = corner deflection computed using the Westergaard equation;
P = applied wheel load;
a = radius of the applied load;
ℓ = radius of relative stiffness; and
k = modulus of subgrade reaction.

The applied wheel load was assumed to be 9,000 lbs (4086 kg), the tire pressure was 90 psi (6.3 kg/cm^2). No load transfer was assumed in this calculation.

A base/subbase material erodibility factor, ERODF, was introduced corresponding to the type of base based on AASHTO recommendations.[39] The granular material is the highest potentially erodible base (ERODF=2) while the lean concrete is considered the least erodible (ERODF=0.25). The asphalt-treated and cement-treated base have the same erodibility factor as 1.0. Unfortunately, no permeable base courses were available for study, and they are expected to have a major effect on reducing faulting.

The nonlinear regression technique used for the doweled pavements was used again to develop a faulting predictive model for undoweled pavements. Only plain concrete pavement sections were available in the NCHRP 1-19 database for undoweled pavements. The database included pavement sections from the following States: Georgia, Illinois, Louisiana, Utah, and California. Two extra sections from New Jersey and Michigan were also added. The database was increased with 24 additional sections from California, including the consideration of half-joint spacing (7.8 ft) (2.38 m), thicker slab (11.4 in) (3.47 m) and lean concrete base (effective k-value = 591 pci) conditions. The final nationwide faulting predictive model generated for undoweled pavements is given as follows:

$$\begin{aligned} FAULT = ESAL^{0.1927} [&0.5294 + 0.3388\ OPENING^{0.3773} \\ &- 0.6085\ (100*DEFL)^{-0.01560} \\ &+ 0.00881\ FI^{0.05829} + 0.0119\ ERODF \\ &- 0.03718\ EDGESUP - 0.001562\ SOILCRS \\ &- 0.004293\ DRAIN] \end{aligned} \quad (42)$$

(Note: See appendix E for an updated, improved version of the equation based on the inclusion of additional field data in the database).

where:

ESAL = total 18-kip ESALs in design life, millions;
OPENING = mean transverse joint opening, in;
DEFL = Westergaard corner deflection, in;
FI = mean air freezing index, degree-days;
ERODF = erodibility factor for base/subbase materials,
= 0.25, if lean concrete base is used,
= 1, if asphalt- or cement-treated base is used,
= 2, if granular base is used;
EDGESUP = 0, if no edge support exists,
= 1, if edge beam/tied PCC shoulder exists;
SOILCRS : AASHTO subgrade soil classification,
= 0, if A-4 to A-7,
= 1, if A-1 to A-3; and
DRAIN = 0, if no edge subdrains exist,
= 1, if edge subdrains exist.

Statistics: $R^2 = 0.526$
SEE = 0.019 in
n = 175

Figure 52 shows a plot of actual versus predicted faulting for the final nationwide predictive model (equation 6.13) for undoweled pavements. This model takes into consideration some climatic variables (e.g., temperature range which determines joint opening, and freezing index) and several mechanistic variables, such as transverse joint opening and slab corner deflection. It could be expected to predict the mean joint faulting value with reasonable accuracy. However, this model has some major deficiencies as follows:

- The database was still limited to only a few States and climatic zones.
- A variety of other situations existed, in which there was not sufficient range of some of the variables (e.g. permeable base, joint sealant condition, etc.).
- Due to time constraints, the model developed remained simple. There is a need to improve the model by considering the combined effects of joint openings, degree of load transfer for joint, corner deflections and the presence of subdrainage.

It is important to note that the user should use this model to predict the faulting with care, avoiding unwarranted extrapolation beyond the data range from which it was generated. However, this mechanistic-empirical model is believed to be reasonably accurate and can be employed in long-term performance evaluations and design applications within the range of data.

A sensitivity analysis of the input variables was performed to detect the significance of each design variable in the model. All independent variables are correlated logically with faulting as shown below:

Variable Change	Effect on Faulting
Increase traffic (ESAL)	Increase
Increase temperature range (TRANGE)	Increase
Increase joint spacing	Increase
Increase joint opening	Increase
Increase corner deflection	Increase
Increase slab thickness	Decrease
Increase k-value	Decrease
Increase Freezing Index	Increase
Use of lean concrete subbase	Decrease
Use of tied PCC shoulder	Decrease
Use of coarse-grained subgrade	Decrease
Use of subdrainage	Decrease

Table 67 shows an example of the effect of these parameters in the model. A set of standard conditions were used in obtaining these plots, and one variable was varied at a time. Transverse joint spacing, base type and edge support have a more significant effect on joint faulting than any of the parameters in the final model.

For situations where dowels are not considered feasible, a sensitivity analysis of the model suggests the use of the following to minimize joint faulting:

- Shorter transverse joint spacing (e.g., less than 15 ft (4.6 m)).
- Thicker slabs.
- Stiffer base/subbases (higher effective k-value).
- Less erodible base/subbases (e.g., lean concrete base).
- Edge support (e.g,. tied concrete shoulders).
- Coarse-grained subgrade soils (drainage).
- Subdrainage pipes.

Plots illustrating the sensitivity of the variables in this model are shown in figures 53 through 57. An interactive computer program named PFAULT was written for both faulting predictive models and is available in an IBM-PC compatible version with documentations. The program input guide is included in Appendix E.

6.3.3 Guidelines For Use Of Dowels

The predictive models for faulting can be used to assist in determining the need for dowel bars and also their required diameter for pavements that have the typical granular, cement-stabilized, asphalt-stabilized or lean conrete base courses. Highly permeable bases cannot be considered by the predictive models.

Dowel bars, properly sized and spaced, will reduce faulting of transverse joints. However, many pavements that contain dowels with inadequate diameters have faulted badly, due to over stressing from repeated heavy truck axle loads. The predictive models can assist in the selection of the proper diameter of dowel bar.

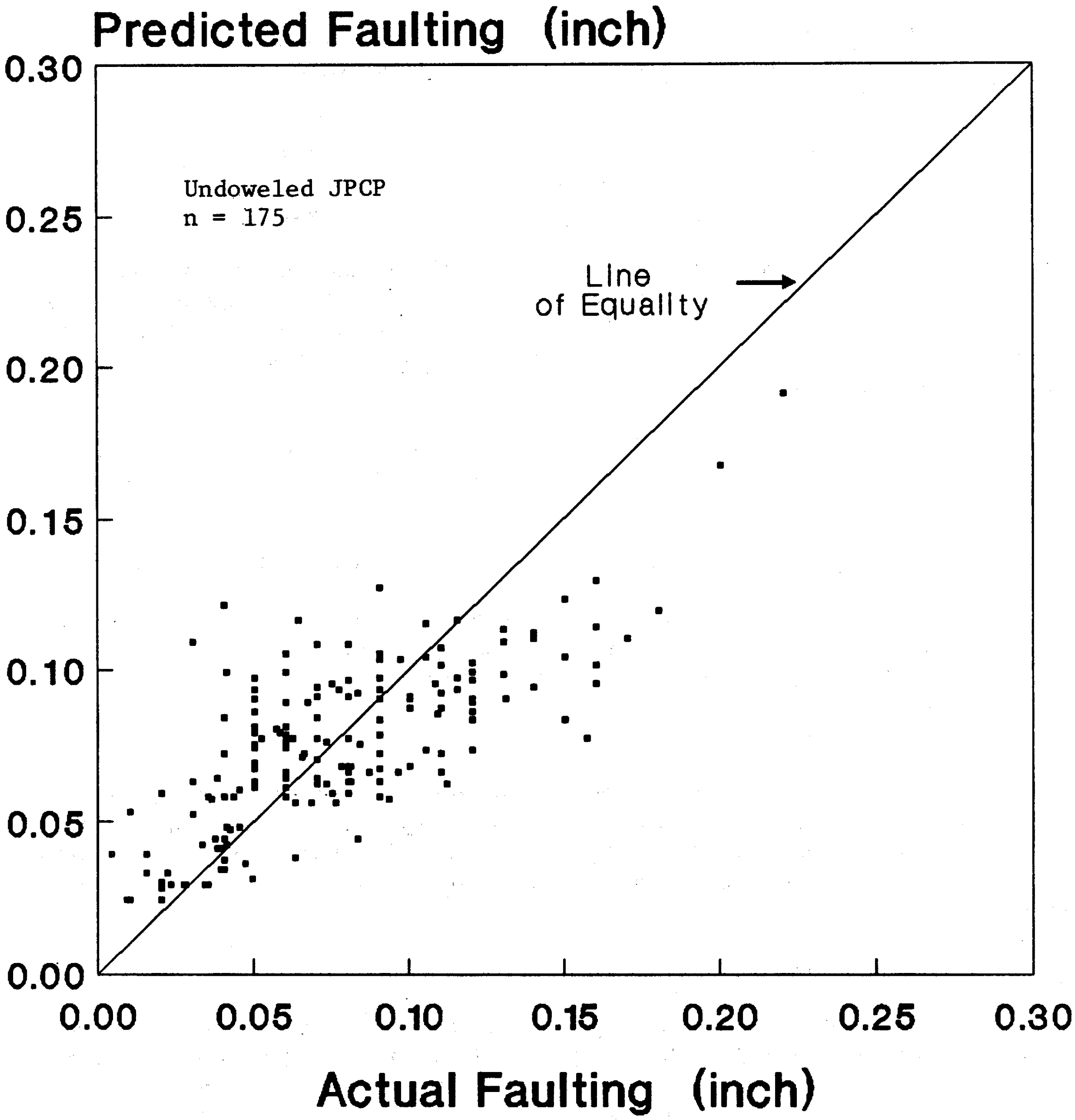


Figure 52. Predicted vs. actual faulting for nationwide model (version 1.0) for undoweled pavements.

Table 67.

Example sensitivity analysis of nationwide faulting predictive model (version 1.0) for undoweled pavements.

Design Parameter	Change in Parameter	Change in Faulting (in)
Traffic (18-kip ESAL), million	10 to 20	0.128 to 0.146
Slab Thickness, in	10 to 12	0.128 to 0.124
Freezing Index, degree-days	100 to 0	0.128 to 0.110
Transverse Joint Spacing, ft	15 to 20	0.128 to 0.147
Base Type	Granular to Lean Concrete	0.128 to 0.074
Shoulder Type (edge support)	AC to Tied PCC	0.128 to 0.070
Subdrains	No to Yes	0.128 to 0.121

Note: 1. Corner deflection is a function of slab thickness. Joint opening is a function of joint spacing and base type. Erodability factor is a function of base type.

2. A set of standard condition was as follows:
   Traffic (18-kip ESAL) = 10 million
   Transverse joint spacing = 15 ft
   Effective subgrade k-value = 200 pci
   Slab thickness = 10 in
   Temperature range = 30 °C
   Freezing Index = 100 degree-days

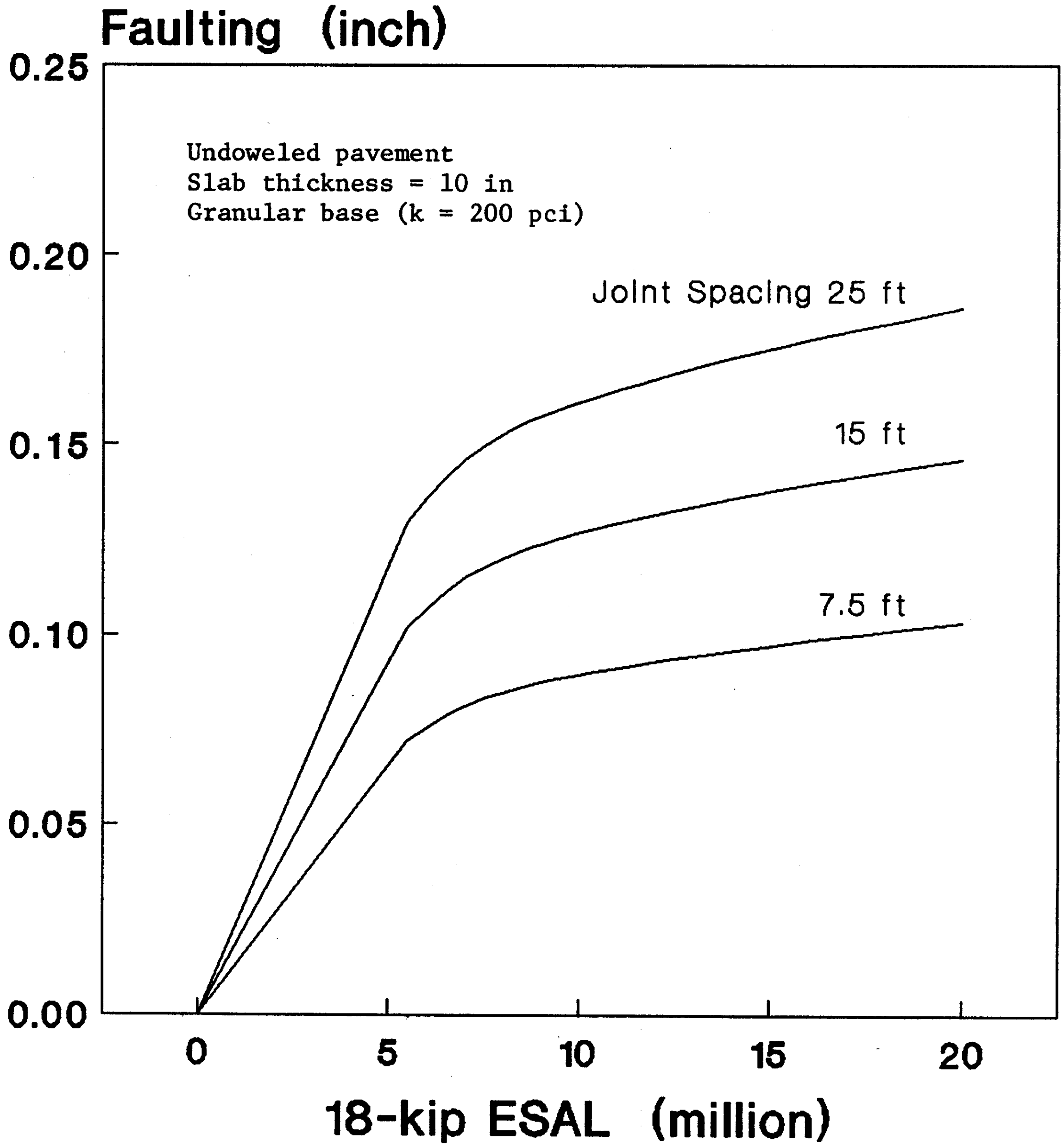


Figure 53. Faulting vs. cumulative 18-kip ESAL with varying joint spacing (PFAULT version 1.0).

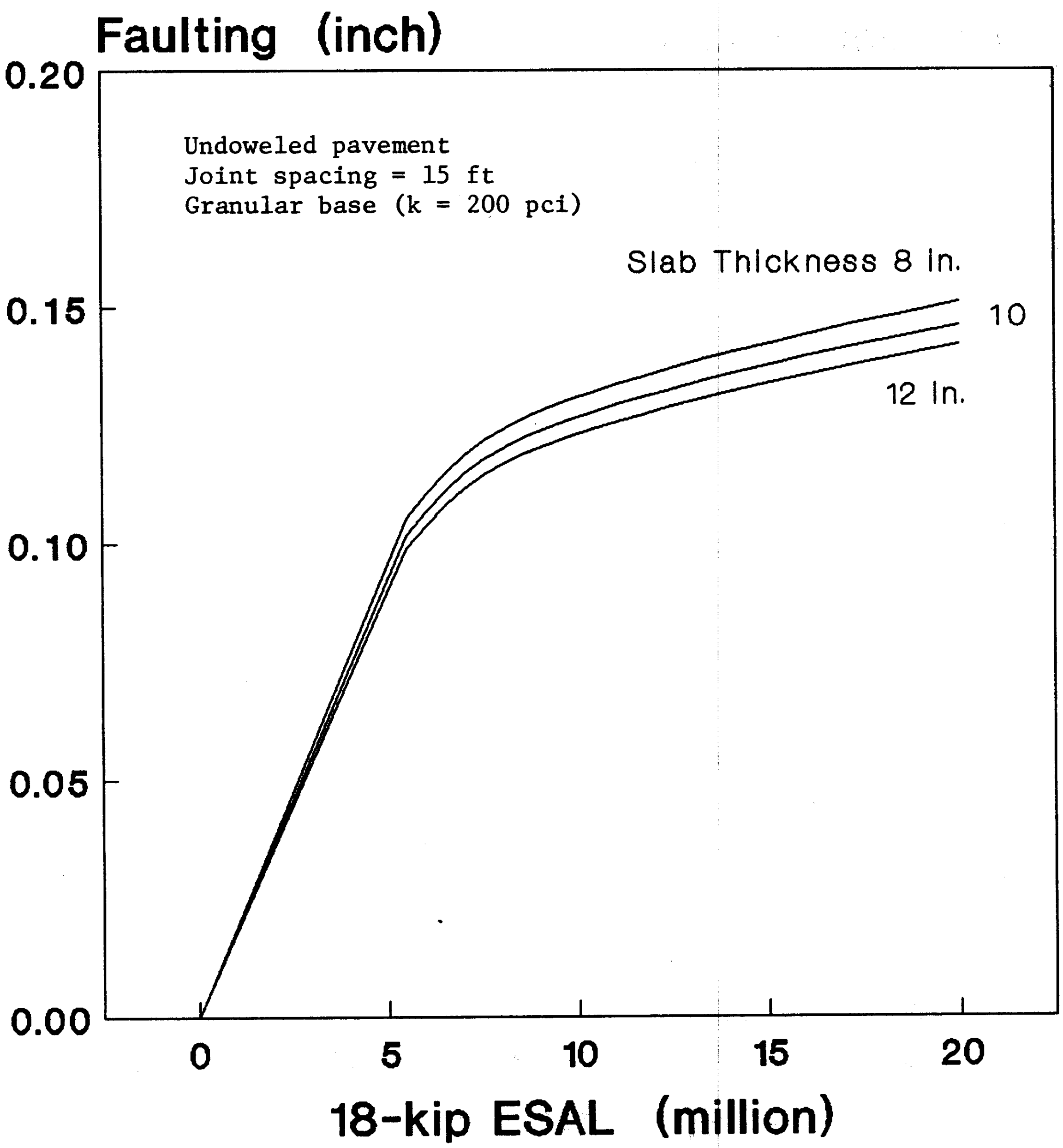


Figure 54. Faulting vs. cumulative 18-kip ESAL with varying slab thickness (PFAULT version 1.0).

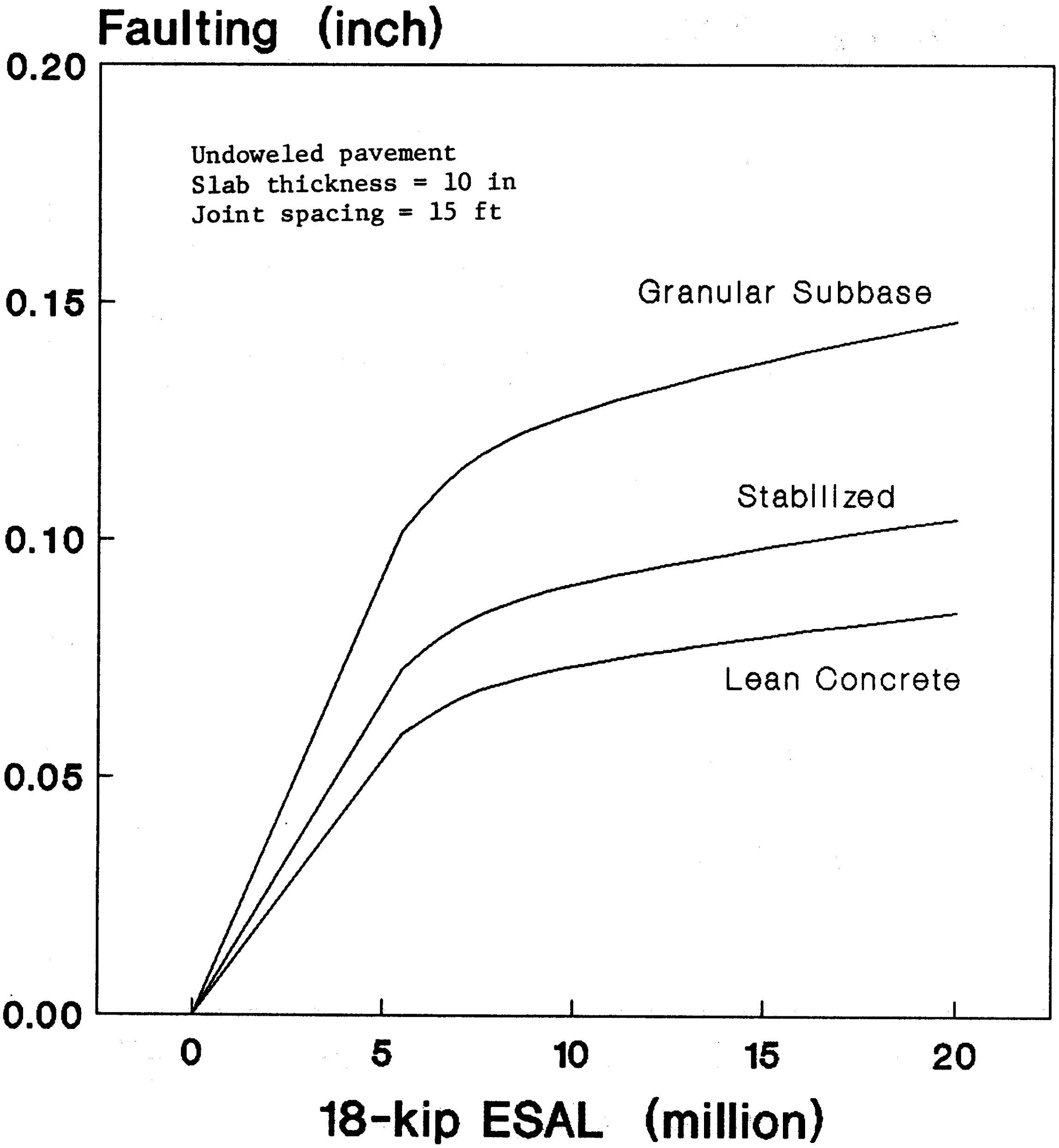


Figure 55. Faulting vs. cumulative 18-kip ESAL with varying base type (PFAULT version 1.0).

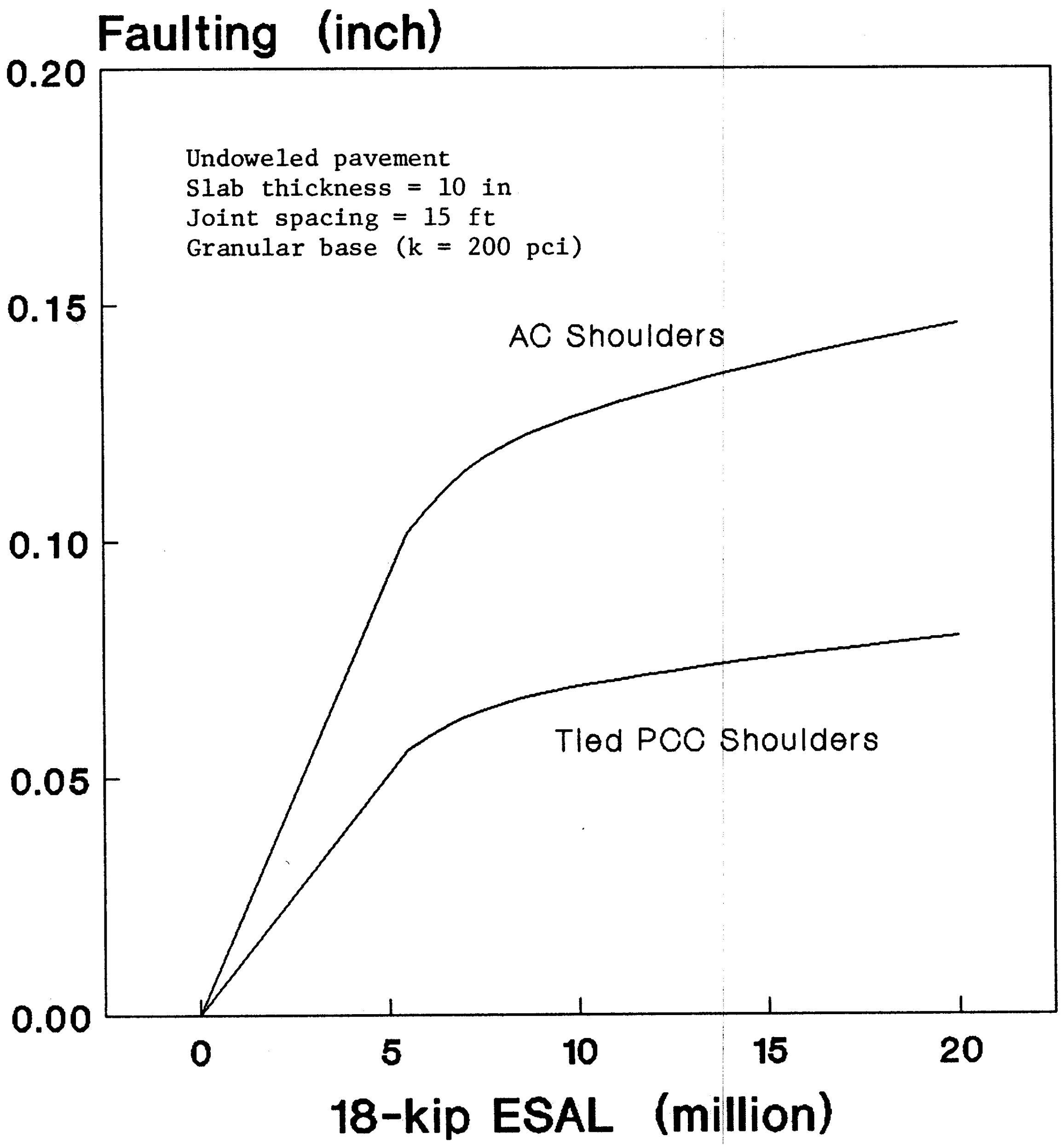


Figure 56. Faulting vs. cumulative 18-kip ESAL with varying shoulder type (PFAULT version 1.0).

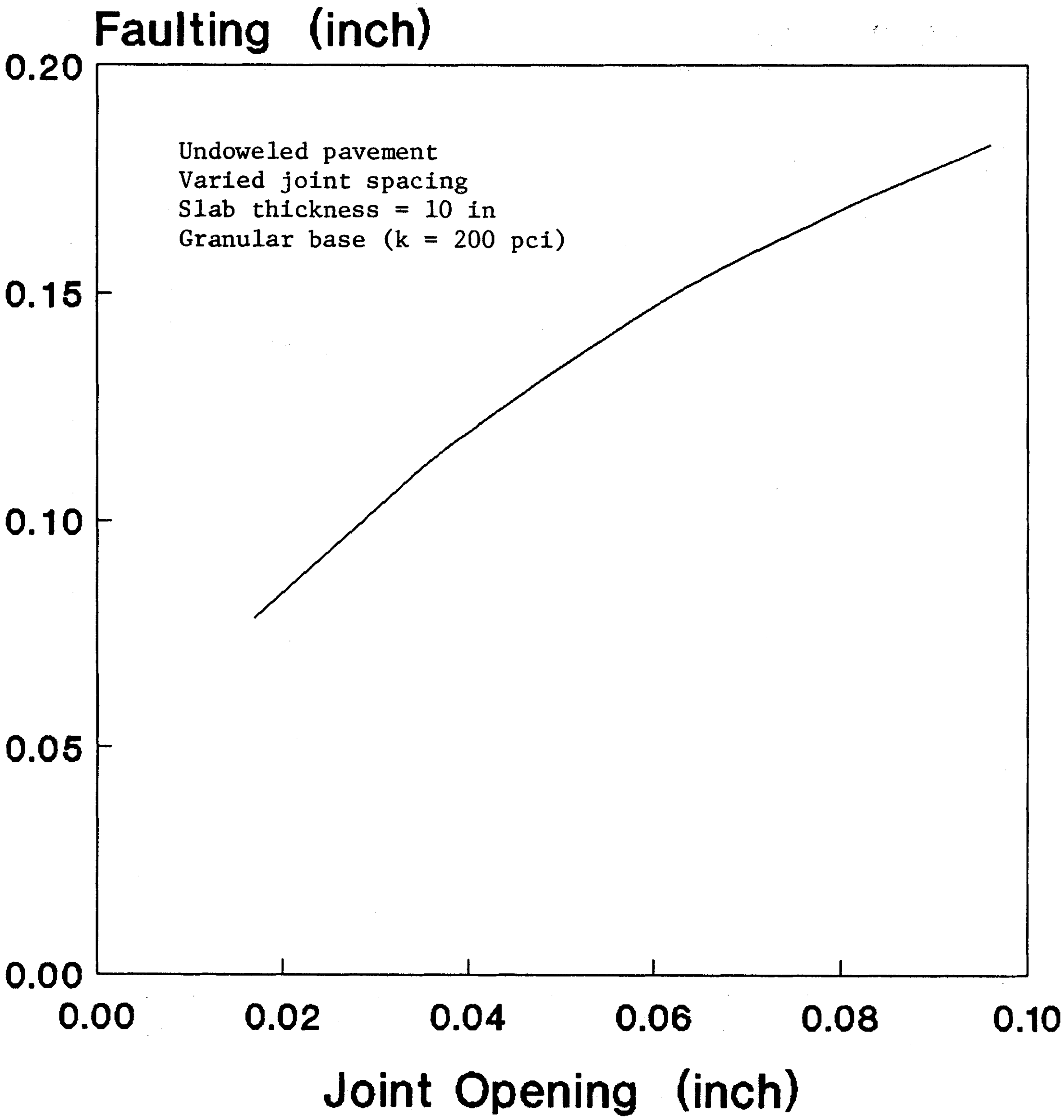


Figure 57. Faulting vs. joint opening (PFAULT version 1.0).

1. **Jointed reinforced concrete pavements:** Dowels are always recommended for JRCP due to the longer joint spacings and subsequent openings. The greater the faulting, the rougher the ride, and the lower the present serviceability index. The required dowel diameter to limit dowel faulting to an acceptable level is determined by using the PFAULT program (or solving equation 40 by hand) for the specific design inputs. A critical level must be selected for use in design. The NCHRP 1-19 database was searched and the joint faulting for those pavements having a rough ride (serviceability index less than 3.0) was computed to be 0.26 in. A value much less than this must be used for design so that there is a high probability for success in limiting faulting. A value for design of approximately one-half of this value appears to give reasonable results, 0.13 in. This value is only approximate and needs alot of further verification.

   Example: A JRCP is being designed having the design inputs shown in table 68. The predicted mean faulting for different dowel diameters are plotted in figure 58. The dowel diameter required to limit faulting to 0.13 in (0.33 cm) is between 1.00 and 1.25 inches (2.5 and 3.2 cm) for 10 million 18-kip ESAL (use 1.25 in (3.2 cm)). If the pavement was loaded with 40 million ESAL, the required dowel diameter is between 1.25 and 1.50 in (3.2 and 3.8 cm) (use 1.375 or 1.50 in (3.5 or 3.8 cm)).

2. **Jointed plain concrete pavements:** Dowels are often not used in JPCP. Aggregate interlock is relied upon to provide load transfer. This is often not adequate to prevent faulting, and many of these pavement develop serious faulting from pumping. The need for dowels can be determined by using the PFAULT program to predict faulting for an undoweled pavement for the design 18-kip ESAL. JPCP in the NCHRP Project 1-19 database having a rough ride (present serviceability index of 3.0 or less) had an average transverse joint faulting of 0.13 in (0.33 cm). For design purposes, however, a lower value should be selected to provide a safety factor in the joint design. Again, a value of about one-half this mean, or 0.07 in (0.18 cm) appears to be reasonable, but this is subject to further verification.

   If the pavement requires dowels, the required dowel diameter can be determined similar to JRCP.

   Example: A JPCP is being designed having the design inputs shown in table 69 (a nonstabilized dense graded base course). The predicted mean faulting without dowel bars for the design 30 million 18-kip ESAL is 0.16 in (0.41 cm) as shown in figure 59, well above the design value of 0.07 in (0.17 cm). A different base course could be tried, such as a lean concrete base. This base results in a mean faulting of 0.11 in (0.28 cm). Thus, the lean concrete base is not adequate and dowels are needed.

   The appropriate dowel diameter must be selected. The predicted mean faulting for different dowel diameters is plotted in figure 60 versus dowel diameter. The dowel diameter required to limit faulting to 0.07 in (0.41 cm) is 1.25 in (3.2 cm) for 30 million 18-kip ESAL if a granular base is used.

Table 68. Predicted faulting for JRCP example problem.

```
                        P F A U L T (Version 1.0)

                TRANSVERSE JOINT FAULTING ANALYSIS FOR
         DOWELED JOINTED REINFORCED OR PLAIN CONCRETE PAVEMENT


           EXAMPLE JRCP DOWEL DESIGN PROBLEM

          *18 kip ESAL During Design Life      -    10.00 millions
          *Slab Thickness                      -    10.00 in
          *Joint Spacing                       -    40.00 ft
          *Effective k-value                   -   200.00 pci
          *Base Type                           -      .00
            0.) granular        1.) asphalt treated
            2.) cement treated  3.) lean concrete
          *Average Annual Temperature Range    -    40.00 °C
          *Coefficient of Expansion (Concrete)-  .10E-04 / °C
           Mean Joint Opening                  -    .1344 in
          *Modulus of Elasticity of PCC        -  .40E+07 psi
          *Poisson's Ratio of PCC              -      .15
           Wheel Load                          -     9000 lbs
           Percentage of Load Transferred      -       45 %
          *Modulus of Elasticity of Dowel      -  .29E+08 psi
          *Modulus of Dowel Support            -  .15E+07 pci
           Dowel Spacing                       -    12.00 in
           # of Effective Dowels at Wheel A    -   2.0075


 DOWEL DIAMETER    - 1.00 in.

            Relative Stiffness of Encased Dowel-    .71642
            Dowel Moment of Inertia            -    .04909
            Bearing Stress                     -    3029.8 psi

            PREDICTED MEAN FAULTING            -     .1518 in


 DOWEL DIAMETER    - 1.25 in.

            Relative Stiffness of Encased Dowel-    .60601
            Dowel Moment of Inertia            -    .11984
            Bearing Stress                     -    2035.8 psi

            PREDICTED MEAN FAULTING            -     .0843 in


 DOWEL DIAMETER    - 1.50 in.

            Relative Stiffness of Encased Dowel-    .52856
            Dowel Moment of Inertia            -    .24851
            Bearing Stress                     -    1472.3 psi

            PREDICTED MEAN FAULTING            -     .0377 in
```

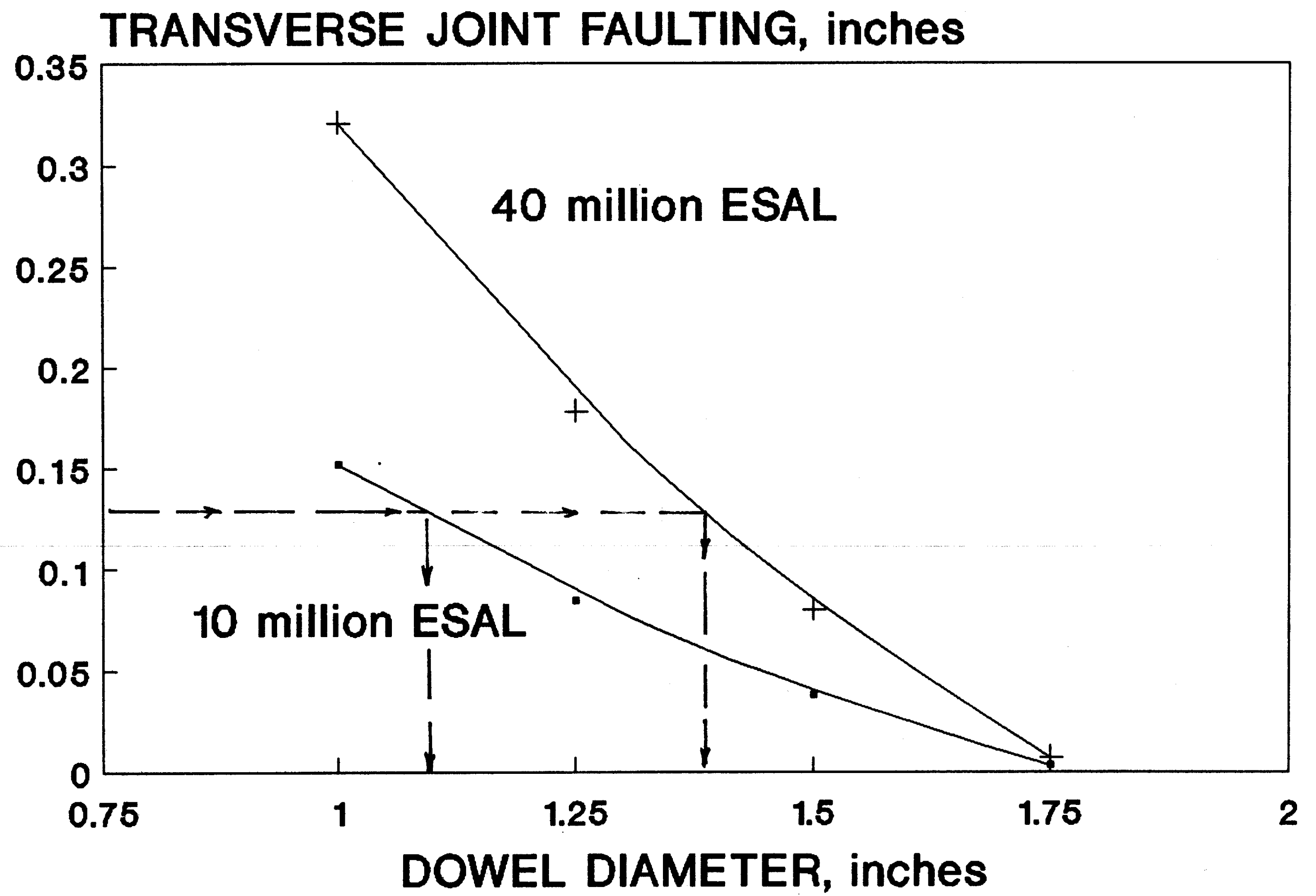


Figure 58. Predicted faulting vs. dowel diameter for JRCP example problem.

Table 69. Predicted faulting for JPCP example problem.

```
                    P F A U L T (Version 1.0)

          TRANSVERSE JOINT FAULTING ANALYSIS FOR
        UNDOWELED JOINTED PLAIN CONCRETE PAVEMENT


 EXAMPLE JRCP DOWEL DESIGN PROBLEM

*18 kip ESAL During Design Life        =    30.00 millions
*Slab Thickness                        =    10.00 inches
*Joint Spacing                         =    15.00 ft
*Effective k-value                     =   200.00 pci
*Base Type                             =      .00
  0.) granular        1.) asphalt treated
  2.) cement treated  3.) lean concrete
 Erodability Factor                    =     2.00
*Average Annual Temperature Range      =    40.00 °C
*Coefficient of Expansion (Concrete)=  .10E-04 /°C
 Mean Joint Opening                    =    .0504 in
*Modulus of Elasticity of PCC          =  .40E+07 psi
*Poisson's Ratio of PCC                =      .15
 Wheel Load                            =     9000 lb
 Percentage of Load Transferred        =        0 %
 Corner Deflection                     =    .0312 in
*Freezing Index (degree-days)          =   250.00
*Subdrainage                           =      .00
      0.) No      1.) Yes
*Subgrade Soil Classification          =      .00
      0.) A4-A7   1.) A1-A3
*Shoulder Type                         =      .00
      0.) AC      1.) Tied PCC


 PREDICTED MEAN FAULTING               =    .1681 in
```

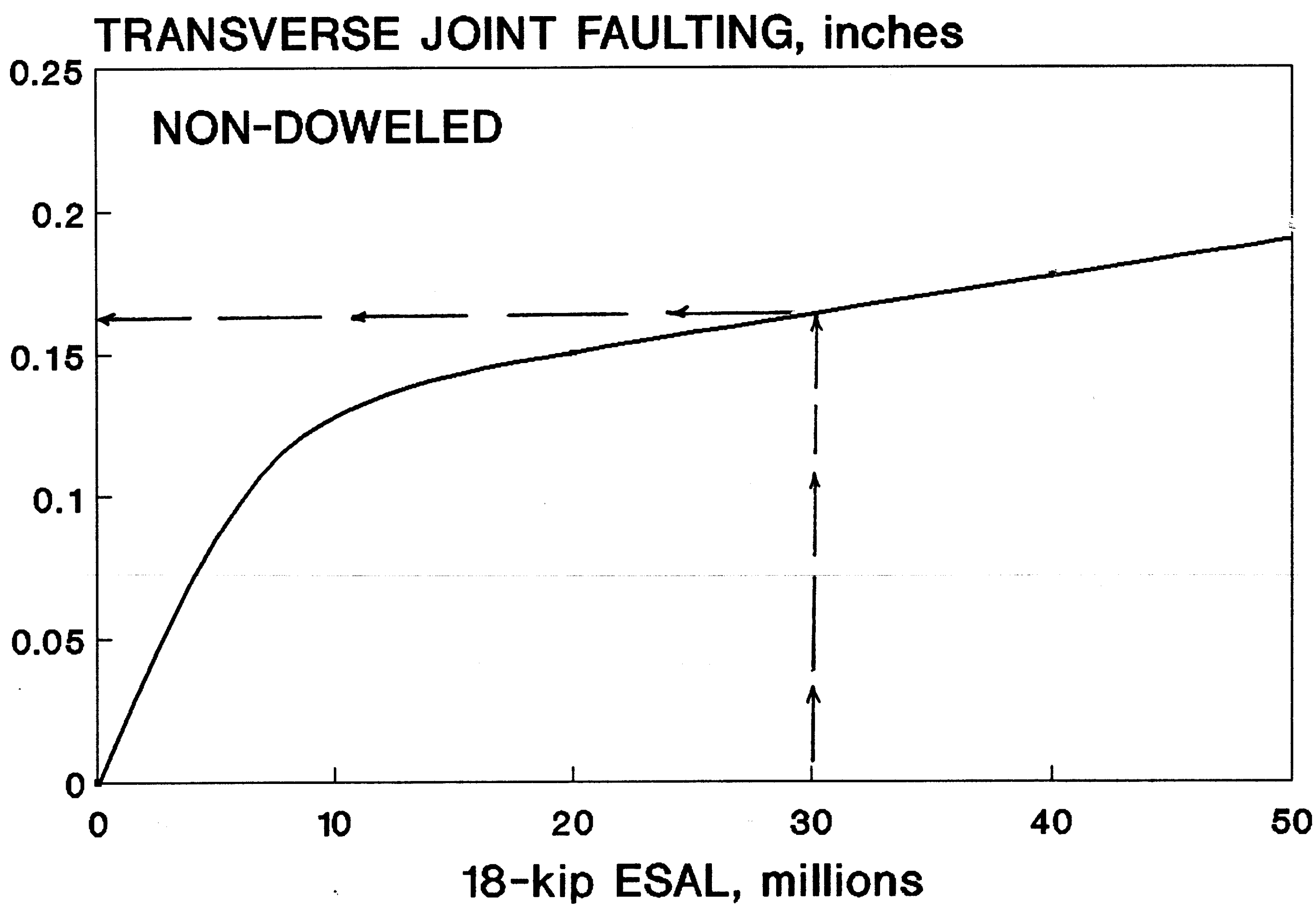


Figure 59. Predicted faulting vs. ESAL for JPCP example problem (nondoweled joint).

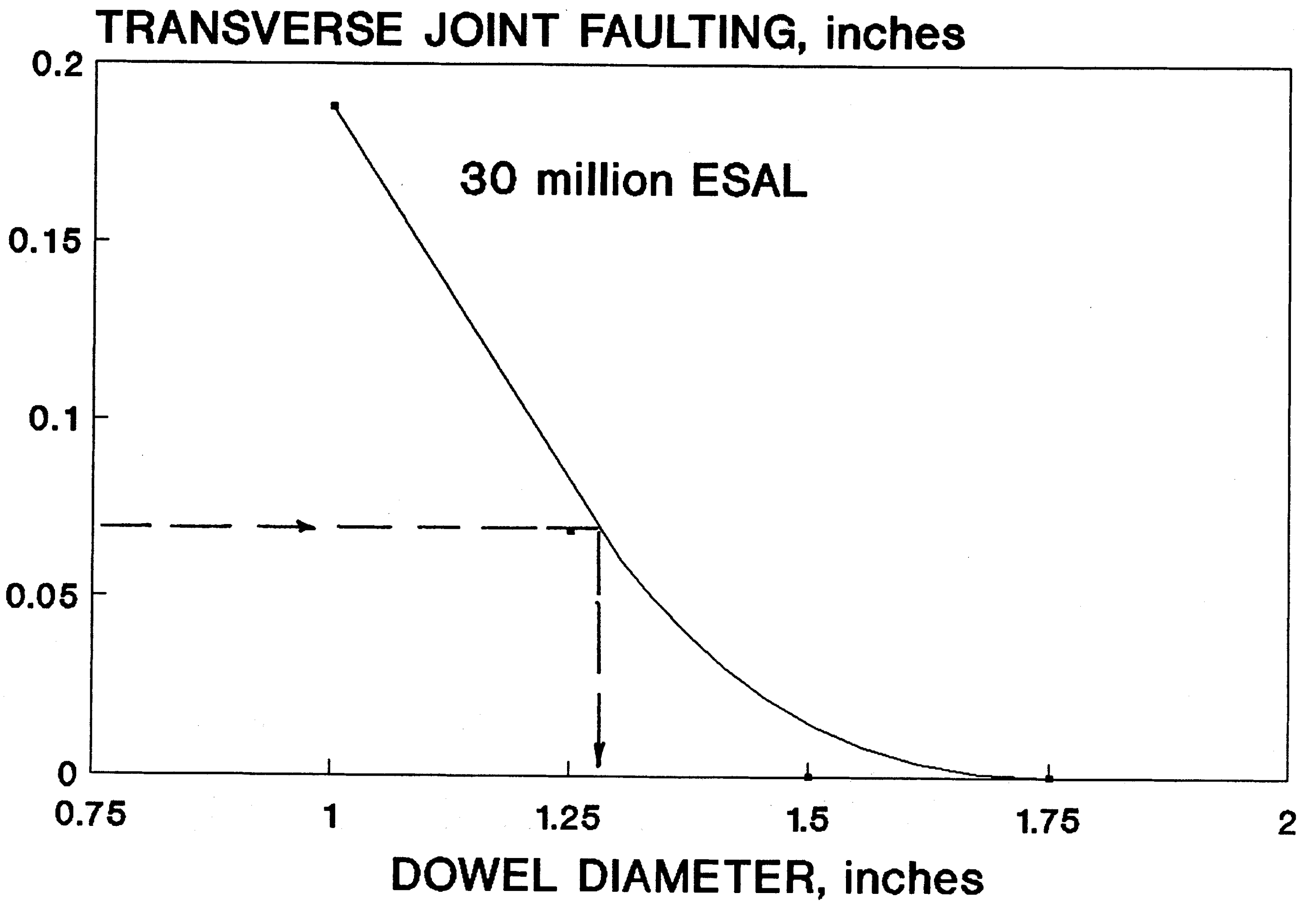


Figure 60. Predicted faulting vs. dowel diameter for JPCP example problem.

6.4 Dowel Misalignment

The design of doweled joints in concrete pavements is based on the assumption that measures will be taken during construction to prevent dowel looseness and misalignment. The aim is to reduce the probability of locked joints, which are usually manifest in transverse cracking, corner breaks and spalling at the dowel-concrete interface. Although analytical studies [92,93,116,117] indicate that stringent limits on dowel placement tolerances are in order (0.5 to 1.0 percent both vertically and horizontally), evidence from laboratory and field investigations suggests that these are quite costly to achieve and may not always be warranted.[95] As a result, no universal consensus exists as to the level of acceptable and practical misalignment.

The FHWA used to specify limits on dowel placement, but this practice was discontinued with the publication of the current FHWA Technical Advisory (No. T 140.18).[118] The recommendation is now to ensure "close tolerances for dowel placement (which) are extremely important for proper functioning of the slab and for long-term performance... Care must be exercised in both specifying dowel placement tolerance and in evaluating the adequacy of construction placement".[119]

The prevailing current practice has been to allow for 0.25 in (0.63 cm) misalignment per 18 in (45.7 cm) of dowel bar length (i.e., 1.4 percent). This can be traced back to research conducted in the mid-1960's.[94] More recently, Parmenter pointed out that "as a result of further laboratory studies and of theoretical comparisons of the effect of misalignment with that of traffic loading, a relaxation of the tolerance on alignment to the range of 3 to 4 percent was suggested" by the British Cement and Concrete Association.[95] Current State practices in the U.S. range from 1 percent (e.g., Illinois, Michigan, Minnesota, New York) to 3 percent (Georgia) or even 4 percent (Tennessee).[120]

Two recent FHWA studies present comprehensive state-of-the-art reviews on the effects of dowel misalignment. The limited amount of laboratory test data and the lack of sufficient data on field performance of jointed concrete pavements with misaligned dowels are pointed out in both of these.[97,88] A summary of previous theoretical investigations and of a brief effort to employ three-dimensional finite element modeling techniques is also presented.[97] These activities need to be intensified in future studies.

The present study makes no recommendations for revisions of current dowel misalignment practices, beyond reinterating the following:[97,88]

1. Dowel misalignment can be severely detrimental to the serviceability and long-term performance of jointed concrete pavements, as evidenced by the relatively few theoretical and experimental studies conducted to date.

2. The need to ensure that dowels are placed with the required precision calls for a review of current construction techniques. The West German practice, for example, of vibrating dowels into the concrete before setting offers great potential in this direction.

6.5 Joint Spacing

Selection of joint spacing is motivated by two conflicting requirements:

1. The need to arrest or control transverse and longitudinal cracking, resulting from the combined effect of temperature or moisture variations and applied traffic loading; and

2. The desire to provide adequate structural capacity that will ensure high functional performance and riding quality throughout the pavement life at a minimum cost.

To address both these issues effectively, it is important to have a thorough understanding of such phenomena as the hydration of concrete during first setting (and the associated contraction undergone by the pavement), temperature curling and moisture warping. The mechanisms by which joints inhibit the detrimental effects of these processes must be identified and exploited. In addition, the introduction of zones of weakness which inevitably occurs at any slab discontinuity must be considered carefully, so that all possible alleviating measures may be taken (e.g., by the introduction of load transfer devices). Accomplishing the objectives of a successful joint design involves careful balancing of priorities, and can be assisted significantly by a sound knowledge of the pertinent theoretical studies, laboratory and field test results, as well as the individual designer's own experience and observations.

6.5.1 Transverse Joints

The main function of transverse joints is to arrest or control the formation of cracks, resulting from contraction experienced by the concrete during its initial setting, as well as from subsequent curling and warping. Thus, optimum joint spacing is a function of the structural characteristics of the pavement system, notably slab thickness and reinforcement as well as subbase type. Spacings encountered in current practice vary as widely as such properties. Clearly stated rules to be followed precisely can, therefore, never replace local experience and observations. Table 70 summarizes the practices of the 50 States with respect to transverse contraction joint spacing, as reported in a 1987 survey by the FHWA for AASHTO.[120]

Consideration of the cracking patterns developing in nonjointed plain PCC pavements may provide some guidelines for contraction transverse joint spacing design. Cracks due to restrained concrete contraction during initial setting generally form at 40-to 150-ft (12.2 to 45.7 m) intervals depending on the pavement system characteristics and its geographic location. Cracks resulting from thermal warping and loading due to the slab's self-weight or subsequently applied traffic, tend to form at 15 to 30-ft (4.6 to 9.1 m) intervals. Ordinarily, therefore, joint spacing for plain PCC pavements ranges between 12 and 20 ft (3.7 and 6.1 m). Considerably longer spacing (27 to 60 ft (8.2 to 18.3 m)) has been used for reinforced PCC pavements. For plain jointed PCC pavements, the PCA recommends a maximum spacing of 20 ft (6.1 m) and spacing of up to 40 ft for reinforced jointed PCC pavements.[122]

Table 70.
Summary of State practices:
Spacing of transverse contraction joints.

STATE	Spacing, ft. Plain	Plain Doweled	Reinf. Doweled
ALABAMA		20	39
ARIZONA	15-13-15-17	15-13-15-17	
ARKANSAS		15	45
CALIFORNIA	12-15-13-14		
COLORADO	12-15-13-14		
CONNECTICUT			40
DELAWARE			40
DIST. OF COLUMBIA			
FLORIDA		<20	
GEORGIA		20	
HAWAII	13-12-18-19		
IDAHO	13-18-17-12	13-15-16-14	
ILLINOIS			40
INDIANA	12-13-19-18	12-13-19-18	40
IOWA	15	20	
KANSAS	15	15	30
KENTUCKY		12-13-17-18	
LOUISIANA		20	58.5
MARYLAND			40
MICHIGAN			41
MINNESOTA	13-16-14-17	13-16-14-17	27
MISSISSIPPI		20	21.25
MISSOURI		30	42.5
MONTANA	12-15-14-13		
NEBRASKA	16.5		
NEW MEXICO		14-14-12-15	
NEW YORK		20	63
NORTH DAKOTA	13		
OHIO	20	17	40
OKLAHOMA	15		
OREGON		14	
PENNSYLVANIA		20	40
PUERTO RICO	20		
SOUTH CAROLINA		20	
SOUTH DAKOTA	15		
TENNESSEE		19-25-24-18	
TEXAS	15	15	60
UTAH	11-10-14-15		
VIRGINIA	20		40
WASHINGTON	9-10-14-13		
WISCONSIN	12-13-19-18	12-13-19-18	40
WYOMING	12-13-16-14		

A rational procedure for determining the joint spacing appropriate at each site is yet to be developed. A report on an experimental JPCP project in Kentucky recommended a 30-ft (9.1 m) maximum joint spacing.[37] Reflecting Minnesota's experience, another called for a maximum of 15 to 20 ft (4.6 to 6.1 m), Michigan indicated that longer spacings may not be detrimental.[124,125] A possible explanation is suggested for these somewhat contradictory observations by distinguishing between short-term and long-term horizontal slab movements.[126] Short-term movements constitute "an immediate response of the pavement to climatic changes" and involve two or even three slabs moving as unit. Though occasionally large in magnitude, short-term movements are independent of joint spacing. Long-term movements, on the other hand, are much more regular, occurring in near-equal magnitudes at each joint. Thus, they are directly related to joint spacing.

Optimum joint spacing for JPCP is considerably influenced by the structural characteristics of the pavement system, such as slab thickness, concrete modulus of rupture and support stiffness.[127] Figure 61 illustrates the general trends. Thus, a longer joint spacing may be tolerated if slab thickness increases. A shorter joint spacing is required for stiffer base layers (e.g., lean PCC) to reduce thermal curling stresses. This is consistent with theoretical investigations indicating that the lumped parameter (L/ℓ), in which L is the slab length and ℓ is the radius of relative stiffness of the slab-foundation system, is generally a better indicator of dimensional effects than the slab size, L, alone.[8]

Randomized JPCP joint spacing was introduced in the 1950's to discourage resonant responses that regular joint spacing may induce in some vehicles operated at or near the legal highway speed limit. Joint spacings generally range between 12 and 25 ft (3.0 and 7.6 m), a typical pattern used being 13-19-18-12 feet (4.0-5.8-5.5-3.6 m). However, this spacing has been reduced recently to prevent transverse cracking on the longer slabs (which has occurred regularly in JPCP in western States). California now employs 12-15-13-14 ft (3.6-4.6-4.0-4.3 m) intervals; Washington specifies 9-10-14-13 ft (2.7-3.0-4.3-4.0 m), while Minnesota opts for a 13-16-14-17 ft (4.0-4.9-4.3-5.2 m) pattern.[92] In all these, intervals in 7.5-ft (2.3 m) multiples are generally avoided. These recommendations appear reasonable.

Joint spacing for JRCP has a major effect on the number of deteriorated joints per mile after the pavement has been in service for a number of years.[42] The shorter spacings (27 ft (8.2 m)) have substantially less deterioration than the longer joint spacing (40 ft (12.2 m)), as shown in figure 62. Direct comparison in Minnesota shows a clear advantage of 27-ft (8.2 m)joint spacings. In addition, the longer the joint spacing, the more transverse cracks that have developed. These cracks have often spalled and faulted. Therefore, it is recommended that a maximum joint spacing of 25 to 30 ft (7.6 to 9.1 m) be used for JRCP.

Transverse joints may also be formed when construction has to be interrupted as, for example, at the end of the day. Efforts should be made to place such construction joints at or near the location of a planned transverse joint, otherwise an adequate load transfer mechanism must be installed to prevent spalling and faulting.

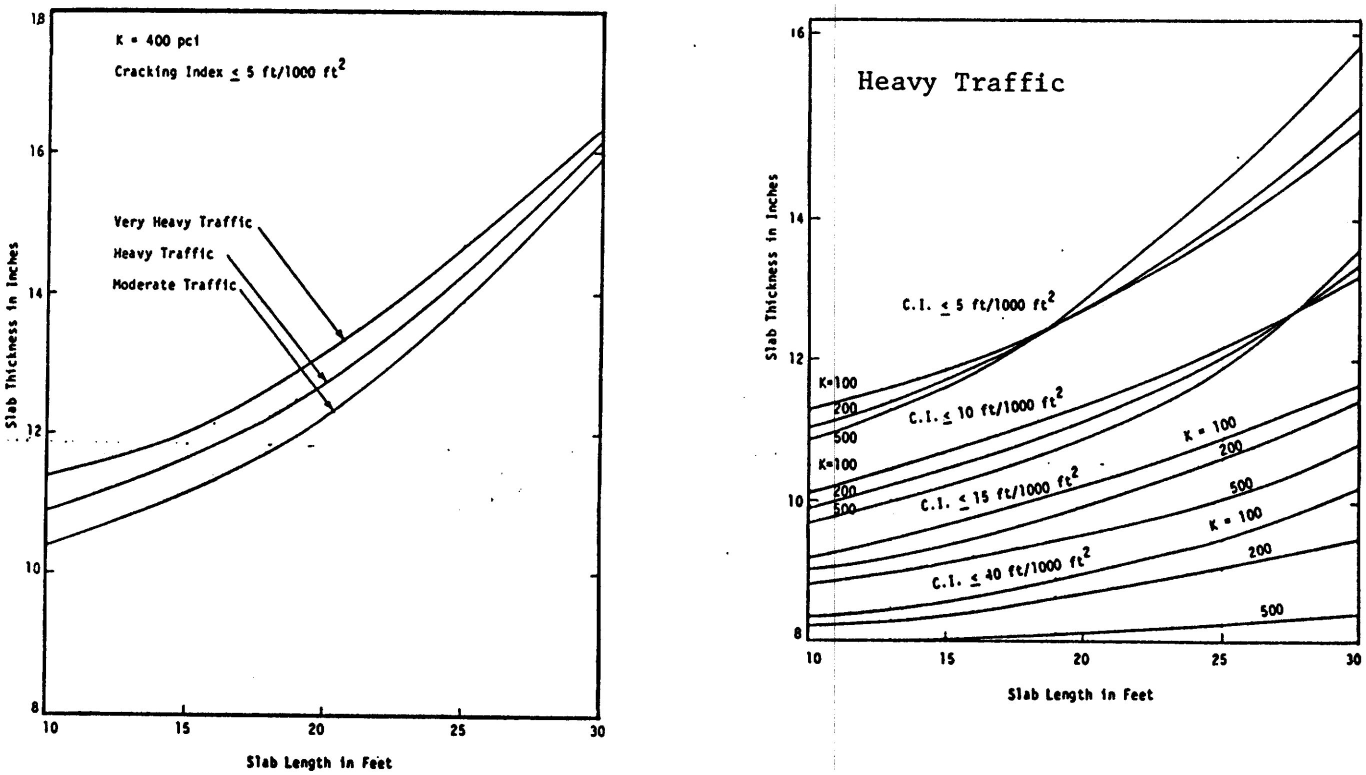


Figure 61. Effect of traffic level and foundation support on slab thickness and joint spacing.

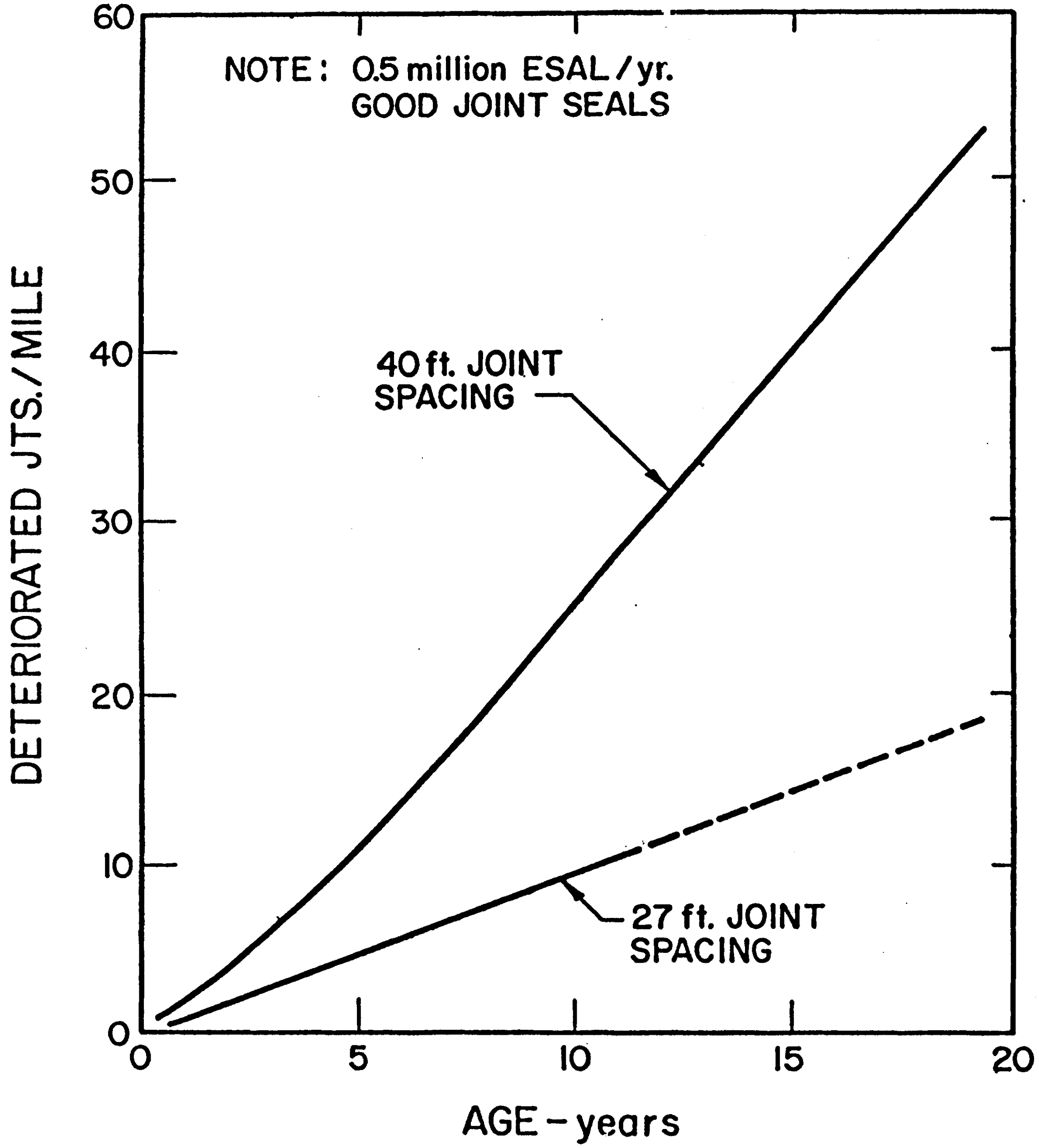


Figure 62. Sensitivity of the Minnesota joint deterioration model to joint spacing and age (cumulative load repetitions).

Fixed objects and unsymmetrical intersections may require additional transverse expansion joints, not usually necessary provided the general spacing recommendations for transverse joints are adhered to. Pertinent state practices are summarized in table 71.

6.5.2 Longitudinal Joints

The nature, causes and mode of formation of longitudinal cracks are similar to those of transverse cracks. Practical considerations beyond crack control dictating the use of longitudinal joints include lane delineation and the finite width of paving equipment. Where multiple lanes can be placed in one pass, longitudinal joints can be sawed or formed using an insert before the concrete sets (not recommended).

The following criteria are recommended by the PCA for the spacing of longitudinal joints:[122]

1. On both two-lane and multilane highway pavements, a spacing of 10 to 13 ft (3.0 to 4.0 m) serves the dual purpose of crack control and lane delineation. Longitudinal joints on arterial streets should also be spaced to provide traffic and parking lane delineation. On these streets, it is customary to allow 10 to 12 ft (3.0 to 3.6 m) for each travel lane and 10 to 12 ft (3.0 to 3.6 m) for parking, that can also be used as a travel or turning lane.

2. Longitudinal joints are usually required for crack control on one-way ramps where the width is 14 ft (4.3 m) or more.

The improper forming of longitudinal joints has led to serious longitudinal cracking. Much of this is attributed to plastic tape inserts, but another cause is late sawing of the joint. Most agencies recommend a one-third-depth cut. West Germany requires a 0.45 depth cut. This is particularly critical when a high friction base is used.

Table 71.
Summary of State practices:
Spacing of transverse expansion joints.

STATE	At Bridge End: Number Used	At Bridge End: Spacing, ft.	Other Locations: Where Used	Other Locations: Spacing, ft.
ALABAMA	1	n/a	At fixed objects	-
ARKANSAS	1; see (2)		At fixed objects;	
CALIFORNIA	(3)		Pressure relief jt.	As needed
COLORADO	1	n/a	At fixed objects	
CONNECTICUT			Ramps;by Engr. Req.	500-1000
DELAWARE	1			
FLORIDA	2	40	Intersections, etc.	Var.
GEORGIA	1	-		
IDAHO	-			
ILLINOIS	1			
INDIANA	1	20'6"		
IOWA	1	40-60		
KANSAS	1		Pressure relief	33
KENTUCKY	3	30-30-48		
LOUISIANA	2	20	Radii of turnouts	1 ea.
MARYLAND				600
MICHIGAN	1-4	41		
MINNESOTA	1		Dowel (14) baskets	as des.
MISSISSIPPI	2	20		
MISSOURI	2	30		
MONTANA	2	20		
NEBRASKA			Designated loc.	
NEVADA				
NEW JERSEY			PCC pavements	78'2"
NEW YORK	2	Var.	Match exist. jt.	Var.
NORTH DAKOTA	-			
OHIO	2	20		
OKLAHOMA	1	-	ramp terminals	-
OREGON	2	40	Terminal @ CR sect.	-
PENNSYLVANIA			when designed	-
SOUTH CAROLINA	3	50		
SOUTH DAKOTA	1	-		
TENNESSEE	2	25	1/2 mile intervals	2640
TEXAS	1	-		
UTAH	2	10.5		
VIRGINIA	1			
WASHINGTON	1			
WISCONSIN	2	20-40		
WYOMING	1			

7.0 CONCLUSIONS AND RECOMMENDATIONS

This final chapter provides conclusions and recommendations on the use of models and methods for rigid pavement design, presents the recommendations for experimental projects to test new rigid pavement designs and describes newly developed joint design procedures.

7.1 Models and Methods for Rigid Pavement Design

1. Basic structural model for computation of stresses and deformations for use in rigid pavement design: The ILLI-SLAB or JSLAB finite element models are efficient to use and can structurally model many key design factors of importance. The ILLI-SLAB program has had extensive checking, revisions and verification over the past 10 years by many researchers and is more free of errors than any other available program.

2. PCC slab design (length, width, thickness and strength): The best fatigue algorithm for computing slab thickness requirements is the Zero-Maintenance procedure. The procedure must be modified and updated to include improved thermal curling procedures, widen lanes/tied shoulders and unbonded lean concrete base layers. Several of the European countries have utilized the basic Zero-Maintenance approach but have improved on the procedures. A new fatigue cracking curve should be developed that is based on field slabs instead of laboratory beams to make the fatigue predictions more accurate.

3. Smoothness/performance: All of the design procedures based upon the AASHO Road Test directly consider the loss of serviceability in the design. However, as the results showed, there are many limitations for the use of the AASHO Road Test prediction model. It is recommended that the combined data from the AASHO Road Test and the NCHRP Project 1-19 COPES database be used to develop an improved serviceability-performance prediction model that directly considers climatic effects. It is believed that the consideration of both serviceability and fatigue damage is important in developing a successful rigid pavement design.

4. Reinforcement design: None of the procedures provide adequate design procedures for reinforcement for JRCP or CRCP that considers both repeated wheel loading (shear) and climatic effects. The CRCP-2 program and later versions provide the most comprehensive procedure available for analyzing crack spacing and crack width. The subgrade drag theory for JRCP design is very inadequate and an improved procedure must be developed, particularly that considers crack aggregate interlock capability and repeated shear loads from traffic.

5. Base/subbase design: One of the most deficient aspects of current design procedures is the lack of ability of the procedures to model the base as an elastic layer that may or may not be bonded to the PCC slab. The common practice of greatly increasing the k-value for stabilized bases has been shown to be inadequate in reducing actual stresses in the slab. The PCA procedure for erodability warrants consideration. The direct consideration of a drainable base and filter appears to have great potential. The stiffness of the base is very important under heavy traffic, which needs to be considered in CRCP and JRCP crack deterioratoin. The ILLI-SLAB finite element program can handle two pavement layers that are bonded or unbonded and having any layer stiffness values.

6. Subgrade: The ability to model the subgrade as an elastic solid is an important consideration. This would make it possible to design rigid pavements using a resilient modulus instead of a k-value. The ILLI-SLAB finite element program can handle an elastic solid subgrade, but cannot handle more than one pavement layer.

7. Shoulder/edge support/curb and gutter: This aspect of design is considered directly in the JCS-1, RISC and PCA program, and indirectly in the AASHTO design procedure (through J Factors). The ILLI-SLAB program can easily model tied PCC shoulders or a tied curb and gutter. It is recommended that the design concepts contained in the JCS-1, RISC and PCA programs be used to directly consider the effect of edge support on the traffic lane, and to structurally design the shoulder. Further joint design procedures are needed for tied concrete shoulders.

8. Transverse joint design: None of the existing design procedures provide the capability of designing a transverse joint analytically for a given project conditions. The available procedures usually provide general recommendations for dowel diameter and spacing, joint spacing, etc., but do not provide any analytical procedure to compute what is actually needed for a given traffic level and slab/foundation design. The only procedure that gives an analytical method for JPCP joint spacing is the Zero-maintenance, where thermal curling is directly considered. Therefore, new analytical procedures were developed for transverse joint design. Chapter 6 describes such procedures for joint design, and provides a microcomputer program for computing the expected faulting for different designs.

9. Slab moisture and thermal factors: The consideration of thermal curl is considered to be of utmost importance in jointed concrete pavement design. Either the WESLIQID, JSLAB or the ILLI-SLAB programs could be used to model thermal curl. The procedures for considering curling stress and combining it with load stress is also important. The procedure used in the Zero-Maintenance program is outdated and there are better approaches in some of the European design procedures that should be considered. Moisture gradients can be approximately considered as an equivalent negative thermal gradient.

10. Drainage system: The subdrainage of a rigid pavement system is considered of vital importance and must be directly considered in design. The concepts and approach used in the FHWA "Highway Subdrainage Design" manual should be used to design the subdrainage system.[128] An adequate filter layer and stiffness of the base are critical factors.

11. Climate: It is important to directly consider climatic factors such as temperature and moisture in design. This can be handled two ways. For serviceability/performance design, climatic factors can be directly included in a multiple regression model developed with field data. For mechanistic fatigue and joint design, the climatic factors can be handled through thermal gradients, moisture gradients (as equivalent thermal gradients), softening of the foundation from moisture or frost and joint opening and closing.

12. Traffic: All of the traffic load design factors can be handled by the ILLI-SLAB program. Some of the design procedures directly consider these factors, but most do not allow, for example, for a change in the lateral distribution of trucks in the traffic lane. The approach used in the

Zero-Maintenance and PCA procedures could be considered for lateral traffic distribution in the traffic lane. The truck lane distribution factors developed under NCHRP Project 1-19 could be used for design as they are the best information available.

13. Reliability of design: The basic probabilistic approach utilized in the AASHTO design guide is recommended for the serviceability/performance approach. The application of reliability concepts to the mechanistic fatigue and joint damage design is much more complicated and has not been developed.

14. Costs: The cost analysis procedures included in RPS-3 provide a reasonable approach to estimating the cost involved. Some updates to these procedures are needed however.

7.2 Design of Potential Experimental Projects

Unique rigid pavement designs were developed for experimental field testing in various climatic zones. These designs include the following features:

1. Trapezoidal slab cross section: Provides a thicker slab at the critical outer lane-shoulder edge that will reduce critical stresses and deflections. The trapezoidal section is expected to have an improved structural performance over that of a slab with uniform thickness. The cost should be the same because grading is similar and the same amount of concrete is used.

2. Widened outside traffic lane: This will result in many fewer truck wheel loads on the outer edge and corner of the the slab and reduce critical fatigue damage to an almost an interior loading condition. Maximum corner deflections at the outer wheel path will also be reduced which should reduce pumping potential. The design and construction of the longitudinal joint needs to be carefully considered to prevent longitudinal cracking on widened lanes.

3. Tied concrete shoulders: Recommended to provide further edge support and to reduce the amount of surface water entering the structural sections. The joints in the shoulder must match the joints in the adjacent traffic lane.

4. Permeable base layer: Placed directly beneath the slab along with a filter layer, these layers are expected to greatly reduce the pumping beneath the slab, which would reduce joint and crack faulting. This layer must have some stability and provide a reasonable level of support to the slab (e.g., preferably opengraded aggreagate treated with asphalt or cement). The other benefit of a permeable layer beneath the slab occurs in wet climates, where the concrete is frost damage susceptible when saturated. Much less spalling and deterioration at transverse and longitudinal joint are expected.

5. Precoated dowel bars: Reduces the amount of corrosion and joint lock-up. This problem is believed to be very serious for long jointed JRCP, and has not only resulted in joint deterioration, but also forced the opening of nearby transverse cracks.

6. Shorter joint spacing for JRCP: This is expected to reduce joint deterioration. The maximum recommended spacing of 27 ft will reduce joint opening/closing greatly, making it possible for a sealant to function to inhibit incompressibles. This is expected to reduce joint deterioration.

An experimental project is recommended for construction by State agencies to test the effects of these experimental features in different climatic regions. The experiment is recommended to be part of the Stategic Highway Research Program (SHRP), Long-Term Pavement Performance (LTPP) studies (Specific Pavement Studies). [131]

7.3 Dowel Design to Prevemt Transverse Joint Faulting

Practical guidelines and procedures for joint design have been prepared. Recommendations are given for joint spacing for JPCP and JRCP, dowel spacing and diameter and dowel coatings. Predictive mechanistic-empirical models were developed for joint faulting both with and without dowel bars. These models were programmed into a personal computer program for use by design engineers in determining joint load transfer requirements for given design situations.

The major product is the analytical procedures to assist the design engineer in determining joint load transfer design, instead of relying on outdated standards. Joint design can now for the first time directly consider the amount of traffic using the pavement and the design of the pavement in determining the amount of load transfer needed to control faulting.

7.4 Limitations in Recommendations

There still are many gaps and limitations in rigid pavement design technology. These limitations exist in the design and analysis of all rigid pavement components. One of the most serious gaps is in the lack of knowledge and analytical procedures to determine the performance of rigid pavements having highly permeable bases. Results from some experimental projects (e.g., US 10 at Clare, Michigan) indicate there may be high potential for reduction of faulting and concrete deterioration when a permeable base is used.

APPENDIX A

SPECIFIC DESIGN EVALUATION USING AASHTO GUIDE FOR NINE CLIMATIC ZONES

The analysis and results described in section 3.2 for four climatic zones should be reviewed prior to reading this appendix. A number of design situations were developed for each of the nine climatic zones described in section 3.1. The specific climatic variables for each of the nine climatic zones is shown in table 22. The design factors were the same as shown in table 25, except the changes made to include only the fine-grained subgrade soil, with dowels for JRCP and without dowels for JPCP, and shorter and longer joint spacings (JRCP only). The 1986 AASHTO Guide was used to develop designs for each of the nine design cells over a range of reliability levels from 50 to 90 percent.

Table 72 shows the specific input parameters and their values for both JPCP and JRCP designs. The climatic design inputs for the new AASHTO Guide for each of the nine climatic zones are shown in table 73. These designs then were evaluated using the models from NCHRP Project 1-19 "PREDICT" program to estimate their distresses and performance. The predictions are shown in each of the design cells from tables 74 to 79.

1. JPCP: The JPCP was designed only with fine-grained subgrade soil and without dowels. The results shown here are similar to the results in the section 3.2 which uses the data of the four major climatic regions averaged from COPES.

Performance of the pavements generally decreases as the climate becomes increasingly wet or colder, even though the design procedure adjusts the design for climate. Significant difference in performance exists generally only between the major zones, e.g., wet-freeze, wet-nonfreeze, dry-freeze and dry-nonfreeze. The drainage coefficient C_d value was found to have a significant influence on the thickness design. Severe pumping was predicted in the freeze climates especially in the wet-freeze zone. This is the typical distress which occurred in the AASHO Road Test. The higher the level of reliability, the better the predicted performance of JPCP designs. However, joint deterioration cannot be eliminated by increasing the level of reliability (or slab thickness).

The results, in general, show that the Guide designs provide adequate structural designs for JPCP over the nine climatic zones except in the wet-freeze zone (I-A cell).

2. JRCP: The JRCP was designed only with fine-grained subgrade soil. The results show that JRCP designs generally give acceptable performance in most zones, but worse in the wet freeze-thaw zone (I-B cell) and the dry-freeze zone (III-A cell). Severe pumping is the major distress in the wet-freeze and wet freeze-thaw zones. The JRCP with 27-ft (8.2 m) joint spacing shows less faulting than with 40-ft (12.2 m) joint spacing. The 40-ft (12.2 m) joint spacing also gives serious joint deterioration while the 27-ft (8.2 m) joint spacing gives better performance. The 1986 AASHTO Design Guide does not provide adequate, coherent guidance on the joint design. Higher level of reliability generally improves the predicted performance of the JRCP designs, but does not affect the amount of joint deterioration.

The results show that the Guide designs provide fair structural designs for JRCP for the nine climatic zones except in wet-freeze and wet freeze-thaw zones. It is interesting to note the performance for JRCP is worse in the I-B and the II-B zones than in either zone I-A or I-C. This may be due to a combination of climatic factors such as excessive freeze than cycles with some excess moisture present.

Based upon the similarity of these results between adjacent zones, it was decided that the four major climatic zones would be used for all further analysis:

I-A	Low temperature - high moisture.
III-A	Low temperature - low moisture.
I-C	High temperature - high moisture.
III-C	High temperature - low moisture.

Table 72. Design inputs for AASHTO performance equation for the nine climatic zones.

Parameter	JPCP	JRCP
Reliability level, %	50/80/90	50/80/90
Design period, year	20	20
Traffic, million 18-kip ESAL	15	15
* Subgrade soil	fine-grained	fine-grained
** Subbase type	4" CTB	6" granular
k-value @ top of subbase, pci	300	200
Initial serviceability	4.5	4.5
Terminal serviceability	2.5	2.5
*** Modulus of rupture, psi	650	650
Concrete E value, psi	4,000,000	4,000,000
Joint spacing, ft	15	27/40
Dowels at joints	no	yes
J factor	4.1	3.2

* Subgrade M_R = 3,000 psi for fine-grained soil, and 7,000 psi for coarse-grained soil

** Subbase E = 1,000,000 psi for CTB and 30,000 psi for granular

*** Third-point loading, at 28 days

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 73. Climatic design inputs for AASHTO guide for the nine climatic zones.

JPCP	Climatic zones	III-A	II-A	I-A
	C_d value	1.0	.85	.8
	LS factor	.5	.75	1.0
	Corrected k-value, pci	175	140	100
	Climatic zones	III-B	II-B	I-B
	C_d value	.95	.85	.8
	LS factor	.5	.75	1.0
	Corrected k-value, pci	175	140	100
	Climatic zones	III-C	II-C	I-C
	C_d value	.9	.8	.8
	LS factor	.5	.75	1.0
	Corrected k-value, pci	175	140	100
JRCP	Climatic zones	III-A	II-A	I-A
	C_d value	1.0	.85	.8
	LS factor	1.0	1.5	2.0
	Corrected k-value, pci	70	41	24
	Climatic zones	III-B	II-B	I-B
	C_d value	.95	.85	.8
	LS factor	1.0	1.5	2.0
	Corrected k-value, pci	70	41	24
	Climatic zones	III-C	II-C	I-C
	C_d value	.9	.8	.8
	LS factor	1.0	1.5	2.0
	Corrected k-value, pci	70	41	24

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 74. Predictions for JPCP using the AASHTO Guide for the nine climatic zones - 50% reliability level.

Design Traffic: 15 million 18-kip ESAL
Design Period: 20 years
Joint Spacing: 15 ft
Subbase: 4 inches CTB
Level of Reliability: 50%

	III-A	II-A	I-A
Slab Thickness, in	10.3	11.3	11.8
Dowel Diameter, in	0	0	0
Pumping	1.9	1.9	1.9
Faulting, in	0.12	0.12	0.12
Cracking, ft/mi	274	146	94
Jt. Det., jts/mi	12	12	12
PSI	3.7	3.4	2.9

	III-B	II-B	I-B
Slab Thickness, in	10.6	11.3	11.8
Dowel Diameter, in	0	0	0
Pumping	0.3	0.4	0.6
Faulting, in	0.09	0.08	0.08
Cracking, ft/mi	89	74	53
Jt. Det., jts/mi	12	12	12
PSI	3.9	3.5	3.3

	III-C	II-C	I-C
Slab Thickness, in	10.9	11.7	11.8
Dowel Diameter, in	0	0	0
Pumping	0.2	0.3	0.7
Faulting, in	0.06	0.06	0.06
Cracking, ft/mi	58	43	31
Jt. Det., jts/mi	12	12	12
PSI	4.0	3.8	3.4

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 75. Predictions for JPCP using the AASHTO Guide for the nine climatic zones - 80% reliability level.

Design Traffic: 15 million 18-kip ESAL
Design Period: 20 years
Joint Spacing: 15 ft
Subbase: 4 in CTB
Level of Reliability: 80%

	III-A	II-A	I-A
Slab Thickness, in	11.3	12.4	12.4
Dowel Diameter, in	0	0	0
Pumping	1.4	1.5	1.7
Faulting, in	0.12	0.11	0.11
Cracking, ft/mi	132	79	52
Jt. Det., jts/mi	12	12	12
PSI	3.8	3.6	3.1

	III-B	II-B	I-B
Slab Thickness, in	11.6	12.4	12.9
Dowel Diameter, in	0	0	0
Pumping	0	0	0
Faulting, in	0.08	0.07	0.07
Cracking, ft/mi	55	45	33
Jt. Det., jts/mi	12	12	12
PSI	4.1	3.7	3.5

	III-C	II-C	I-C
Slab Thickness, in	11.9	12.8	12.9
Dowel Diameter, in	0	0	0
Pumping	0	0	0.3
Faulting, in	0.06	0.05	0.05
Cracking, ft/mi	36	27	19
Jt. Det., jts/mi	12	12	12
PSI	4.1	4.0	3.6

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 76. Predictions for JPCP using the AASHTO Guide for the nine climatic zones - 90% reliability level.

Design Traffic: 15 million 18-kip ESAL
Design Period: 20 years
Joint Spacing: 15 ft
Subbase: 4 in CTB
Level of Reliability: 90%

	III-A	II-A	I-A
Slab Thickness, in	11.8	13.0	13.0
Dowel Diameter, in	0	0	0
Pumping	1.2	1.3	1.5
Faulting, in	0.11	0.11	0.11
Cracking, ft/mi	97	59	40
Jt. Det., jts/mi	12	12	12
PSI	3.9	3.7	3.2

	III-B	II-B	I-B
Slab Thickness, in	12.1	13.0	13.5
Dowel Diameter, in	0	0	0
Pumping	0	0	0
Faulting, in	0.08	0.07	0.07
Cracking, ft/mi	44	36	26
Jt. Det., jts/mi	12	12	12
PSI	4.2	3.8	3.6

	III-C	II-C	I-C
Slab Thickness, in	12.5	13.4	13.5
Dowel Diameter, in	0	0	0
Pumping	0	0	0
Faulting, in	0.05	0.05	0.05
Cracking, ft/mi	28	21	15
Jt. Det., jts/mi	12	12	12
PSI	4.3	4.1	3.7

1 in = 2.54 cm
1 ft = 0.3048 m
1 psi = 0.07031 kg/cm^2
1 psi/in = 0.02768 kg/cm^3

Table 77. Predictions for JRCP using AASHTO Guide for the nine climatic zones - 50% reliability level.

Design Traffic: 15 million 18-kip ESAL
Design Period: 20 years
Subbase: 6 in granular
Level of Reliability: 50%

	III-A		II-A		I-A	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	9.4"	9.4"	10.4"	10.4"	10.9"	10.9"
Dowel Diameter	1.125"	1.125"	1.25"	1.25"	1.375"	1.375"
*As	0.047	0.069	0.052	0.077	0.054	0.08
Pumping	0	0	0.9	0.9	2.1	2.1
Faulting, in	0.05	0.09	0.2	0.07	0.03	0.08
Cracking, ft/mi	1263	1263	977	977	1009	1009
Jt. Det., jts/mi	0	0	0	50	0	50
PSI	3.4	3.3	3.6	3.5	3.4	3.3

	III-B		II-B		I-B	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	9.6"	9.6"	10.4"	10.4'	10.9"	10.9"
Dowel Diameter	1.25"	1.25"	1.25"	1.25"	1.375"	1.375"
As	0.048	0.071	0.052	0.077	0.054	0.08
Pumping	0	0	1.4	1.4	2.5	2.5
Faulting, in	0.01	0.06	0.02	0.02	0.05	0.09
Cracking, ft/mi	1187	1187	991	991	1206	1206
Jt. Det., jts/mi	0	36	0	36	0	36
PSI	3.5	3.4	3.4	3.3	3.3	3.2

	III-C		II-C		I-C	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	9.9"	9.9"	10.7"	10.7"	10.9"	10.9"
Dowel Diameter	1.24"	1.25"	1.375"	1.375"	1.375"	1.375"
As	0.049	0.073	0.053	0.079	0.054	0.08
Pumping	0	0	0.5	0.5	0.5	0.5
Faulting, in	0.01	0.06	0	0.04	0.01	0.06
Cracking, ft/mi	1097	1095	922	920	927	927
Jt. Det., jts/mi	0	35	0	35	0	35
PSI	3.5	3.4	3.5	3.4	3.2	3.1

*NOTES: As = Area of reinforcement, in^2/ft width of slab.

1 in = 2.54 cm
1 psi/in = 0.02768 kg/cm^3
1 psi = 0.07031 kg/cm^2
1 ft = 0.3048 m

Table 78. Predictions for JRCP using AASHTO Guide for the nine climatic zones - 80% reliability level.

Design Traffic: 15 million 18-kip ESAL
Design Period: 20 years
Subbase: 6 in granular
Level of Reliability: 80%

	III-A		II-A		I-A	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	10.3"	10.3"	11.3"	10.3"	11.8"	11.8"
Dowel Diameter	1.25"	1.25"	1.375"	1.375"	1.5"	1.5"
*As	.051	.076	.056	.083	.059	.087
Pumping	0	0	0.5	0.5	1.7	1.7
Faulting, in	0.01	0.06	0	0.04	0	0.06
Cracking, ft/mi	999	997	842	842	839	840
Jt. Det., jts/mi	0	0	0	50	0	50
PSI	3.7	3.6	3.7	3.6	3.5	3.4

	III-B		II-B		I-B	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	10.5"	10.5"	11.3"	11.3'	11.8"	11.8"
Dowel Diameter	1.25"	1.25"	1.375"	1.375"	1.5"	1.5"
As	052	.078	.056	.083	.059	.087
Pumping	0	0	1.0	1.0	2.2	2.2
Faulting, in	0	0.05	0	0.04	0.02	0.07
Cracking, ft/mi	958	954	843	843	944	944
Jt. Det., jts/mi	0	36	0	36	0	36
PSI	3.7	3.7	3.7	3.6	3.4	3.3

	III-C		II-C		I-C	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	10.8"	10.8"	11.7"	11.7"	11.8"	11.8"
Dowel Diameter	1.375"	1.375"	1.5"	1.5"	1.5"	1.5"
As	0.54	.08	.058	.086	.059	.087
Pumping	0	0	0	0	1.3	1.3
Faulting, in	0	0.04	0	0.02	0	0.04
Cracking, ft/mi	904	904	800	800	801	802
Jt. Det., jts/mi	0	35	0	35	0	35
PSI	3.7	3.6	3.6	3.5	3.3	3.2

*NOTES: As = Area of reinforcement, in^2/ft with of slab.

1 in = 2.54 cm
1 psi/in = 0.02768 kg/cm^3
1 psi = 0.07031 kg/cm^2
1 ft = 0.3048 m

Table 79. Predictions for JRCP using AASHTO Guide for the nine climatic zones - 90% reliability level.

Design Traffic: 15 million 18-kip ESAL
Design Period: 20 years
Subbase: 6 in granular
Level of Reliability: 90%

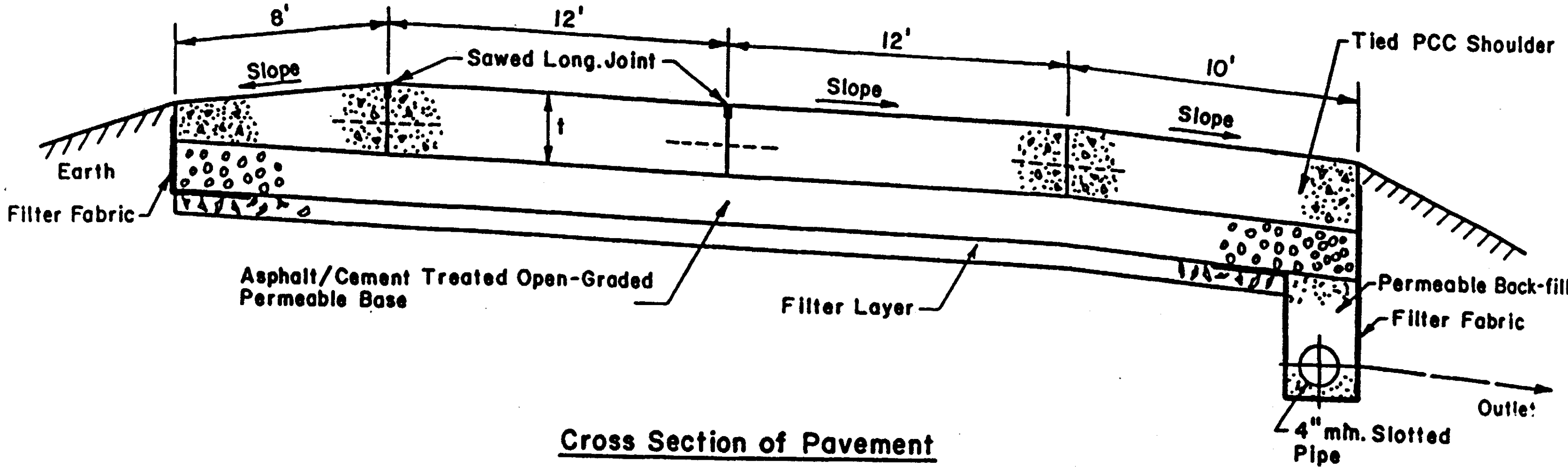
	III-A		II-A		I-A	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	10.7"	10.7"	11.8"	11.8"	12.4"	12.4"
Dowel Diameter	1.375"	1.375"	1.5"	1.5"	1.5"	1.5"
*As	.053	.079	.059	.087	.062	.092
Pumping	0	0	.3	.3	1.6	1.6
Faulting, in	0	0.04	0	0.03	0	0.05
Cracking, ft/mi	924	922	792	793	780	780
Jt. Det., jts/mi	0	50	0	50	0	50
PSI	3.8	3.7	3.8	3.7	3.5	3.4
	III-B		**II-B**		**I-B**	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	11"	11"	11.8"	11.8"	12.5"	12.4"
Dowel Diameter	1.375"	1.375"	1.5"	1.5"	1.5"	1.5"
As	.055	.081	.059	.087	.062	.092
Pumping	0	0	.8	.8	2.0	2.0
Faulting, in	0	0.04	0	0.03	0.01	0.06
Cracking, ft/mi	876	877	792	792	855	855
Jt. Det., jts/mi	0	36	0	36	0	36
PSI	3.8	3.7	3.7	3.6	3.4	3.4
	III-C		**II-C**		**I-C**	
Joint Spacing	27'	40'	27'	40'	27'	40'
Slab Thickness	11.3"	11.3"	12.2"	12.2"	12.4"	12.4"
Dowel Diameter	1.375"	1.375"	1.5"	1.5"	1.5"	1.5"
As	0.56	.083	.061	.09	.062	.092
Pumping	0	0	0	0	1.1	1.1
Faulting, in	0	0.03	0	0.02	0	0.03
Cracking, ft/mi	840	840	761	761	754	754
Jt. Det., jts/mi	0	35	0	35	0	35
PSI	3.8	3.7	3.7	3.6	3.3	3.3

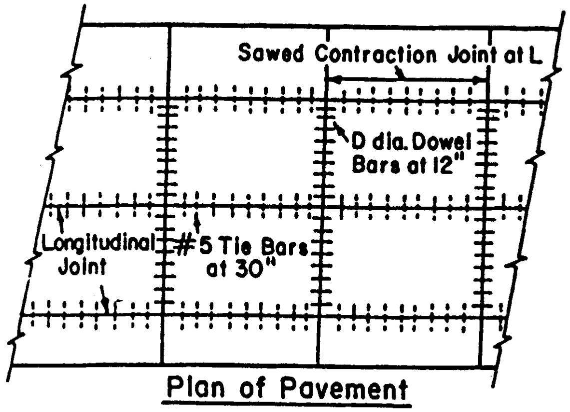
*NOTES: As = Area of reinforcement, in^2/ft width of slab.

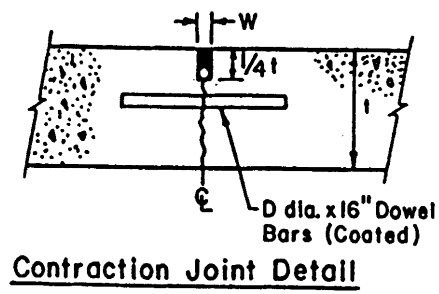
1 in = 2.54 cm 1 psi = 0.07031 kg/cm^2
1 ft = 0.3048 m 1 psi/in = 0.02768 kg/cm^3

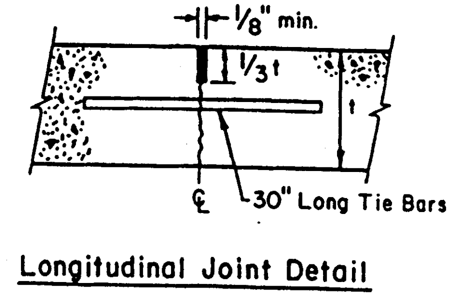
APPENDIX B

NEW RIGID PAVEMENT EXPERIMENTAL DESIGNS







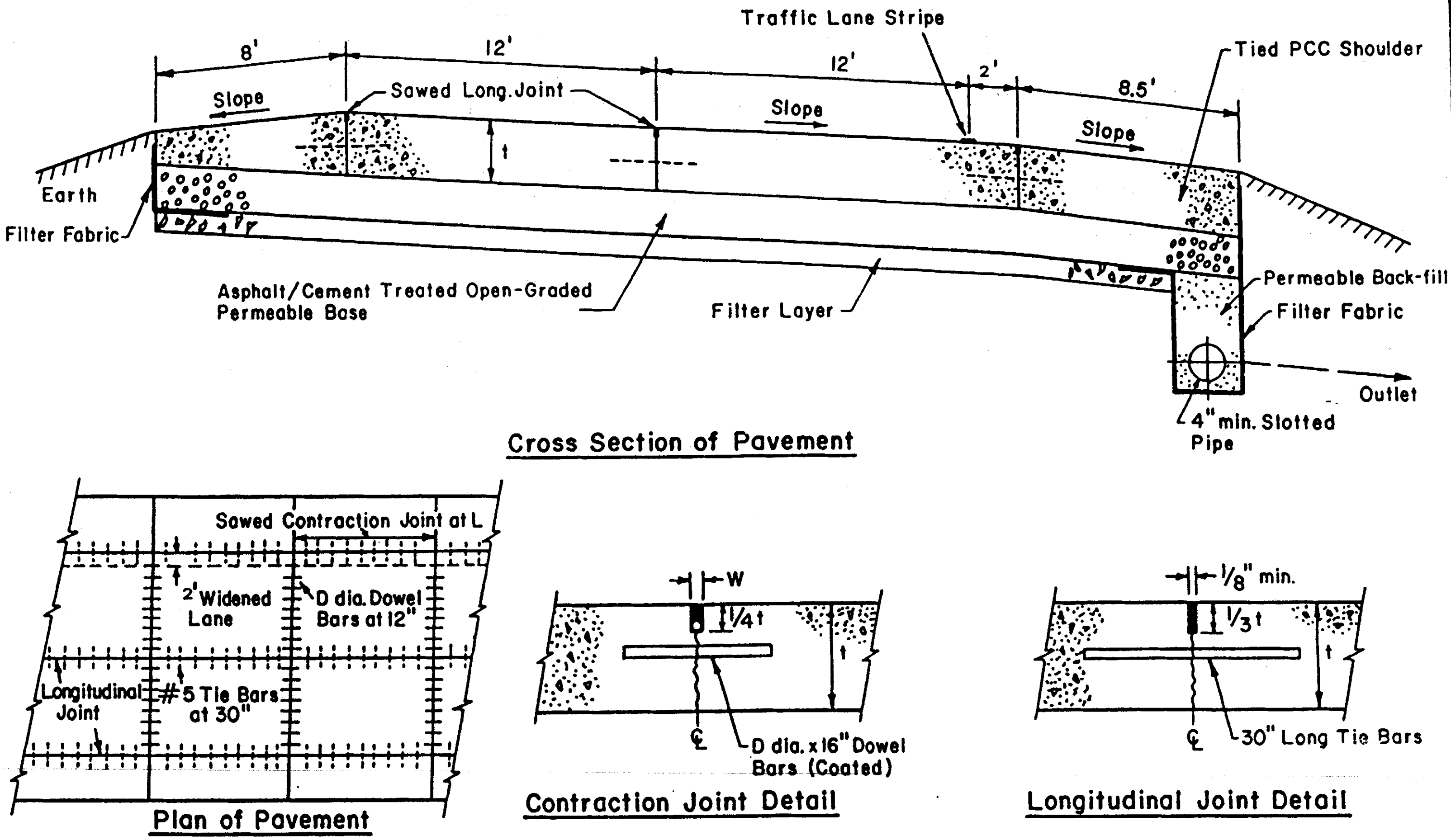


General Notes

1. Slab thickness (t), dowel diameter (D) and transverse joint spacing (L) to be determined by analysis of traffic loadings and subgrade support at site.
2. The gradation of the open-graded permeable base material must meet the requirements in the "Highway Subdrainage Design" manual (FHWA, 1980).
3. Pavement type may be either JPCP or JRCP. Transverse joint spacing for JRCP is a maximum of 30 ft.
4. The only difference in design between climatic zones is joint reservoir dimension, depth of drainage pipe and frost heave protection. See attached information.

JOINTED CONCRETE PAVEMENT
Proposed DESIGN ID: NO. 1
Type of Highway: Interstate
Suitable Environment Condition: See General Note 4.
DESIGN: UNIVERSITY OF ILLINOIS

Figure 63. Proposed Design ID: No. 1.



General Notes

1. Slab thickness (t), dowel diameter (D) and transverse joint spacing (L) to be determined by analysis of traffic loadings and subgrade support at site.
2. The gradation of the open-graded permeable base material must meet the requirements in the "Highway Subdrainage Design" manual (FHWA, 1980).
3. Pavement type may be either JPCP or JRCP. Transverse joint spacing for JRCP is a maximum of 30 ft.
4. The only difference in design between climatic zones is joint reservoir dimension, depth of drainage pipe and frost heave protection. See attached information.

JOINTED CONCRETE PAVEMENT

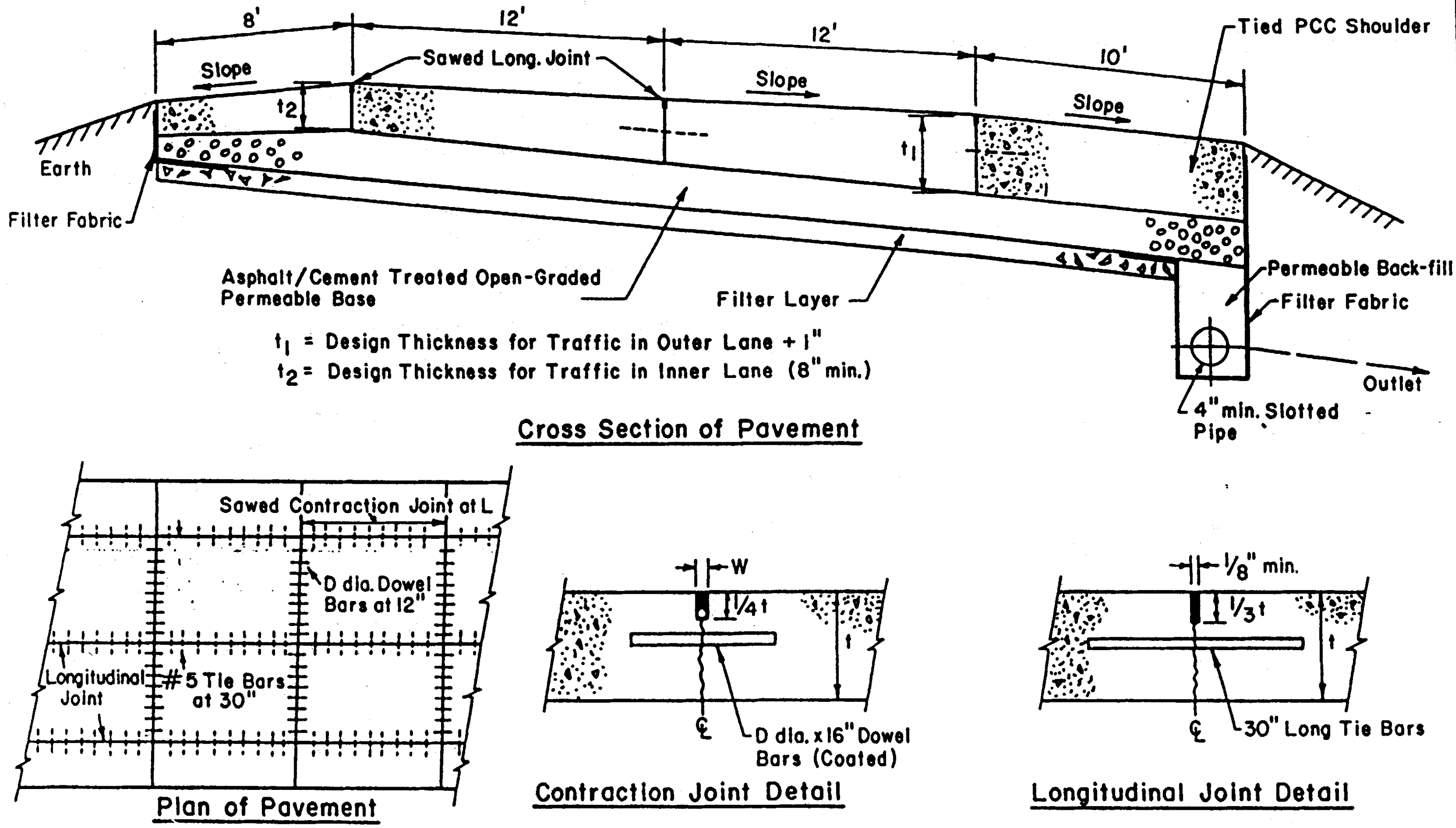
Proposed DESIGN ID: NO. 2

Type of Highway: Interstate

Suitable Environment Condition:
See General Note 4.

DESIGN: UNIVERSITY OF ILLINOIS

Figure 64. Proposed Design ID: No. 2.

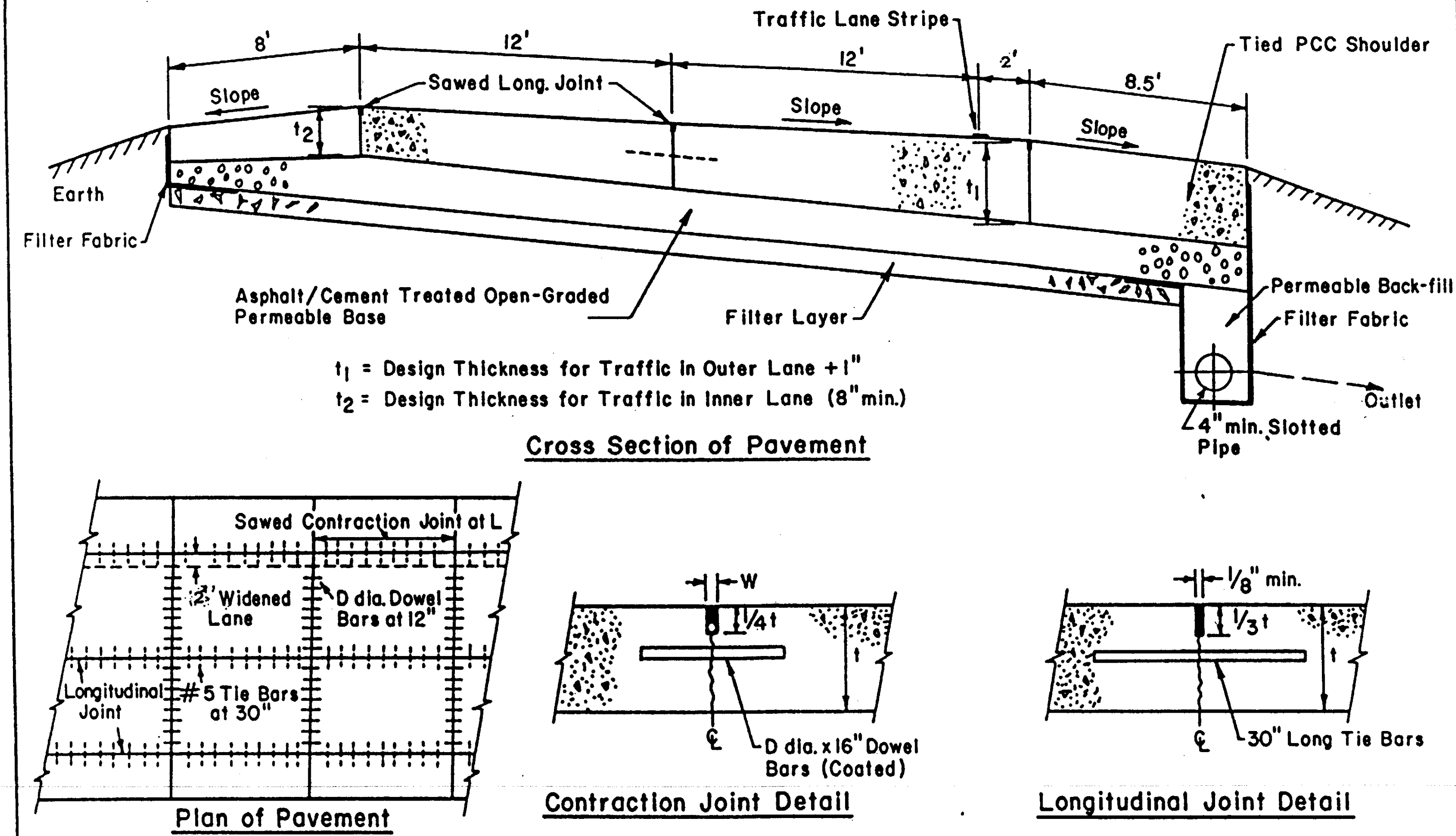


General Notes

1. Slab thickness (t), dowel diameter (D) and transverse joint spacing (L) to be determined by analysis of traffic loadings and subgrade support at site.
2. The gradation of the open-graded permeable base material must meet the requirements in the "Highway Subdrainage Design" manual (FHWA, 1980).
3. Pavement type may be either JPCP or JRCP. Transverse joint spacing for JRCP is a maximum of 30 ft.
4. The only difference in design between climatic zones is joint reservoir dimension, depth of drainage pipe and frost heave protection. See attached information.

JOINTED CONCRETE PAVEMENT
Proposed DESIGN ID: NO. 3
Type of Highway: Interstate
Suitable Environment Condition: See General Note 4.
DESIGN: UNIVERSITY OF ILLINOIS

Figure 65. Proposed Design ID: No. 3.



General Notes

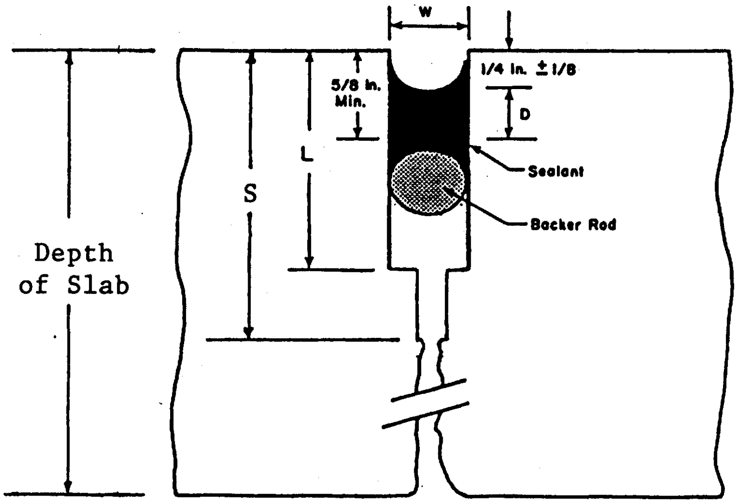
1. Slab thickness (t), dowel diameter (D) and transverse joint spacing (L) to be determined by analysis of traffic loadings and subgrade support at site.
2. The gradation of the open-graded permeable base material must meet the requirements in the "Highway Subdrainage Design" manual (FHWA, 1980).
3. Pavement type may be either JPCP or JRCP. Transverse joint spacing for JRCP is a maximum of 30 ft.
4. The only difference in design between climatic zones is joint reservoir dimension, depth of the drainage pipe and frost heave protection. See attached information.

JOINTED CONCRETE PAVEMENT
Proposed DESIGN ID: NO. 4
Type of Highway: Interstate
Suitable Environment Condition: See General Note 4.
DESIGN: UNIVERSITY OF ILLINOIS

Figure 66. Proposed Design ID: No. 4.

ATTACHMENT 1

1. Recommendation for Transverse Joint Dimension Design:
   1) Low modulus silicone or preformed compression sealant is recommended for use in joint sealing in all nine climatic zones.
   2) Recommendations for width of silicone sealant versus transverse joint spacing are shown as follows:



S = Depth of initial saw cut (1/4 slab depth)

Joint Spacing (ft)	D (in)	L (in)	W (in) Climatic Zones I-A	II-A	III-A	I-B	II-B	III-B	I-C	II-C	III-C
< 15	.25	1.25	.25*	.25	.25	.25	.25	.25	.25	.25	.25
16 - 20	.25	1.25	.25	.25	.25	.25	.25	.25	.25	.25	.25
21 - 25	.25	1.50	.375	.375	.375	.25	.25	.25	.25	.25	.25
26 - 30	.25	1.50	.375	.375	.375	.375	.375	.375	.25	.25	.25

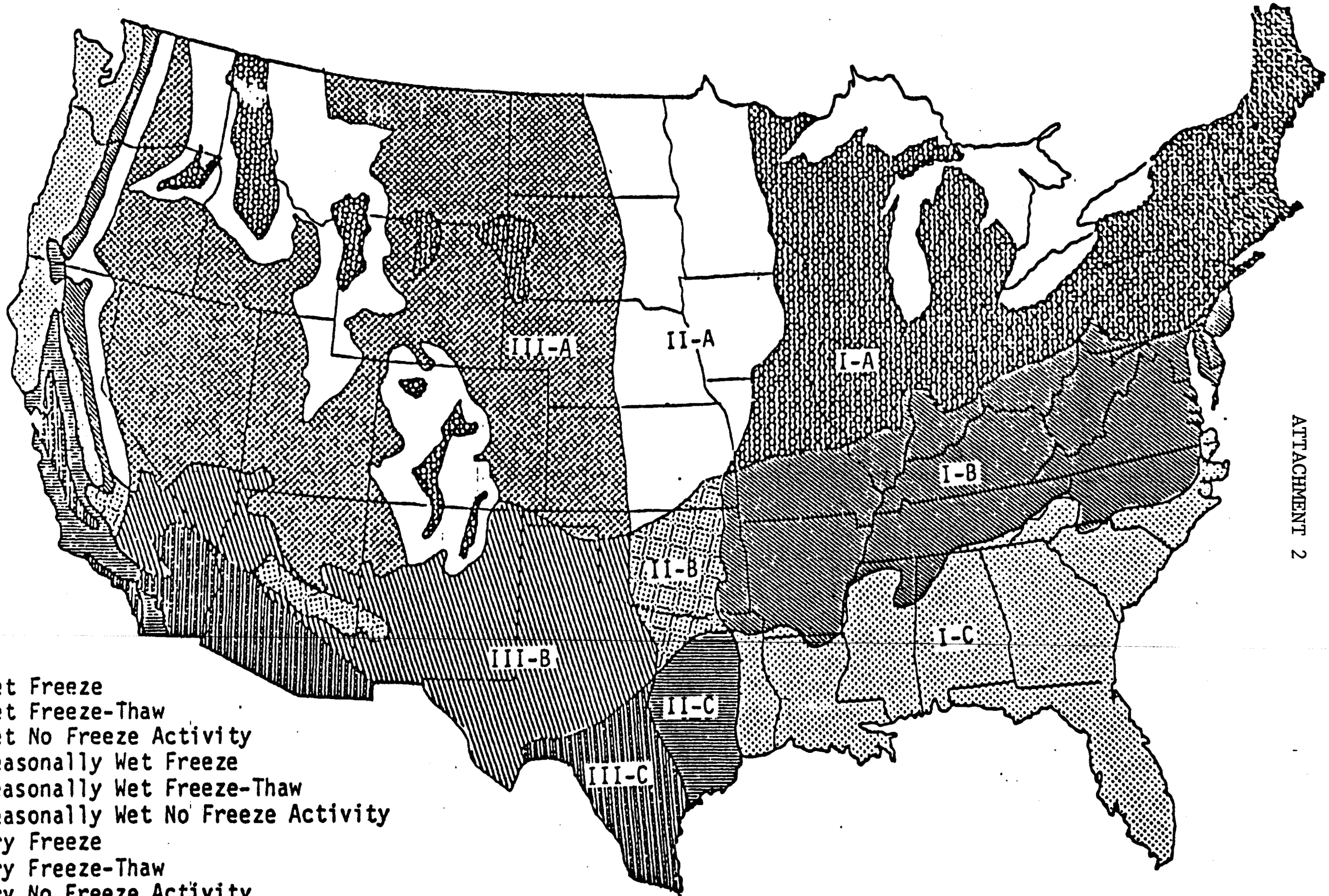
* Recommended silicone joint sealant width in inches

   3) A minimum width of 0.25 in is recommended for longitudinal joints.

2. The depth of drainage pipe to be installed should be below the frost penetration line to protect the pipe from freezing up and to provide effective collection of water flow from beneath the pavement.

3. In deep freeze climates, measures to prevent frost heaving should be taken.

ATTACHMENT 2



I-A, Wet Freeze
I-B, Wet Freeze-Thaw
I-C, Wet No Freeze Activity
II-A, Seasonally Wet Freeze
II-B, Seasonally Wet Freeze-Thaw
II-C, Seasonally Wet No Freeze Activity
III-A, Dry Freeze
III-B, Dry Freeze-Thaw
III-C, Dry No Freeze Activity

Climatic Zones Derived from Thornthwaite Calculations, Based on Material Behavior.

Recommended Research and Performance Evaluation Work Plan

I. Objectives of Design:

An experimental project containing a minimum of four new unique jointed concrete pavement designs is recommended. It is possible that one such experimental project could be constructed in each of the nine climatic zones in the United States. However, the main objective is to obtain results in each State where the experiment is constructed to assist them in rigid pavement design. A full factorial design is recommended as shown below with two major factors that are varied and numerous factors that are held constant. An individual State could vary additional factors if desired, but these two should be investigated as a minimum to make it possible to analyze the effect of climatic zones across the U.S. The designs of the individual sections are based on providing adequate transverse joint load transfer, reducing JRCP joint movement, positive subdrainage, edge support, reduction in slab curling and reduction in critical edge stresses and deflections. The experimental designs are as follows:

	DESIGN SECTION NO.			
UNIQUE FEATURES	1	2	3	4
Main Factors:				
1) Tapered crosssection			x	x
2) Widened lane		x		x
Constant Factors:				
Tied PCC shoulder	x	x	x	x
Asphalt/cement treated open-graded permeable base	x	x	x	x
Longitudinal drainage pipe	x	x	x	x
Precoated dowel bars	x	x	x	x
Shorter joint spacing (JRCP)	x	x	x	x
Slab thickness (constant)	x	x	x	x

These four test sections shall be constructed sequentially (in random order) along the same roadway to achieve uniform soil support, traffic loading and climatic condition. The recommended length for each test section is 1500 ft plus a 500-ft transition. At least two replicate sections and any desired conventional design sections used by the state may also be constructed along the test site at random locations.

II. Data Measuring Devices Required:

The data collection procedure of this project shall be similar to the Strategic Highway Research Program (SHRP) for the Long-Term Pavement Performance (LTPP) studies.[131] The types of data measuring devices required for data collection are provided in the Data Collection Guide of the SHRP LTPP.

III. Data Analysis and Evaluation Procedure:

The data analysis procedure shall be similar to that developed for the SHRP LTPP studies. The data obtained from the experimental projects in each State where it is constructed shall be used to:

- Compare the performance and costs of unique versus conventional designs.
- Develop improved designs, particularly with regard to joint design, subdrainage, edge support and slab thickness.

Further analysis can be conducted as to the effect of climate if the experimental project is constructed in different climatic zones.

APPENDIX C

DRAINAGE ANALYSIS AND DESIGN FOR IMPROVED CONCRETE PAVEMENT DESIGN

INTRODUCTION

It is difficult to separate the design of subsurface drainage from the design of other elements of a highway. Although this has been done in the past, it is not recommended that this even be attempted. Pavement design procedures have now integrated strutural considerations into the thickness design procedure for the addition and maintenance of good drainage in a pavement system. A major consideration when including drainage systems is to ensure the drainage system will collect and remove all water entering the pavement. This section deals with the analysis of subsurface drainage for recommended concrete pavement sections in which open-graded material is used as the base using the Highway Subdrainage Design Manual as a guide for the drainage design.[128]

The analysis and design of highway subsurface drainage systems involves the consideration of subsurface water from two sources:

- Groundwater, which is defined as the water existing in the natural ground in the zone of saturation below the pavement.
- Infiltration, which is defined as surface water that gets into the pavement structural section by seeping down through joints or cracks in the pavement surface or from ditches along the side of the road.

In the present analysis, only gravity type ground water will be designed for, and it is assumed that artesian conditions are absent. The free water from melting ice lenses that commonly exists above the water table during the spring thaw is also accounted for in the analysis when it is applicable.

ELEMENTS OF SUBDRAINAGE

In order to have an effective subdrainage system, the following elements must be incorporated into the design of the drainage system:

- Interceptor - That portion of the drainage system that directly interrupts the flow of water and diverts it from its path toward the pavement.
- Collector - That portion of the drainage system that allows the water being intercepted to move away from the point of interception without damaging the pavement or surrounding soil.
- Removal - That portion of the drainage system that ensures the water being intercepted and collected will be removed from the site with sufficient speed that the water will not interact with the pavement materials.

Pavement designers today are beginning to recognize the necessity of good drainage, and innovative drinage systems are being design and constructed for both new designs and rehabilitation projects. Wick drains are beig used for problem areas, vertical fin drains are being installed, and new materials are being used in standard longitudinal drains.[128,129,130] For pavement design, the most common generic applications of drainage are:

- The drainage blanket.
- The longitudinal drain.
- The transverse and horizontal drain combination.

Each of these installations has advantages and disadvantages, particularly in the type of water the drainage installation will remove from the pavement. This is one of the most important considerations in selecting a drainage installation.

Drainage Blanket

The horizontal drainage blanket can be used separately beneath a pavement or as an integral part of the pavement load carrying structure. The drainage blanket used in the pavement structure removes water from surface or side infiltration. Unless the water table is exceptionally high or artesian conditions exist, the system in figure 67 will not remove water table water. When the water table is high enough, the drainage layer can remove the groundwater from both gravity and artesian sources as shown in figure 68. The use of a layer to serve as a drainage system requires an adequate thickness of material with very high coefficient of permeability, a positive outlet for the water collected, and in some instances, the use of one or more protective filter layers.

Longitudinal Drain

The longitudinal drain is located parallel to the roadway centerline both in horizontal and vertical alignment. Depending on the geometry of the system, and the depth to water table, this drainage system can function as an interceptor or a collector. A deep trench is generally used when it is desired to lower the water table. Shallow trenches are used when the removal of surface infiltration is the reason for drainage. Figure 67 illustrates the use of longitudinal drains as the collector for the drainage blanket. The longitudinal drain typically requires the addition of a collector pipe and a protective filter of some kind.

Transverse and Horizontal Drains

Subsurface drains that run transversely beneath the roadway are classified as transverse drains. Transverse drains may involve a trench, collector pipe and protective filter. When the general direction of the groundwater flow tends to be parallel to the roadway, transverse drains can be more effective than longitudinal drains in intercepting the flow.

DATA REQUIRED FOR ANALYSIS AND DESIGN

There are three categories of data required for analysis and design of subsurfaçe drainage:

- The geometry of the flow domain.
- The properties of the materials in the pavement and in the drain.
- The climatological data.

Geometry of Flow Domain

Nearly every geometric design features of a highway can exert some influence upon the analysis and design of subsurface drainage. For the analysis of initial design being presented here, only the cross section of pavement is known. Other information such as longitudinal grades, depth of cut and fill, and details of ditches and other surface drainage facilities are not available. Specific assumptions with respect to these quantities will be made.

Material Properties

The most important property that controls the flow of subsurface water is the coefficient of permeability of material. An assumption has to be made with regard to the permeability of subgrade material. The coefficient of permeability of open-graded materials used in this analysis were obtained from figure 69.

Climatological Data

Based on the results of infiltration tests performed on pavements in Connecticut, it was recommended that, for design purposes, the infiltration rate be taken as 2.4 ft^3/day/ft of crack (0.22 m^3/day/m). In this analysis an infiltration rate of 3.0 ft^3/day/ft (0.28) has been used. An indication of the depth to which freezing temperatures may penetrate into pavement or underlying subgrade can be helpful in assessing the seriousness of possible frost action. Maps giving average or maximum depth of frost penetration such as figure 70 may be very helpful.

QUANTITY OF WATER TO BE REMOVED

Infiltration

Infiltration can be estimated at 2.4 ft^3/day/ft (0.22) of crack as the results of infiltration tests performed on pavements in Connecticut suggest. The following expression is recommended to more accurately estimate infiltration rate, q_i:

$$q_i = I_c[N_c/W + W_c/Wc_s] + k_p \qquad (1)$$

where:

q_i = the design infiltration rate (ft^3/day/ft of drainage layer)
I_c = the crack infiltration rate (ft^3/day/ft of crack)
N_c = the number of contributing longitudinal cracks (ft)
W_c = the length of contributing transverse crack or joints (ft)
W = the width of granular base subjected to infiltration (ft)
c_s = the spacing of transverse cracks or joints (ft)
k_p = the rate of infiltration through the uncracked pavement surface (ft^3/day/ft)

For portland cement concrete pavements, k_p is practically insignificant and is ignored in this analysis. To be more conservative, a value of 3.0 $ft^3/day/ft$ for I_c is used in this analysis. The value of N_c is taken as:

$$N_c = N+1 \qquad (2)$$

where N is the number of traffic lanes.

The value of c_s is taken as the regular transverse joint spacing.

Groundwater

For the case of gravity drainage, the average inflow, q_g, is estimated using figure 71. In this figure, L_i is the radius of influence obtained from the expression:

$$L_i = 3.8\ (H-H_0) \qquad (3)$$

Where $H-H_0$ is the amount of drawdown of water table in feet.

Once the value of q_2, the upward flow, is obtained from figure 71 the average inflow rate q_g is obtained from the relationship:

$$q_g = q_2/0.5W \qquad (4)$$

Meltwater

The inclusion of meltwater from ice lenses is a critical component of the water to be removed from pavements in the northern United States. Table 80 and figure 72 may be incorporated in estimating the quantity of flow attributed to melting ice lenses, and are self-explanatory.

Combining Flows

The following combinations of flows can occur depending on the highway cross section, and are used in this analysis:

$$q_n = q_i \qquad (5)\ \text{cut}$$
$$q_n = q_i+q_g \qquad (6)\ \text{cut}$$
$$q_n = q_i+q_m \qquad (7)\ \text{cut or fill}$$

In this analysis, artesian flow and the outflow are not considered, as stated previously.

ANALYSIS AND DESIGN OF DRAINAGE LAYERS

Once the quantity of flow rate, q_n has been computed, determination of thickness H_d, and permeability k_d, of the drainage layer required to transmit the inflow to a suitable outlet is the next step. H_d and k_d can be determined by the use of figure 73. Since only the cross section of pavement is known and other factors such as groundwater condition, topography of area, subgrade material properties are unknown, a sensitivity analysis has been carried out to show the degree of importance of each factor in the final design.

Sensitivity Analysis

Fill

The section shown in figure 74 is used for the analysis in this problem. The following assumptions were made:

1. $k_{subgrade} = 2.7*10^{-5}$ ft/day ML-CL soil $(1.7*10^{-5}$ m/day).
2. $I_c = 3.0$ ft^3/day/ft (0.28 m^3/day/m).
3. $k_{open-graded} = 6000$ ft/day. (1829 m/day)
4. Groundwater table at considerable depth.

The transverse slope, g_t, was a variable.

Since the groundwater table is at considerable depth, it is reasonable to assume that $q_g = 0.0$, and the quantity of flow through the pavement can be obtained from equation 7.

The quantity of flow from melted ice, q_m, obtained from figure 72 is 0.006 ft^3/dat/ft. This assumes high frost potential for the subgrade soil.

Flow from infiltration is obtained from equation 1. The following variables are assumed to be used in equation 1:

1. slab length = 15 ft (4.6 m).
2. N_c=2+1=3.
3. w_c=34 ft
4. w=24 ft
5. c_s=5 ft
6. The slope variable g_t was used as 1 and 2 percent:

The results of the calculation procedure yielded the following thicknesses for the open graded base:

g_t % = 1 %, Hd = 2.6 in (6.6 cm)
g_t % = 2 %, Hd = 1.7 in (4.3 cm)

The calculated thickness for this situation is 4 in (10.2 cm). It is apparent from the above analysis that the recommended thickness is more than adequate.

An important variable in this analysis is the permeability of subgrade material and its frost characteristic. Working with the same thickness and material for the open-graded base, 4 in, an adequate permeability of the subgrade is to 0.6 ft/day (0.18 m/day). This value was obtained by back-calculating the quantity of flow, q_n, and then the rate of flow from ice melt, q_m, was obtained. It was assumed that the quantity of flow from infiltration, q_i, was equal to 1.3 ft^3/day/ft^2. It was also assumed that the subgrade material had very high frost susceptibility. These combinations lead to a very high quantity of flow from ice melt, q_m. It would be expected that the design inflow rate from this source would be substantially lower than the value obtained from this analysis.

Moreover, material with the coefficient of permeability of 0.6 ft/day (0.18 m/day) may have a much lower frost susceptibility than what was assumed in this analysis.

Cut (Hilly terrain)

The crosssection shown in figure 75 was used for this analysis. Since a deep longitudinal trench is being used on the side of pavement it may be reasonable to assume that flow from the groundwater through the open-graded base itself will be minimal. Therefore, equation 7 is used to obtain q_n. The following assumptions were made in the analysis:

1. $k_{subgrade} = 2.7x10^{-5}$ ft/day ($1.7*10^{-5}$ m/day)
2. Subgrade soil ML-CL.
3. $I_c = 3.0$ cf/day/sf (0.9 $m^3/day/m^2$).
4. The dimensions of the pavement were the same as used in the previous example.
5. The longitudinal slope, g_1 is again a variable factor.

Given the above information, the following thicknesses are obtained:

g_1%	$H_d$in.
3	2.6 (6.6 cm)
4	3.4 (8.6 cm)

From the above analysis, it becomes clear that the longitudinal slope is a very critical factor in the design of an open-graded base. It is recommended that for greater longitudinal slopes the use of transverse drains be incorporated into subsurface drainage.

Groundwater Flow

This is a case where a shallow longitudinal side drain is used in the pavement instead of a deep one and the permeability of subgrade material is large enough that groundwater seeps into the drainage blanket as shown in figure 76. The following assumptions have been made here:

1. $k_{subgrade} = 0.03$ ft/day (0.009 m/day).
2. The same pavement dimensions were used.
3. The variable in this examination is $H - H_0$, the amount of drawdown of the groundwater table.

Using figure 77, a number of trials were examined and q_g was obtained for each trial. The results are tabulated below:

$H - H_0$ (ft.)	q_g ($ft^3/day/ft^2$)
5	0.008
10	0.01
50	0.012

These calculations show that change in the drawdown has little effect on the quantity of the flow from the groundwater into the drainage blanket. This highlights the fact that the most influential factor in figure 77 is the permeability of the subgrade material. Assuming that $q_i = 1$ $ft^3/dat/ft^2$ and $H - H_0 = 5.0$ following thicknesses for a given open-graded and subgrade material were obtained:

H_d in	$k_{o,p}$ ft/day	k_s ft/day
4	6000	6.7
4	14000	20.0
4	36000	57.0
5	6000	12.7
5	14000	34.0
5	36000	93.0

Again, the thickness of 4 in (10.2 cm) would appear to satisfy nearly all conditions of moisture in the pavement. Thicker drainage blankets would be needed to satisfy structural layer requirements, but not drainage.

The following discussion is provided to highlight important considerations in the overall design and implementaion of a drainage system which are separate from sizing the installation. They have been directly extracted from the FHWA Highway Drainage Manual.[128] Designers are referred to the manual for more design details.

FILTER REQUIREMENTS

It has long been recognized that when water flows from a fine grained soil into a coarser one there is a tendency for particles of the finer soil to be washed into the voids of the coarse soil. This can lead to clogging and an overall reduction of permeability. It has also been established that this tendency for intrusion of fines into the pores of a granular material can be initiated or aggravated by the pumping action caused by the repetitive loading of traffic. Thus, it is particularly important that measures be taken to prevent pavement drainage layers from becoming contaminated in this way.

In order to protect the drainage layers from intrusion of fines, the granular material must satisfy certain filter criteria. If these criteria are not satisfied, then a protective filter must be designed and placed between the fine and coarse soils to prevent intrusion and clogging. If a filter cannot be constructed, filter fabrics should be considered. If some recognition of filter requirements is not taken, the drainage system will fail.

Commonly a protective filter consists of a layer of granular soil whose gradation and other characteristics satisfy established filter criteria. However, in recent years a number of different types of drainage fabrics and mats have become available and have been used for this purpose. The choice between aggregate filters and drainage fabric should be based on a careful evaluation of the history of performance, availability and economy. The following criterion is recommended to guide the design of protective filter:

$$(d_{15})_{filter} <= 5(d_{85})_{protected\ soil}$$
$$(d_{15})_{filter} => 5(d_{15})_{protected\ soil}$$
$$(d_{50})_{filter} <= 25(d_{50})_{protected\ soil}$$
$$(d_5)_{filter} => 0.074\ mm$$
$$(c_u)_{filter} = (d_{60})_{filter}/(d_{10})_{filter} <= 20$$

Special consideration should be given to the stability of the granular drainage layer and filter materials during construction. Experience has shown that, while certain open graded drainage layer and filter materials are stable when confined, they may lack the stability required for ease in placement and compaction, during which time there is little confinement. Often, stability can be achieved with a minor adjustment in gradation with little sacrifice in permeability.

COLLECTION SYSTEMS

The collection system is most commonly a set of perforated or slotted pipes that remove water from the pavement drainage layer and convey it to suitable outlets (removal system) outside of the roadway limits. The design of such systems includes consideration of the following:

- The type of pipe to be utilized.
- The location and depth of transverse and longitudinal collectors and their outlets.
- The slope of the collector pipes.
- The size of the pipes.
- Provisions for adequate filter protection to provide sufficient drainage capacity and to prevent flushing of drainage aggregates into the pipes through the slots or perforations.

A wide variety of types and sizes of suitable pipes is readily available in most localities, making the selection of the pipe depend upon the specific soil conditions at the site, load requirements, required durability of the pipe, and environmental considerations, including the possible presence of corrosive conditions.

The required thickness and permeability of a pavement drainage layer are also dependent upon the length of the path the water most take in flowing out of this layer. The length of this flow path, in turn, is largely dependent upon the location of the longitudinal and transverse collector drains.

In many instances, the longitudinal roadway grade or the cross slope governs the grade of the collector pipes, i.e. the pipes are simply set at a constant depth below the roadway surface. However, practical construction and operational factors dictate that slopes of collector pipes should not be less than one percent for smooth bore pipes and two percent for corrugated pipes. Thus, in areas where the longitudinal grade or cross slope is very flat, it may be necessary to steepen the grade of the collector pipe to meet these minimum requirements. Since the size and flow capacity of the collectors will be dependent in part upon the pipe gradient, in some instances, it may be advisable to consider the steepening of the pipe gradient in order to achieve a reduction in pipe size. Minimum recommended diameters for PVC pipes and all other pipes are 3 inches and 4 in, respectively.

The position of the longitudinal collectors within the roadway cross section and their depth is dependent upon a number of factors including the desirability of draining the shoulder area, the likelihood of frost action, the depth of frost penetration, and economic considerations. In many situations where there is no significant depth of frost penetration and where it is not necessary to attempt to drawdown a high groundwater table the longitudinal collector pipes can be placed in shallow trenches as shown in figure 76. However, where there is a substantial depth of frost penetration, deeper trenches should be used as shown in figure 77.

There are no rules for establishing the location of transverse collector drains. However, once the preferred direction of flow within the pavement drainage layer has been established, it is a simple matter to select trial locations of the transverse collectors so as to control the length of the flow path in such a way that reasonably consistent thickness of drainage layers are produced. Of course, transverse collector drains should always be provided at critical locations, such as at gradepoints and adjacent to superelevation transition zones, where the cross slope approaches zero. Transverse collectors (interceptor drains) will be required at more frequent intervals where the longitudinal grade is steep relative to the cross slope and where a groundwater condition is present.

Many features of the design of longitudinal collector drains are also applicable to transverse collectors. Included in these are the requirements for minimum pipe size and gradient, adequate depth to minimize the effects of freezing, and adequate filter protection to prevent both flushing of drainage aggregates into pipe perforations and slots and clogging of the drain backfill by fines carried into the drain by groundwater.

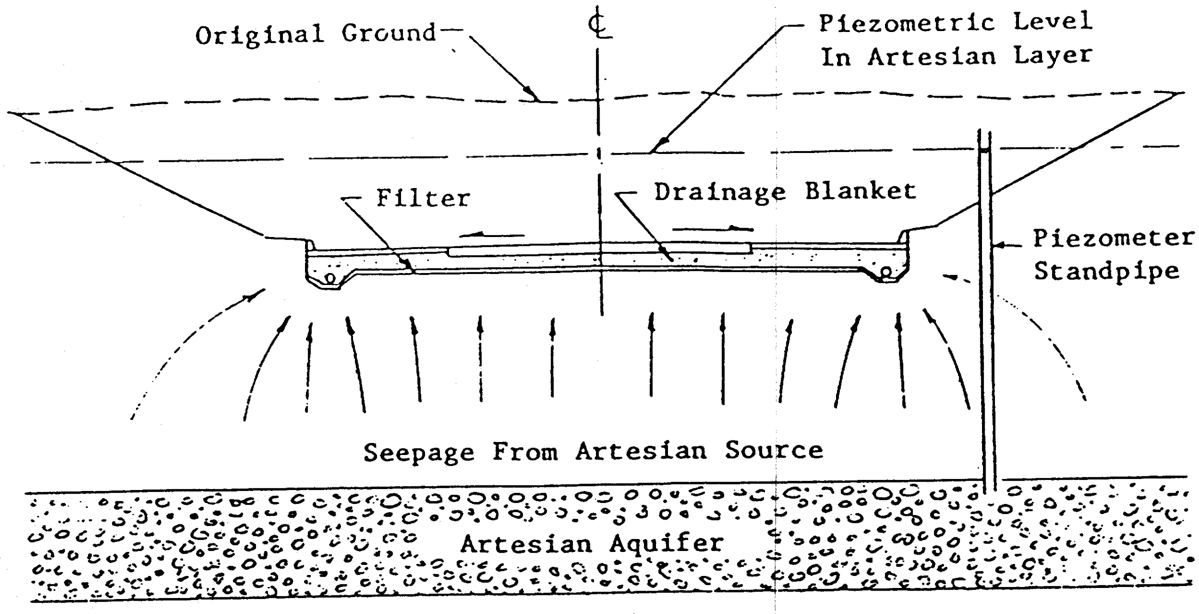
There has been some adverse experience associated with the installation of transverse drains in areas of seasonal frost, where a general frost heaving has occurred except where the transverse drains were installed, thus leading to poor riding quality during winter months. The possibility of such occurrences should be given careful consideration during the design stage. If the use of transverse drains is considered absolutely necessary under existing conditions consideration should be given to methods of minimizing the frost heaving and its effects.

The location of outlets in the removal system is often dictated by topographic and geometric features of the highway and the overall drainage pattern adjacent to the highway. Since the size of the longitudinal collector pipes is dependent upon the outlet spacing, this feature of the collection system should be given very careful consideration.

Perhaps the controlling feature of the removal system is the exit point. It must be protected from natural and manmade hazards. The protection generally consists of a combination of screens or valves and a marker.

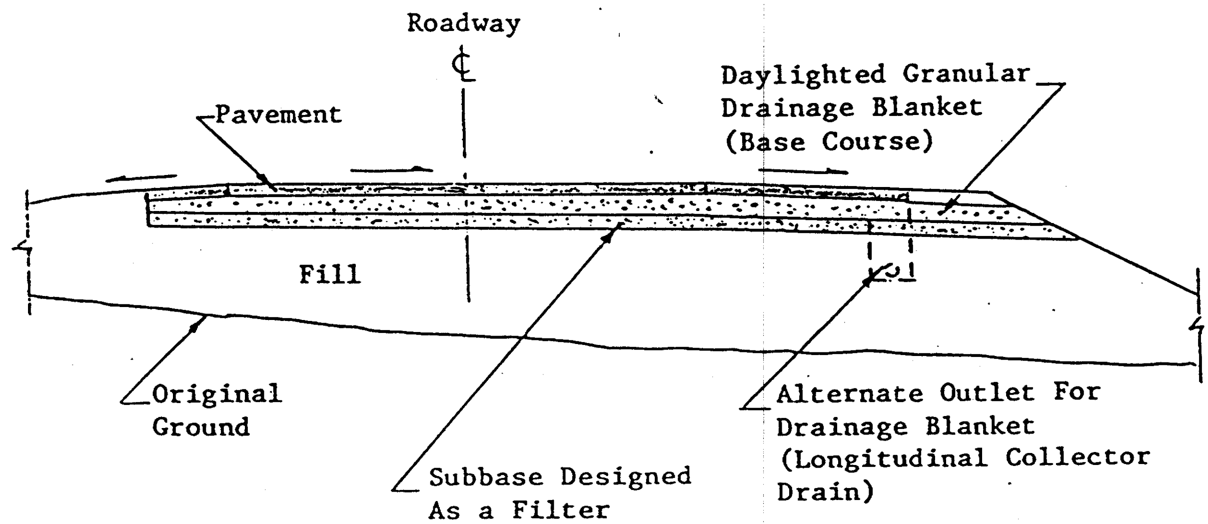
SUMMARY

This section has demonstrated that drainage layers can be designed with materials and dimensions that are acceptable in todays pavements. The size restrictions will be more related to structural adequacy and construction limitations than to the ability to remove water from the pavement. Design procedures can be utilized that include the drainage layer as a structural component of the final pavement. It is the responsibility of the pavement design engineer to make certain that the field conditions have been thoroughly evaluated to determine the need for drainage additions to the pavement being designed. There can be no doubt any more that pavements with moisture present will not perform as well as a pavement that has the moisture removed with a suitable drainage system.



(a)

Figure 67. Drainage Blanket for Intercepting Groundwater.



(b)

Figure 68. Drainage blanket for intercepting surface infiltration, with alternate longitudinal drain.

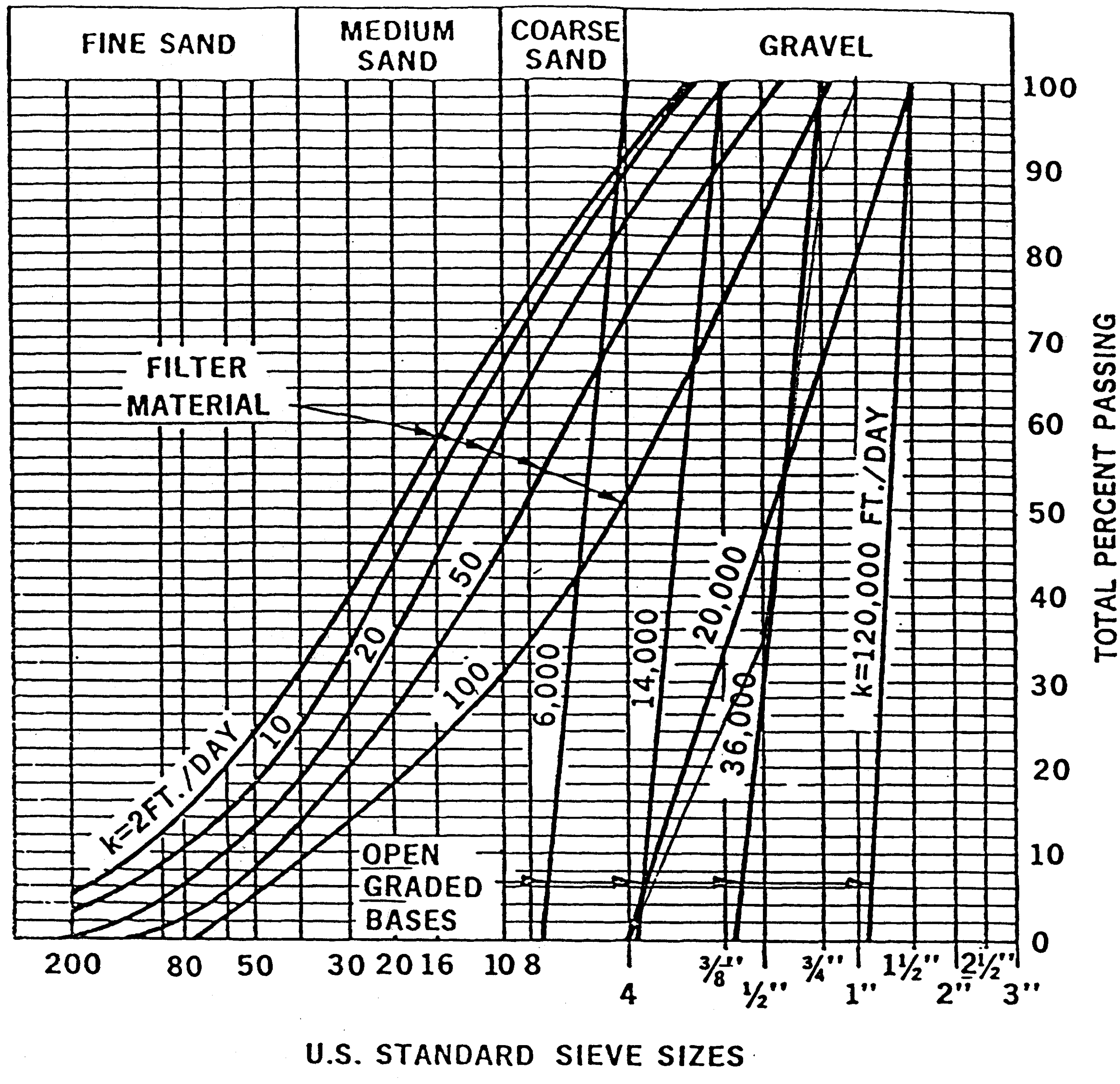


Figure 69. Typical gradations and permeabilities of open-graded bases and filter materials. (5,16)

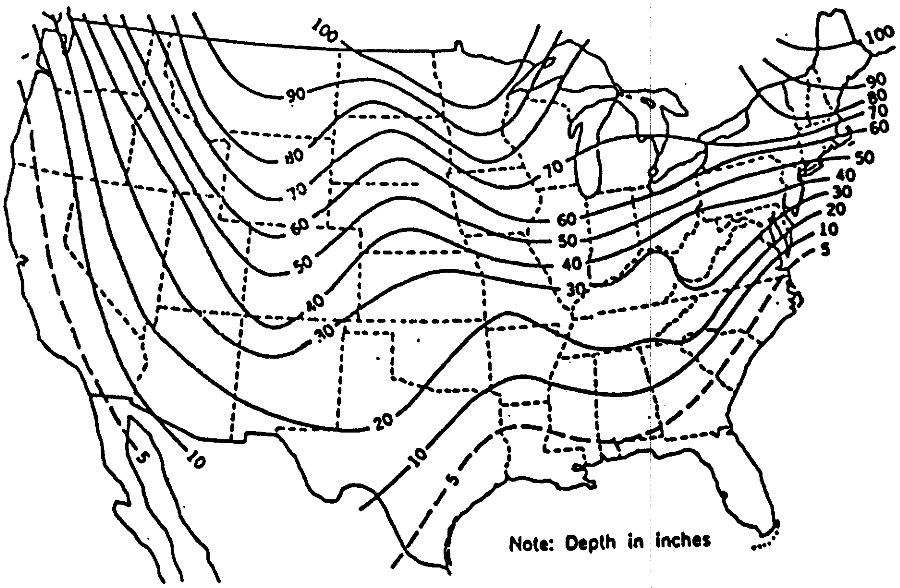


Figure 70. Maximum depth of frost penetration in the United States. (69)

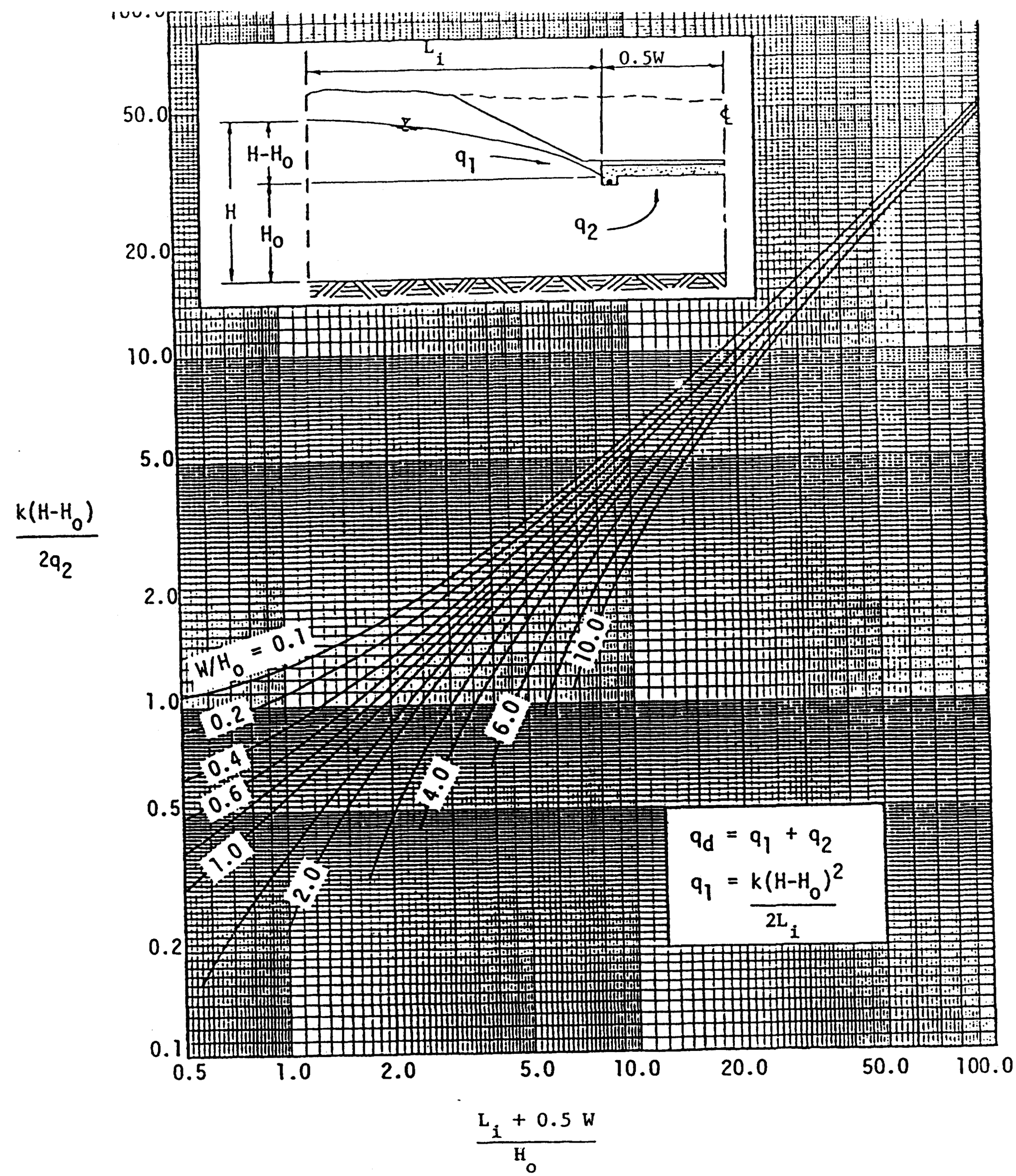


Figure 71. Chart for determingin flow rate in horizontal drainage blanket.(12,75)

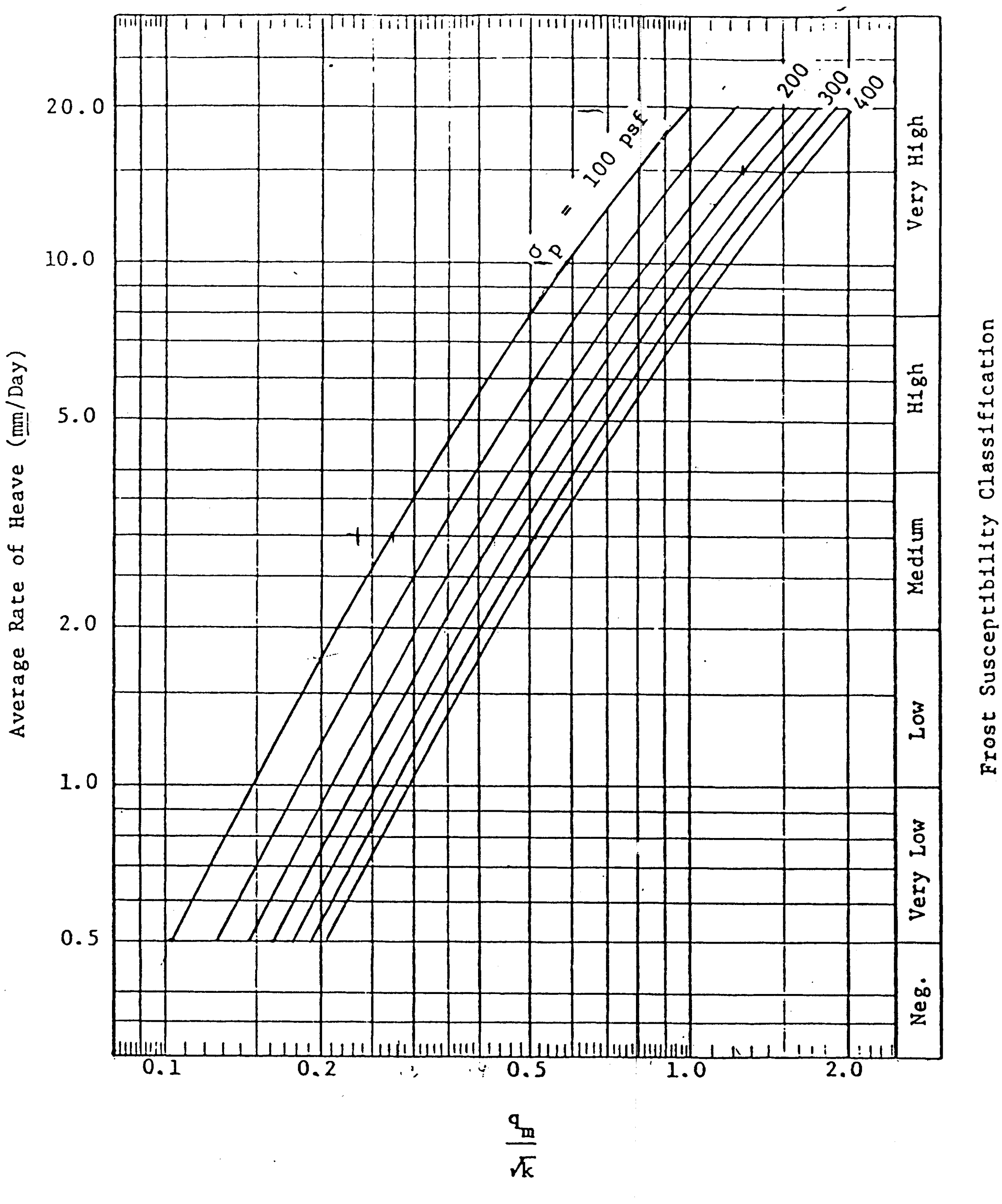


Figure 72. Chart for estimating design inflow rate of melt water from ice lenses.

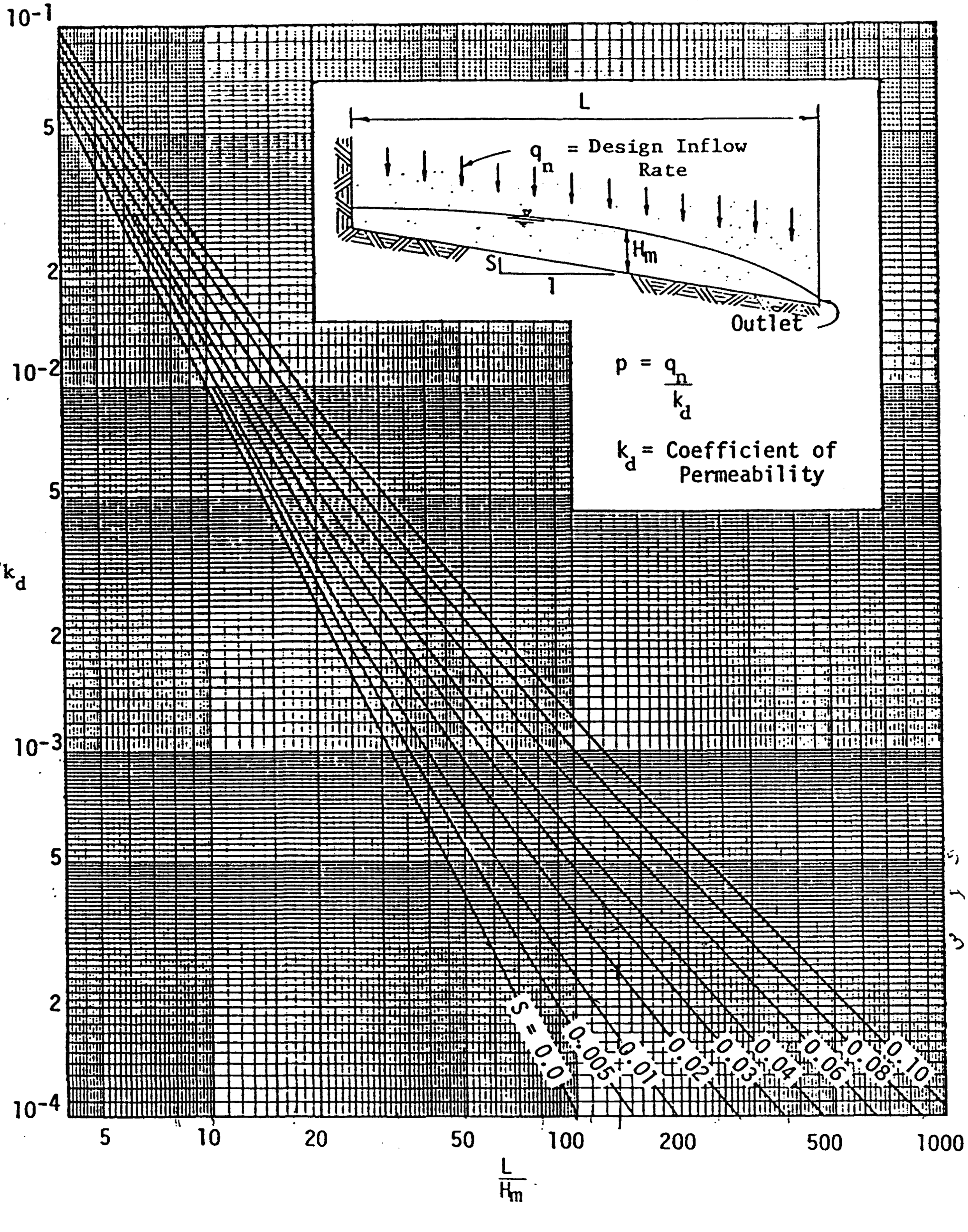


Figure 73. Chart for estimating maximum depth of flow caused by steady inflow. (12)

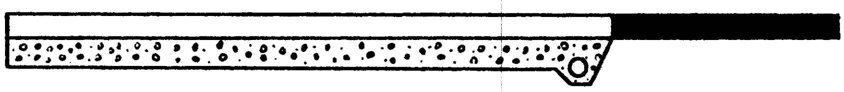


Figure 74. Pavement geometry for drainage layer analysis.

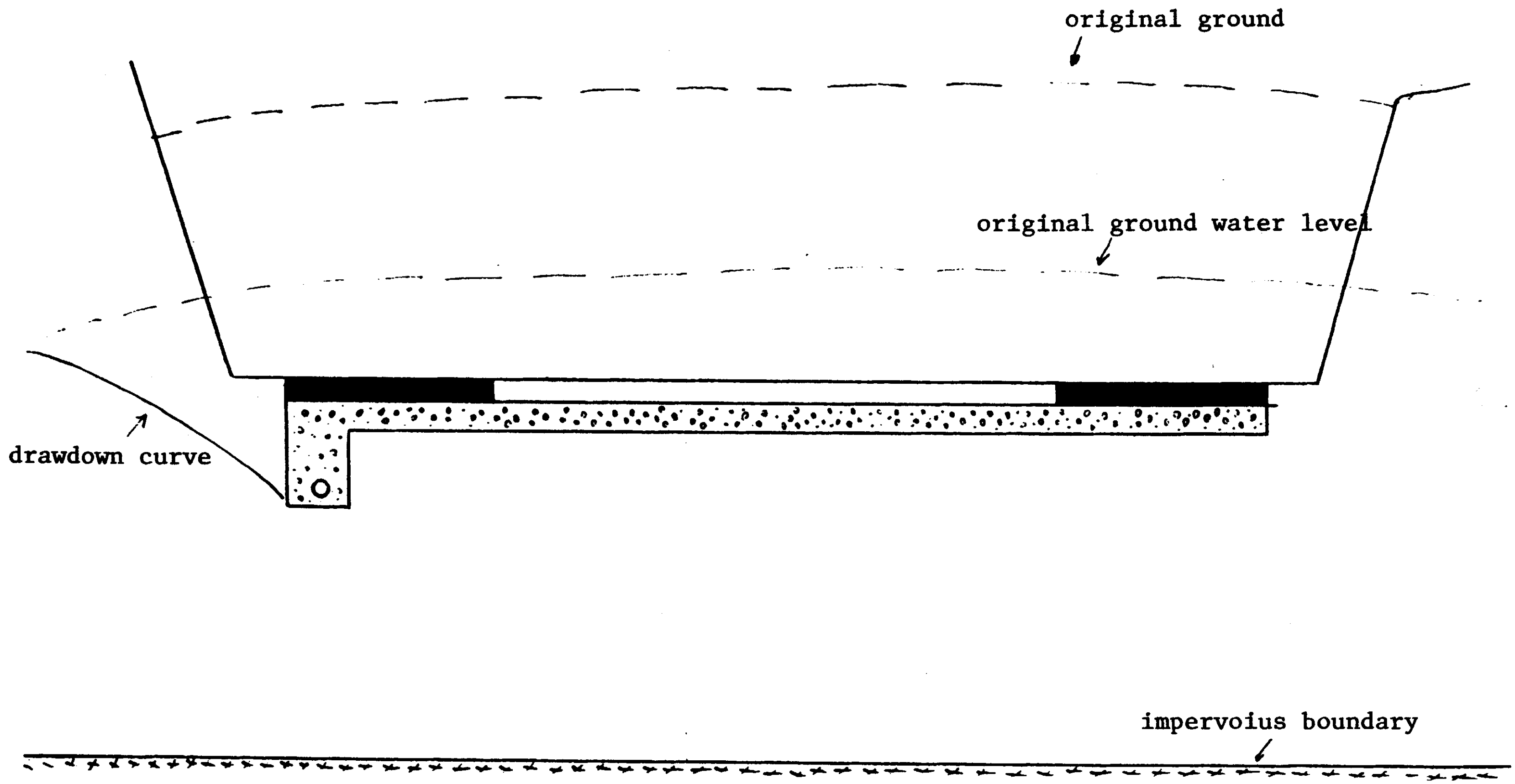
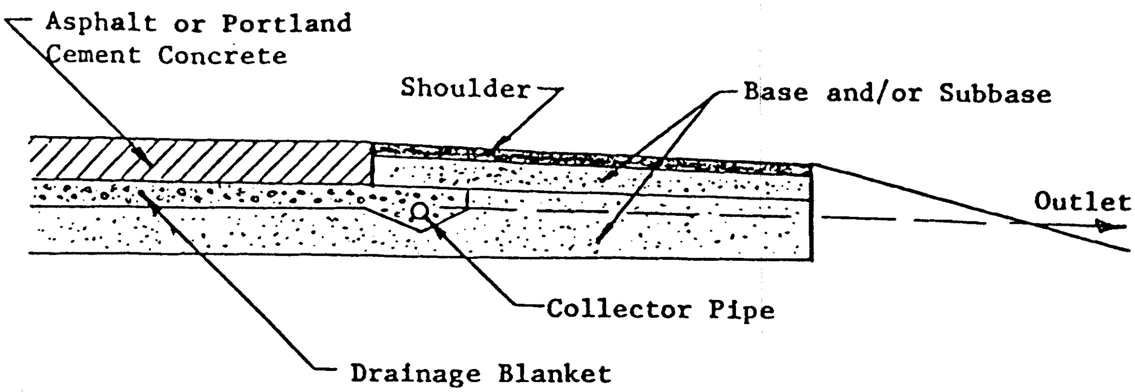
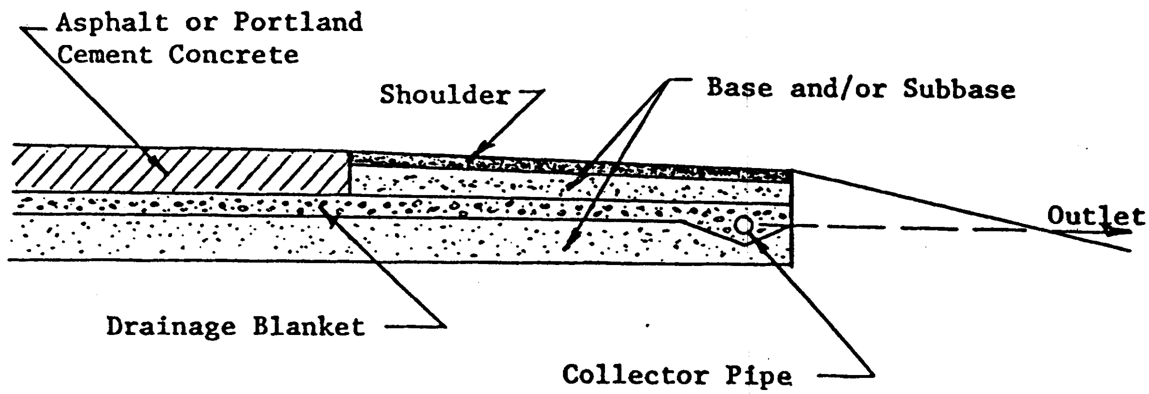


Figure 75. Use of longitudinal drain to lower water table.

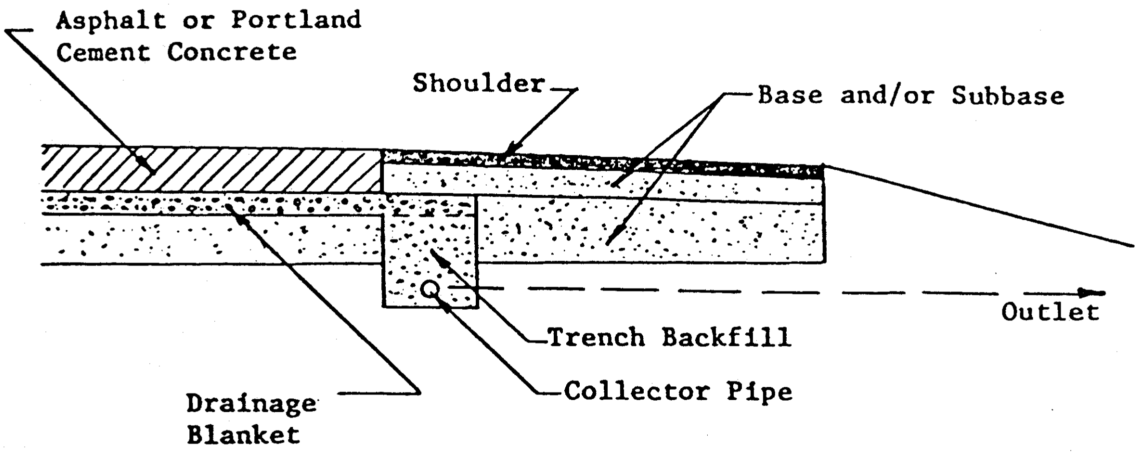


(a)

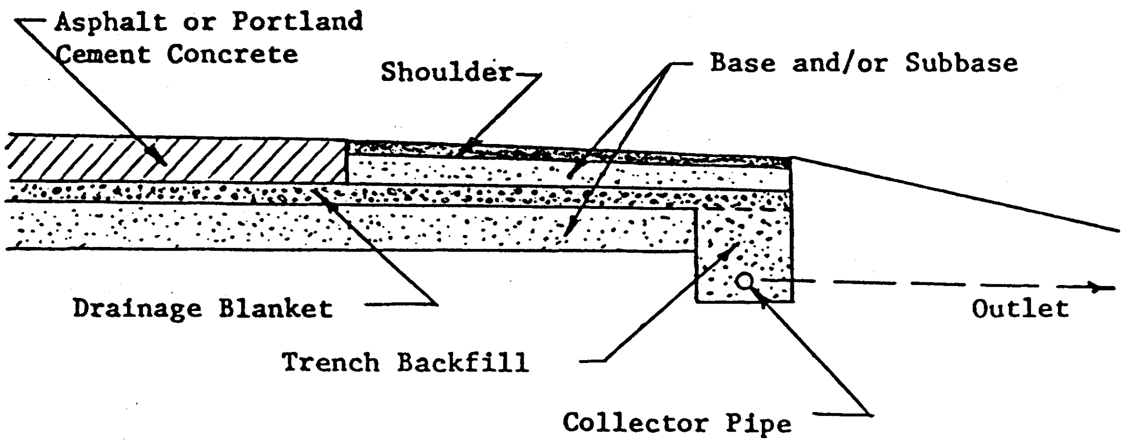


(b)

Figure 76. Typical location of shallow longitudinal collector pipes.



(a)



(b)

Figure 77. Typical location of deep longitudinal collector pipes.

Table 80. Guidelines for selection of heave rate or frost susceptibility classification.

Unified Classification Soil Type	Symbol	Percent < 0.02 mm	Heave Rate mm/day	Frost Suscept. Classification
Gravels and Sandy Gravels	GP	0.4	3.0	Medium
	GW	0.7-1.0	0.3-1.0	Neg. to Low
		1.0-1.5	1.0-3.5	Low to Medium
		1.5-4.0	3.5-2.0	Medium
Silty and Sandy Gravels	GP-GM	2.0-3.0	1.0-3.0	Low to Medium
	GW-GM	3.0-7.0	3.0-4.5	Medium to High
	GM			
Clayey and Silty Gravels	GW-GC	4.2	2.5	Medium
	GM-GC	15.0	5.0	High
	GC	15.0-30.0	2.5-5.0	Medium to High
Sands and Gravely Sands	SP	1.0-2.0	0.8	Very Low
	SW	2.0	3.0	Medium
Silty and Gravely Sands	SP-SM,	1.5-2.0	0.2-1.5	Neg. to Low
	SW-SM,	2.0-5.0	1.5-6.0	Low to High
	SM	5.0-9.0	6.0-9.0	High to Very High
		9.0-22.0	9.0-5.5	
Clayey and Silty Sands	SM-SC	9.5-35.0	5.0-7.0	High
	SC			
Silts and Organic Silts	ML-OL,	23.0-33.0	1.1-14.0	Low to Very High
	ML	33.0-45.0	14.0-25.0	Very High
		45.0-65.0	25.0	Very High
Clayey Silts	ML-CL	60.0-75.0	13.0	Very High
Gravely and Sandy Clays	CL	38.0-65.0	7.0-10.0	High to Very High
Lean Clays	CL	65.0	5.0	High
	CL-OL	30.0-70.0	4.0	High
Fat Clays	CH	60.0	0.8	Very Low

APPENDIX D

EXPERIMENTAL PROJECT RESEARCH STUDY - NEW DESIGNS FOR RIGID PAVEMENTS

PROJECT DESCRIPTION FORM (PDF)

EXPERIMENTAL OBJECTIVE

Determine the performance of new unique jointed concrete pavement designs that emphasize load transfer, subdrainage and reduction of edge stresses and deflections. The effect of climatic conditions will also be determined as the experimental projects are constructed in different climatic areas. This form is to be used by agencies to express their interest in participating and to designate candidate project(s) for consideration.

ADMINISTRATIVE SECTION

1. Participating Agency

Name of Agency: ______________________________

Principal Contacts:

Name ____________	Name ____________
Title ____________	Title ____________
Address ____________	Address ____________
City ____________	City ____________
State ______ Zip ______	State ______ Zip ______
Telephone ( ) ____________	Telephone ( ) ____________
( ) ____________	( ) ____________

2. Technical Assistance Requested by Participating Agency to perform performance monitoring (equipment, instrumentation, computer hardware/software, etc.):

____

____

____

____

____

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TECHNICAL SECTION (Potential Project Site and Environment)

Potential Site A

Site Location:

Highway designation (example: I74) ____________

Nearby city ____________________________________

Bound direction___________

Start mile post _________ End mile post ___________

Start station No. ________ End station No. ___________

Project length (mile) ____________

Traffic:

Current ADT __________ truck % ___________

18-kip ESALs per year __________________

Project Type:

New alignment or reconstruction of existing pavement ______

No. of traffic lanes (one direction) ____________

Urban/Rural ___________

Climatic Classification (See map of nine climatic zones):

___ dry freeze ___ wet-dry freeze ___ wet freeze

___ dry freeze-thaw ___ wet-dry freeze-thaw ___ wet freeze-thaw

___ dry nonfreeze ___ wet-dry nonfreeze ___ wet nonfreeze

Potential Site B

Site Location:

Highway designation (example: I74) ____________

Nearby city ____________________________________

Bound direction___________

Start mile post _________ End mile post ___________

Start station No. ________ End station No. ___________

Project length (mile) ____________

Traffic:

Current ADT __________ truck % ___________

18-kip ESALs per year __________________

Project Type:

New alignment or reconstruction of existing pavement ______

No. of traffic lanes (one direction) ____________

Urban/Rural ___________

Climatic Classification (See map of nine climatic zones):

___ dry freeze ___ wet-dry freeze ___ wet freeze

___ dry freeze-thaw ___ wet-dry freeze-thaw ___ wet freeze-thaw

___ dry nonfreeze ___ wet-dry nonfreeze ___ wet nonfreeze

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RESEARCH SECTION

1. This experimental project will test new unique jointed concrete pavement designs. The experiment consists of the construction of several sections having unique designs along a section of in service highway or along a new alignment. It is planned that one such experimental project will be constructed in each of nine climatic zones across the United States so that the suitability of the designs in different climates can be determined.
2. The designs of the individual sections were based on providing adequate transverse joint load transfer, positive subdrainage and a reduction in critical edge stresses and deflections. Rigid pavements having one or more of these design features have been constructed before, either in the U.S. or abroad and have shown improved performance.
3. The performance monitoring task for this experimental project shall be similar to the "Strategic Highway Research Program (SHRP) for Long-Term Pavement Performance (LTPP)" studies.
4. A performance monitoring plan shall be prepared after the application is accepted. The data to be collected in the SHRP-LTPP has been categorized as: 1) inventory, 2) monitouring, 3) traffic, 4) environmental, 5) maintenance and 6) rehabilitation. The participating agency shall identify a minimum data set required required for this experimental project. As a minimum requirement, a performance monitoring plan shall contain the components as follows:
   1) monitoring data categories (roughness, skid, distress, deflection, traffic, environmental),
   2) data measuring devices and technology availability,
   3) monitoring task personnel and source of technical assistance,
   4) data collection procedure, schedule and forms, and
   5) data storage.
5. The participating agency shall install necessary data measuring devices in the experimental project site during construction.
6. The performance monitoring task personnel shall perform the approved performance monitoring plan after the experimental project is constructed.

QUESTIONS/COMMENTS BY APPLICANT

____

____

____

____

____

Name and Title of Applicant: ______________________________

Signature: ____________________ Date: ______________

APPROVAL/COMMENTS

ADMINISTRATIVE SECTION:

TECHNICAL SECTION:

RESEARCH SECTION:

Name of the Project Monitor: ______________________________

Signature: ______________________ Date: ________________

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APPENDIX E

INPUT GUIDE FOR "PFAULT" PROGRAM

I PFAULT Program

PFAULT computes the **mean** faulting for transverse joints for jointed concrete pavements, both with and without dowels. Even though the models include some mechanistic terms, the user should not select inputs that are out of the range of the original data used to develop the predictive models, or poor predictions may result.

The PFAULT program is interactive and menu driven. To use this program, place the disk into the disk drive and type PFAULT <ENTER>. The screen will display the MAIN MENU; five options are available:

1. Display instructions
2. Create a new data file
3. Modify an existing data file
4. Run faulting program
5. Exit - return to DOS

A brief introduction and instruction for the PFAULT program is displayed when the user selects Option 1.

To prepare an input data file, the user has two alternatives:

1) Select Option 2, to create a new data file; or
2) Select Option 3, to modify an existing data file.

The interactive data file creating or modifying process (Options 2 or 3) also includes checks on the data to determine if the values entered are within a practical range. The data file created or modified under the interactive data file creating process is NOT saved after leaving these Options. However, the user can go back to modify the data file as many times as he wishes, provided he does not exit from the program. Since the PFAULT program is run in batch process, the user is required to save the data file as a batch input file before he leaves Option 2 (or Option 3).

To run the program, the user must select Option 4 in the main menu. The screen will ask the user to enter the input file name and the output file name to which the output is to be written. After both file names are entered, the program output is immediately displayed on the screen and also written to the output file name which was specified by the user.

II Program Input

The PFAULT program contains the following direct input variables:

Variable	Description
PTYP	Load transfer type, 1 for doweled and 2 for undoweled jointed concrete pavements
ESAL	Total number of 18-kip equivalent single-axle loads in design lane, million

THK	PCC slab thickness, inches
SPJNT	Transverse joint spacing, feet
KVAL	Effective modulus of subgrade reaction at the top of base/subbase, pci
BASE	Base type, 0 for granular, 1 for asphalt treated, 2 for cement treated without granular subbase, 3 for cement treated with granular subbase, and 4 for lean concrete base
CPOIS	Poisson's ratio of PCC
ECON	Modulus of elasticity of PCC, psi
TEMP	Average annual temperature range (average maximum air temperature in July minus average minimum air temperature in January), ^{o}C
ALPHA	Coefficient of expansion of PCC, $/^{o}C$
DMOD	Modulus of dowel support, pci
ESTL	Modulus of elasticity of dowel bar, psi
SUBSOLC	AASHTO subgrade soil classification, 0 for A-4 to A-7 (fine-grained soils) and 1 for A-1 to A-3 (coarse-grained soils)
EDSUP	Shoulder type, 0 for asphalt concrete and 1 for tied PCC shoulders
FI	Corps of Engineers' mean air freezing index, degree-days below 32 ^{o}F
SUBD	Subdrainage, 0 for no longitudinal drain pipes and 1 for with longitudinal drain pipes

A map of the distribution of mean freezing index values in the U.S is provided in figure 78. Other variables calculated by the program are as follows:

Variable	Description
JWID	Mean transverse joint opening computed using the equation presented on p.255, inches
DEFL	Corner deflection computed using the Westergaard corner equation, inches (see p.266)
ERODF	Erodability factor, 0.5 for the lean concrete base, 1.0 for the cement treated base with granular subbase, 1.5 for the cement treated base without granular subbase, 2.0 for the asphalt treated base, and 2.5 for the granular base

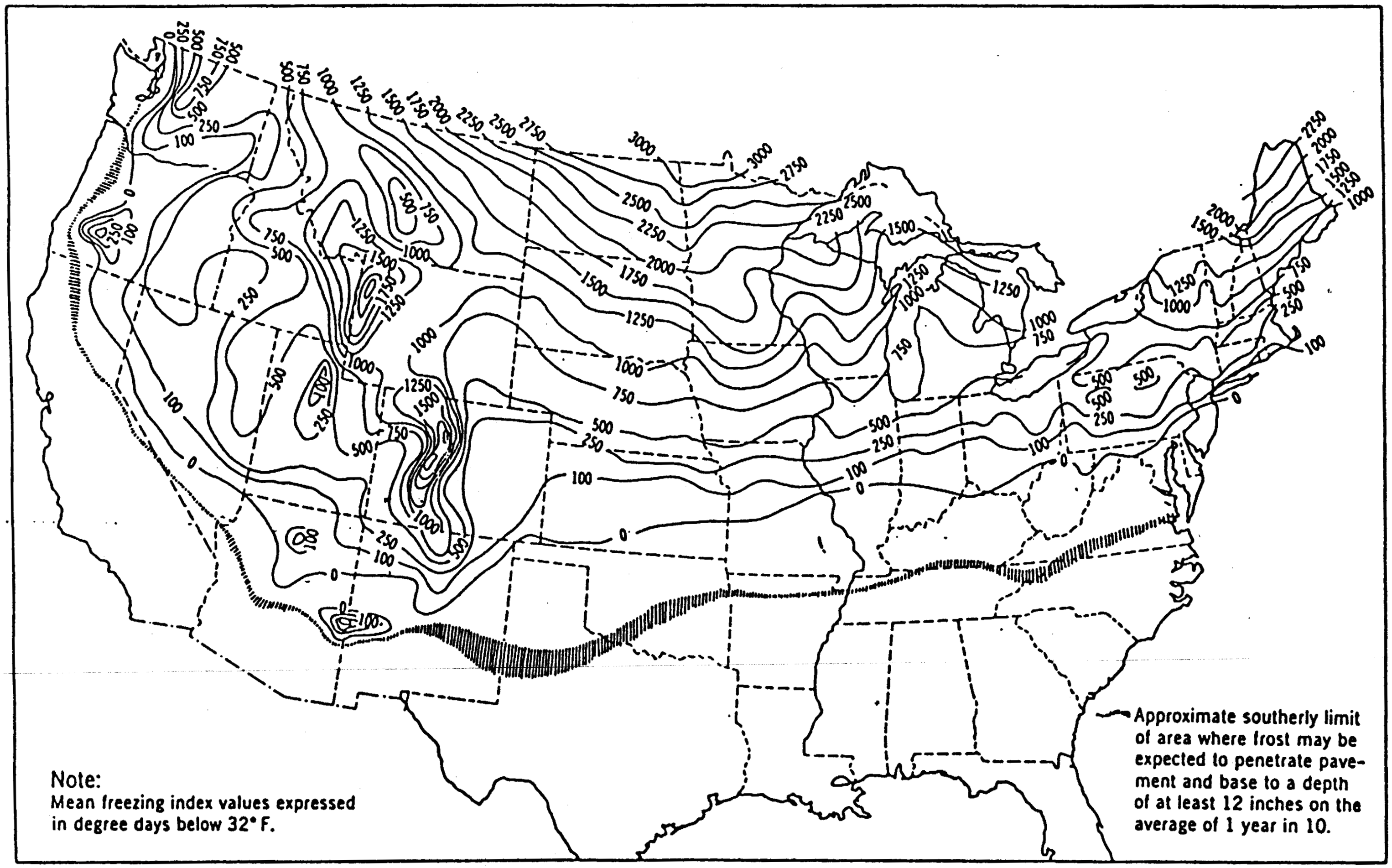

Figure 78. Distribution of mean freezing-index values in continental United States.
(From Corps of Engineers EM 1110-345-306).

SRAD Radius of relative stiffness, in

BETA Relative stiffness of dowel encased in concrete, /in

IDOW Moment of inertia of dowel, in^4

BMAX(J) The maximum concrete bearing stress, psi (see p. 244)

FAULT Predicted mean joint faulting, in

The PFAULT program responses interactively on the screen to the user's input. The program will give an error message "Input is out-of-range" if the input value for certain variable is out of the practical range. The allowable range of the value will be given on the screen. At the same time, the F1 function key can be used to obtain help information for certain inputs.

III Program Output

For doweled pavements, the program output gives a list of all input variables and their values, dowel properties and predicted joint faulting corresponding to each trial dowel diameter. The program considers four dowel diameters, 1.00, 1.25, 1.50 and 1.75 in. For undoweled pavements, the program gives a list of all input variables and the mean predicted joint faulting.

IV Design Examples

The following two design examples illustrate the inputs and outputs from PFAULT program for both doweled and undoweled jointed pavements.

Design Example 1:

A jointed reinforced concrete pavement located in a wet-freeze climate will be designed with round precoated solid steel dowels spaced at 12 inches. The design factors and climatic variables are given as follows:

Design traffic = 10 million 18-kip ESAL, outer lane
Slab thickness = 9 in
Transverse joint spacing = 40 ft

Base type = granular
Effective k-value = 200 pci (top of base)
Poisson's ratio of PCC = 0.15

Modulus of elasticity of PCC = 4.0×10^6 psi
Average annual temperature range = 40 °C
Coefficient of expansion of PCC = 1.0×10^{-5} /°C

Modulus of Elasticity of dowel = 2.9×10^7 psi
Modulus of dowel support = 1.5×10^6 pci

Figure 79 shows the input screens from the PFAULT. Figure 80 shows the list of variables and results from PFAULT. In figure 80, each direct input variable to the PFAULT follows an asterisk (*). The dowel properties and the predicted mean joint faulting corresponding to each trial dowel diameter is given.

The user is able to adjust the design factors or select a dowel diameter to achieve an acceptable faulting level for design.

Design Example 2:

A jointed plain concrete pavement is located in a dry-nonfreeze area. It is desired to estimate the mean faulting of the transverse joints without dowels over the design period. The design factors and climatic variables are given as follows:

Design traffic = 20 million 18-kip ESAL (design lane)
Slab thickness = 9 in
Transverse joint spacing = 15 ft

Base type = asphalt treated
Effective k-value = 300 pci
Poisson's ratio of PCC = 0.15

Modulus of elasticity of PCC = 4.0×10^6 psi
Average annual temperature range = 35 ^oC
Coefficient of expansion of PCC = 1.0×10^{-5} $/^oC$

AASHTO subgrade soil classification = A-7-6 (fine-grained)
Shoulder type = AC shoulder
Freezing Index = 0
Subdrainage = N/A

Figure 81 shows the input screens from PFAULT. The list of variables and the predicted mean joint faulting is given in figure 82. It must be remembered that this is an expected mean faulting.

It is possible for the user to adjust the design factors in an attempt to achieve an acceptable faulting level (e.g. change the base type, or add dowels).

PFAULT	LOAD TRANSFER TYPE AND INPUT VARIABLES	Screen 1

```
Problem Description:>  DESIGN EXAMPLE 1

Load Transfer Type                          :       1.00
   1. Doweled
   2. Undoweled
No. 18-kip ESAL Design Life                 :      10.00 millions
Slab Thickness                              :       9.00 inches
Transverse Joint Spacing                    :      40.00 feet
Effective k-value                           :     200.00 pci
Base Type                                   :        .00
   0. granular         1. asphalt treated
   2. cement treated   3. lean concrete
Poisson Ratio of PCC                        :        .15
Modulus of Elasticity of PCC                :   .4000E+07 psi
Average Annual Temperature Range            :      40.00 °C
   (ave. max. in July - ave. min. in January)
Coefficient of Expansion of PCC             :   .1000E-04 /°C
```

CR: Next field, F1: Help, F2: Jump to end of screen

Figure 79. Input screens from PFAULT for design example 1.

PFAULT	DOWEL VARIABLES	Screen 2

Modulus of Elasticity of Dowel	:	.2900E+08	psi
Modulus of Dowel Support	:	.1500E+07	pci

CR: Next field, F1: Help, F2: Jump to end of screen

Figure 79. Input screens from PFAULT for design example 1 (continued).

P F A U L T

TRANSVERSE JOINT FAULTING ANALYSIS FOR
DOWELED JOINTED REINFORCED OR PLAIN CONCRETE PAVEMENT

DESIGN EXAMPLE 1

```
*18 kip ESAL During Design Life     =     10.00 millions
*Slab Thickness                     =      9.00 inches
*Joint Spacing                      =     40.00 feet
*Effective k-value                  =    200.00 pci
*Base Type                          =       .00
  0.) granular        1.) asphalt treated
  2.) cement treated  3.) lean concrete
*Average Annual Temperature Range   =     40.00 °C
*Coefficient of Expansion (Concrete)=  .10E-04 /°C
 Mean Joint Opening                 =     .1344 inch
*Modulus of Elasticity of PCC       =  .40E+07 psi
*Poisson's Ratio of PCC             =       .15
 Wheel Load                         =      9000 lbs
 Percentage of Load Transferred     =        45 %
*Modulus of Elasticity of Dowel     =  .29E+08 psi
*Modulus of Dowel Support           =  .15E+07 pci
 Dowel Spacing                      =     12.00 inches
 # of Effective Dowels at Wheel A   =    1.9218


DOWEL DIAMETER   = 1.00 in

          Relative Stiffness of Encased Dowel=   .71642
          Dowel Moment of Inertia            =   .04909
          Bearing Stress                     =   3164.9 psi

          PREDICTED MEAN FAULTING            =    .1599 in.


DOWEL DIAMETER   = 1.25 in

          Relative Stiffness of Encased Dowel=   .60601
          Dowel Moment of Inertia            =   .11984
          Bearing Stress                     =   2126.6 psi

          PREDICTED MEAN FAULTING            =    .0911 in.


DOWEL DIAMETER   = 1.50 in

          Relative Stiffness of Encased Dowel=   .52856
          Dowel Moment of Inertia            =   .24851
          Bearing Stress                     =   1537.9 psi

          PREDICTED MEAN FAULTING            =    .0436 in.


DOWEL DIAMETER   = 1.75 in

          Relative Stiffness of Encased Dowel=   .47085
          Dowel Moment of Inertia            =   .46039
          Bearing Stress                     =   1169.9 psi

          PREDICTED MEAN FAULTING            =    .0086 in.
```

Figure 80. Outputs from PFAULT version 1.1 for design example 1.

PFAULT	LOAD TRANSFER TYPE AND INPUT VARIABLES	Screen 1

```
Problem Description:>   DESIGN EXAMPLE 2

Load Transfer Type                        :        2.00
   1. Doweled
   2. Undoweled
No. 18-kip ESAL Design Life               :       20.00 millions
Slab Thickness                            :        9.00 inches
Transverse Joint Spacing                  :       15.00 feet
Effective k-value                         :      300.00 pci
Base Type                                 :        2.00
   0. granular         1. asphalt treated
   2. cement treated   3. lean concrete
Poisson Ratio of PCC                      :         .15
Modulus of Elasticity of PCC              :   .4000E+07 psi
Average Annual Temperature Range          :       35.00 °C
   (ave. max. in July - ave. min. in January)
Coefficient of Expansion of PCC           :   .1000E-04 /°C
```

CR: Next field, F1: Help, F2: Jump to end of screen

Figure 81. Input screens from PFAULT for design example 2.

PFAULT	VARIABLES FOR UNDOWELED CONCRETE PAVEMENTS	Screen 3

```
AASHTO Subgrade Soil Classification  :   0.
   0. = A4 - A7
   1. = A1 - A3

Shoulder Type                        :   0.
   0. = AC
   1. = Tied PCC

Freezing Index (Degree Days)         : 500.00

Subdrainage                          :   0.
   0. = No
   1. = Yes
```

CR: Next field, F1: Help, F2: Jump to end of screen

Figure 81. Input screens from PFAULT for design example 2 (continued).

P F A U L T

TRANSVERSE JOINT FAULTING ANALYSIS FOR
UNDOWELED JOINTED PLAIN CONCRETE PAVEMENT

```
 DESIGN EXAMPLE 2

*18 kip ESAL During Design Life     =    20.00 millions
*Slab Thickness                     =     9.00 inches
*Joint Spacing                      =    15.00 feet
*Effective k-value                  =   300.00 pci
*Base Type                          =     2.00
  0.) granular        1.) asphalt treated
  2.) cement treated  3.) lean concrete
 Erodability Factor                 =     1.00
*Average Annual Temperature Range   =    35.00 °C
*Coefficient of Expansion (Concrete)=  .10E-04 /°C
 Mean Joint Opening                 =    .0380 inch
*Modulus of Elasticity of PCC       =  .40E+07 psi
*Poisson's Ratio of PCC             =      .15
 Wheel Load                         =     9000 lbs
 Percentage of Load Transferred     =        0 %
 Corner Deflection                  =    .0286 inch
*Freezing Index (degree-days)       =      .00
*Subdrainage                        =      .00
      0.) No      1.) Yes
*Subgrade Soil Classification       =      .00
      0.) A4-A7   1.) A1-A3
*Shoulder Type                      =      .00
      0.) AC      1.) Tied PCC


 PREDICTED MEAN FAULTING            =    .0915 in.
```

Figure 82. Outputs from PFAULT for design example 2.

MODIFICATIONS TO THE PFAULT MODELS\PROGRAM FOR VERSION 1.1

The PFAULT program predicts transverse joint faulting for both doweled and nondoweled jointed concrete pavements. Dowels are defined as the conventional solid steel circular load transfer devices.

The original PFAULT 1.0 program includes two predictive mechanistic-empirical faulting models that were originally developed using the NCHRP Project 1-19 COPES database.[42] This data was collected from seven States and generally represents conventional jointed plain and reinforced concrete pavements constructed in the 1960's and 1970's. Additional data was added from California (24 sections), New Jersey (1) and Michigan (1). The models contain several mechanistic variables for the purpose of improving the modeling of joint faulting.

The models were slightly revised to reflect some additional data obtained from 12 experimental sections on I94 near Rothsay, MN. These sections had relatively long 27 ft. joint spacings over granular, asphalt, and cement treated bases, both with and without dowels. The original of these sections due to the fact that the original database did not contain nondoweled, long jointed pavements in a cold climate. The new model predicts the actual faulting of these pavements more accurately.

Figures 83 and 84 show graphically the accuracy in prediction of the revised models over the data base from which they were derived.

The predictive model for doweled transverse joints is as follows:

$$\text{PFAULT} = \text{ESAL}^{0.5377}[2.2073 + 002171\ \text{BSTRESS}^{0.4918} + 0.0003292\ \text{JSPACE}^{1.0793} - 2.1397\ \text{KVALUE}^{0.01305}]$$

The predictive model for undoweled transverse joints is as follows:

$$\text{PFAULT} = \text{ESAL}^{0.3157}[0.4531 + 0.3367\ \text{OPENING}^{0.3322} - 0.5376\ (100\ \text{DEFL})^{-0.008437} + 0.0009092\ \text{FI}^{0.5998} + 0.004654\ \text{ERODAF} - 0.03608\ \text{EDGESUP} - 0.01087\ \text{SOILCRS} - 0.0099467\ \text{DRAIN}\ ]$$

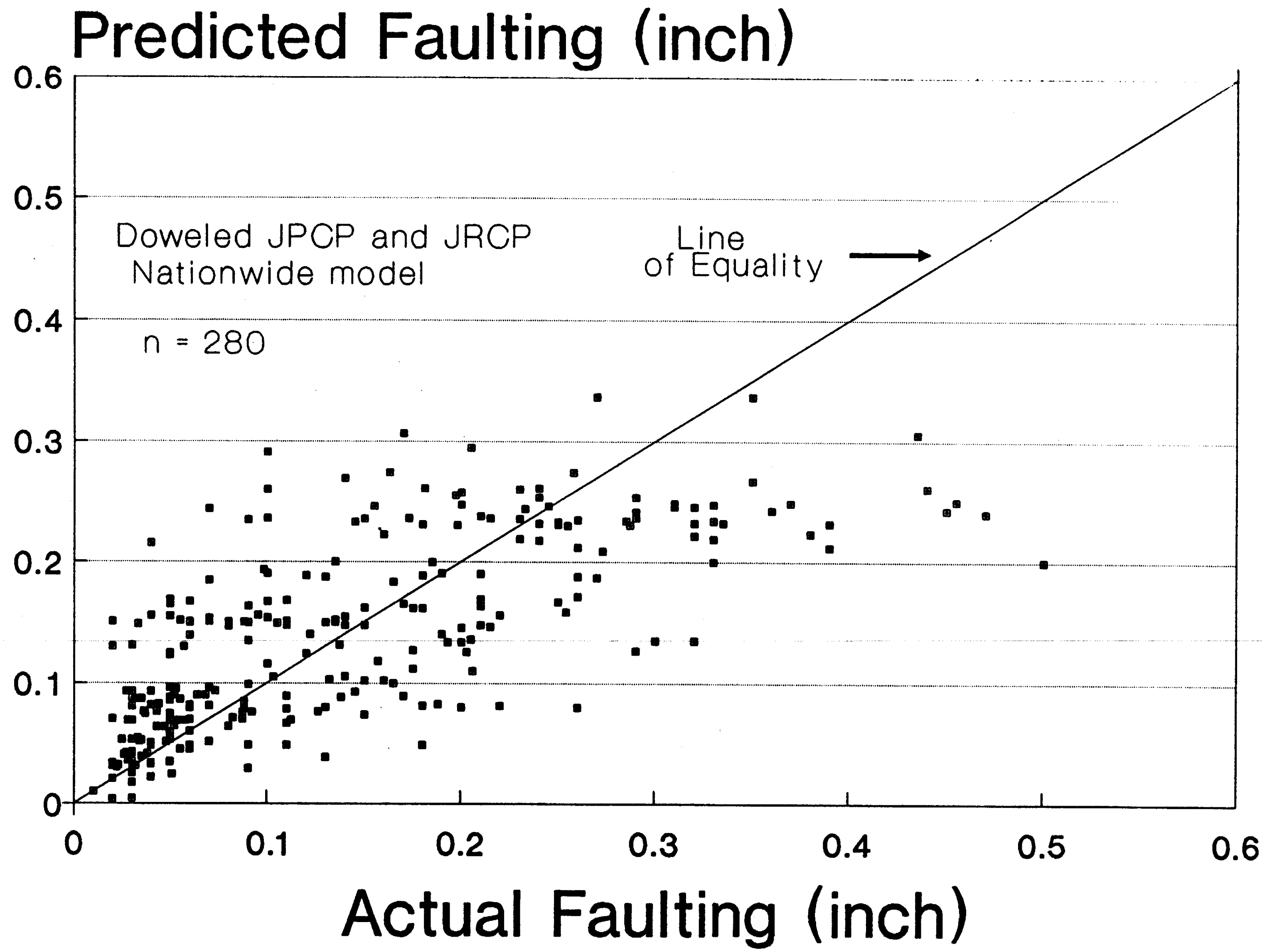

Figure 83. Predicted vs. actual joint faulting for doweled joints.

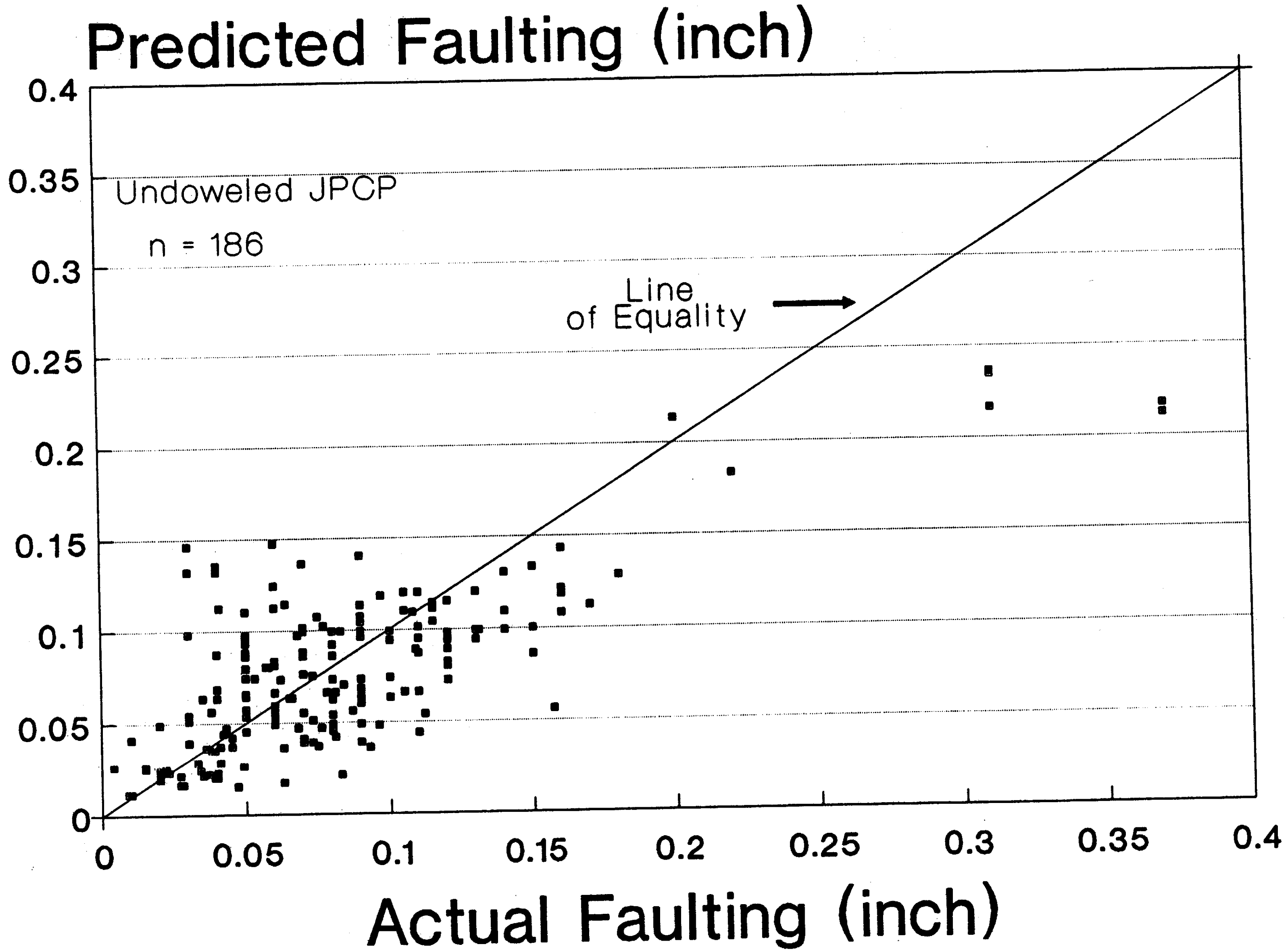

Figure 84. Predicted vs. actual joint faulting for undoweled joints.

Where:

PFAULT = Mean faulting of transverse joints, in (measured 1 ft from slab edge)

ESAL = Accumulated equivalent 18-kip single-axle loads in traffic lane, millions

BSTRESS = Dowel/concrete bearing stress calculated using Friberg's procedure with effective length of 1.0 l insteda of 1.8 l (where l is the radius of relative stiffness), psi

JSPACE = Transverse joint spacing, ft

KVALUE = Effective k-value on top of the base layer, psi/in

OPENING = Calculated joint opening for input temperature range, in

= CON JSPACE*12 [a TRANGE + e]

CON = adjustment factor ue to subbase/slab frictional restraint (0.65 for stabilized base, 0.80 for granular base)

a = Thermal coefficient of contraction of PCC, per degree C

TRANGE = Temperature range (maximum mean daily air temperature in July minus minimum mean daily air temperature in January), degrees C

e = drying shrinkage coefficient of PCC (0.5 - 2.5 x 10^{-4} strain)

DEFL = Corner deflection computed from Westergaard's equation, in (unprotected)

FI = Freezing Index, degree days below freezing

ERODAF = Erodibility factor for base materials,
= 0.5 if lean concrete base
= 1.0 if cement treated base with granular subbase
= 1.5 if cement treated base without granular subbase
= 2.0 if asphalt treated base
= 2.5 if granular base

EDGESUP = 0, if no tied concrete shoulder exists
1, if tied concrete shoulder exists

SOILCRS = AASHO subgrade soil classification
= 0, if A-4 to A-7
= 1, if A-1 to A-3

DRAIN = 0, if no longitudinal edge subdrains exist
= 1, if longitudinal edge subdrains exist

STATISTICS	DOWELED	UNDOWELED
R^2	0.53	0.55
Standard error	0.05	0.03 in
No. Sections	280	186

Ranges of the input variables that were used to develop the models are as follows:

VARIABLE	DOWELED	UNDOWELED
FAULT, in	0.01-0.50	0.00 - 0.37
ESAL, millions	0.6-15.7	0.3-35.9
TRANGE, Deg. C	26-46	17-46
FI, Deg. Days	0-2250	0-2250
SLAB THICK, in	8-12.5	8-13
DOWEL DIA, in	0.0-1.625	---
JSPACE, ft	15-100	7.8-30
KVALUE, psi/in	70-800	115-900
BASE TYPE	Gran	Gran
Dense Graded only)	Cement Trt.	Cement Trt.
	Asph. Trt.	Asph. Trt.
EDGESUP	AC and PCC	AC and PCC
SOILCRS	Fine and Coarse	Fine and Coarse
DRAIN	No and Yes	No and Yes

To further show the limitations of the database, tables 81 and 82 show a summary of the number of sections that fall into several variable combinations for doweled and undoweled joints. For example, there were a lot of sections (e.g., 65) having undoweled joints in cold climates (with TRANGE > 32), with a treated (asphalt or cement) base, with joint spacing less than 18 ft and a slab less than 10 in thick. There were no sections for the same conditions if the base was nontreated granular.

Even though the models contain several mechanistic variables, they are still heavily empirical and should not be used much beyond the ranges and the combinations of inputs from which they were developed. In particular, for example, the program cannot be used to predict faulting for pavements having an open-graded drainage layer directly beneath the slab since there were no permeable bases in the database. This drainage layer will completely change the state of pumping and erosion beneath the slab and thus faulting may be much different.

Table 81. Number of sections that fall into various combinations of variable ranges for undoweled joints.

		SLAB THICKNESS < 10 IN		SLAB THICKNESS >= 10 IN	
		JT SPACE < 18 FT	JT SPACE >= 18 FT	JT SPACE < 18 FT	JT SPACE >= 18 FT
UNTREATED BASE (GRANULAR)	TRANGE <=32 Deg.C	---	2	---	3
	TRANGE > 32 Deg.C	---	4	1	1
TREATED BASE (CAM, BAM)	TRANGE <=32 Deg.C	55	18	9	17
	TRANGE > 32 Deg.C	65	7	4	---

Note: TRANGE <= 32 Deg.C in California, Georgia, and Louisiana

Note: TRANGE > 32 Deg.C in Minnesota, Illinois, California Nebraska, and Utah

Table 82. Number of sections that fall into various combinations of variable ranges for doweled joints.

	SLAB THICKNESS < 10 IN		SLAB THICKNESS >= 10 IN	
	JT SPACE < 45 FT	JT SPACE >= 45 FT	JT SPACE < 45 FT	JT SPACE >= 45 FT
TRANGE <= 32 Deg.C	2	---	---	20
TRANGE > 32 Deg.C	75	8	29	146

Note: TRANGE <= 32 Deg.C in California, Georgia, and Louisiana

Note: TRANGE > 32 Deg.C in Minnesota, Illinois, California Nebraska, and Utah

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*U. S. GOVERNMENT PRINTING OFFICE:1989-617-000:03693