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16. Abstract

The primary factors to consider in the thickness and reinforcement design for continuously reinforced concrete pavements are the structural response variables; crack spacing, crack width, and maximum steel stress. They plant an important role in the outcome of the pavement performance, and can be related to the major distresses common to CRC pavements.

In the previous report, 177-16, the relationships between the significant input variables and the structural responses predicted by the CRCP-2 computer model were quantified using regression techniques and expressed as a set of nomographs. This set of design chargs enalbes us to graphically predict the final crack spacing, crack width, and steel stress in CRC pavement.

This report determines the design limiting criteria for these structural responses. Previous investigations of the design criteria were reviewed and the most recently developed analytical models studied. The basic procedures used to establish the design criteria included examination of the major distresses such as punchout, spalling, and steel rupture and study of correlations between these distresses and the corresponding structural responses at appropriate levels. The procedure for use of these limiting cirteria in CRCP design is outlined along with a series of guidelines for the selection of CRCP design input values.

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LIMITING CRITERIA FOR THE DESIGN OF CRCP

by

B. Frank McCullough, J. C. M. Ma, and C. S. Noble

Research Report Number 177-17

Development and Implementation of the Design, Construction
and Rehabilitation of Rigid Pavements

Research Project 3-8-75-177

conducted for

Texas
State Department of Highways and Public Transportation

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

by the

CENTER FOR HIGHWAY RESEARCH

THE UNIVERSITY OF TEXAS AT AUSTIN

August 1979

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

PREFACE

This report establishes the design criteria for continuously reinforced concrete pavements. It also presents recommendations and guidelines for the design of continusously reinforced concrete pavement.

The results from this study are presented as a subsequent report following the report, "Nomographs for the Design of Continuously Reinforced Concrete Pavements."

Our thanks are extended to the entire staff of the Center for Highway Research, who provided support during the preparation of this report. Particular thanks are due to Mrs. Marie Fisher, Miss Betty Delgado, Miss Kimberly Hill and Miss Julie Muckleroy for typing and drafting of the manuscript, and Mr. Arthur Frakes for his great efforts in editing and coordinating the report preparation.

B. Frank McCullough
James C. M. Ma
Christopher S. Noble

August 1979

LIST OF REPORTS

Report No. 177-1, "Drying Shrinkage and Temperature Drop Stresses in Jointed Reinforced Concrete Pavement," by Felipe R. Vallejo, B. Frank McCullough, and W. Ronald Hudson, describes the development of a computerized system capable of analysis and design of a concrete pavement slab for drying shrinkage and temperature drop. August 1975.

Report No. 177-2, "A Sensitivity Analysis of Continuously Reinforced Concrete Pavement Model CRCP-1 for Highways," by Chypin Chiang, B. Frank McCullough, and W. Ronald Hudson, describes the overall importance of this model, the relative importance of the input variables of the model and recommendations for efficient use of the computer program. August 1975.

Report No. 177-3, "A Study of the Performance of the Mays Ride Meter," by Yi Chin Hu, Hugh J. Williamson, B. Frank McCullough, and W. Ronald Hudson, discusses the accuracy of measurements made by the Mays Ride Meter and their relationship to roughness measurements made with the Surface Dynamics Profilometer. January 1977.

Report No. 177-4, "Laboratory Study of the Effect of Non-Uniform Foundation Support on CRC Pavements," by Enrique Jiminez, B. Frank McCullough, and W. Ronald Hudson, describes the laboratory tests of CRC slab models with voids beneath them. Deflection, crack width, load transfer, spalling, and cracking are considered. Also used is the SLAB 49 computer program that models the CRC laboratory slab as a theoretical approach. The physical laboratory results and the theoretical solutions are compared and analyzed, and the accuracy is determined. August 1977.

Report No. 177-6, "Sixteenth Year Progress Report on Experimental Continuously Reinforced Concrete Pavement in Walker County," by Thomas P. Chesney, and B. Frank McCullough, presents a summary of data collection and analysis over a 16-year period. During that period, numerous findings resulted in changes in specifications and design standards. These data will be valuable for shaping guidelines and for future construction. April 1976.

Report No. 177-7, "Continuously Reinforced Concrete Pavement: Structural Performance and Design/Construction Variables," by Pieter J. Strauss, B. Frank McCullough, and W. Ronald Hudson, describes a detailed analysis of design, construction, and environmental variables that may have an effect on the structural performance of a CRCP. May 1977.

Report No. 177-9, "CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavements," by James Ma and B. Frank McCullough, describes the modification of a computerized system capable of analysis of a continuously reinforced concrete pavement based on drying shrinkage and temperature drop. August 1977.

Report No. 177-10, "Development of Photographic Techniques for Performance Condition Surveys," by Pieter J. Strauss, James Long, and B. Frank McCullough, discusses the development of a technique for surveying heavily trafficked highways without interrupting the flow of traffic. May 1977.

Report No. 177-11, "A Sensitivity Analysis of Rigid Pavement-Overlay Design Procedure," by B. C. Nayak, B. Frank McCullough, and W. Ronald Hudson, gives a sensitivity analysis of input variables of Federal Highway Administration computer-based overlay design procedure RPOD1. June 1977.

Report No. 177-12, "A Study of CRCP Performance: New Construction versus Overlay," by James I. Daniel, B. Frank McCullough, and W. Ronald Hudson, documents the performance of several continuously reinforced concrete pavements (CRCP) in Texas. April 1978.

Report No. 177-13, "A Rigid Pavement Overlay Design Procedure for Texas SDHPT," by Otto Schnitter, B. Frank McCullough, and W. Ronald Hudson, describes a procedure recommended for use by the Texas SDHPT for designing both rigid and flexible overlays on existing rigid pavements. The procedure incorporates the results of condition surveys to predict the existing pavement remaining life, field and lab testing to determine material properties, and elastic layer theory to predict the critical stresses in the pavement structure. May 1978.

Report No. 177-14, "A Methodology to Determine an Optimum Time to Overlay," by James I. Daniel, B. Frank McCullough, and W. Ronald Hudson, describes the development of a mathematical model for predicting the optimum time to overlay an existing rigid pavement.

Report No. 177-15, "Precast Repair of Continuously Reinforced Concrete Pavement," by Gary E. Elkins, B. Frank McCullough, and W. Ronald Hudson, describes an investigation into the applicability of using precast slabs to repair CRCP, presents alternate repair strategies, and makes new recommendations on installation and field testing procedures. May 1979.

Report No. 177-16, "Nomographs for the Design of CRCP Steel Reinforcement," by C. S. Noble, B. F.McCullough, and J. C. M. Ma, presents the results of an analytical study undertaken to develop regression equations and nomographs for use as a supplementary tool in the design of steel reinforcement in continuously reinforced concrete pavement by the Texas State Department of Highways and Public Transportation. August 1979.

Report No. 177-17, "Limiting Criteria for the Design of CRCP," by B. Frank McCullough, J. C. M. Ma, and C. S. Noble, presents a set of criteria which limits values of a set of variables to be used in the design of CRCP. These criteria are to be used in conjunction with Report No. 177-16. August 1979.

ABSTRACT

The primary factors to consider in the thickness and reinforcement design for continuously reinforced concrete pavements are the structural response variables; crack spacing, crack width, and maximum steel stress. They play an important role in the outcome of the pavement performance, and can be related to the major distresses common to CRC pavements.

In the previous report, 177-16, the relationships between the significant input variables and the structural responses predicted by the CRCP-2 computer model are quantified using regression techniques and expressed as a set of nomographs. This set of design charts enables us to graphically predict the final crack spacing, crack width, and steel stress in CRC pavement.

This report determines the design limiting criteria for these structural responses. Previous investigations of the design criteria were reveiwed and the most recently developed analytical models studied. The basic procedures used to establish the design criteria included examination of the major distresses such as punchout, spalling, and steel rupture and study of correlations between these distresses and the corresponding structural responses at appropriate levels.

Finally, the procedure for use of these limiting criteria in CRCP design is outlined, along with a series of guidelines for the selection of CRCP input values.

KEY WORDS: continuously reinforced concrete pavement, crack spacing, crack width, steel stress, distress, CRCP-2, design criteria, structural response, punchout, spalling, steel rupture.

SUMMARY

Limiting criteria on the values of those factors affecting the behavior of CRCP have been developed in this study. These limiting values on the variables, crack spacing, crack width and steel stress are to be used in conjunction with Research Report No. 177-16, for the design of steel reinforcement in CRCP. First, the mechanistic behavior of CRCP is discussed from the point of view of distress types and structural responses to load. The various limiting criteria are established for the control of each of the variables crack spacing, crack width and steel stress. Finally, the method for use of these limiting criteria in CRCP design is outlined, along with a series of guidelines for selection of CRCP design input values.

IMPLEMENTATION STATEMENT

The limiting criteria established in this report, the suggested procedure for their use in CRCP design and the guidelines for selection of CRC design input values can be used in conjunction with the nomographs presented in Research Report No. 177-16 for the design of CRCP. The procedure is comprehensive, is easy to use, and allows the designer to specify a range of values for design parameters corresponding to the uncertainty in the design.

This design procedure should be incorporated into the Texas State Department of Highways and Public Transportation manual for the design of continuously reinforced concrete pavement as soon as possible.

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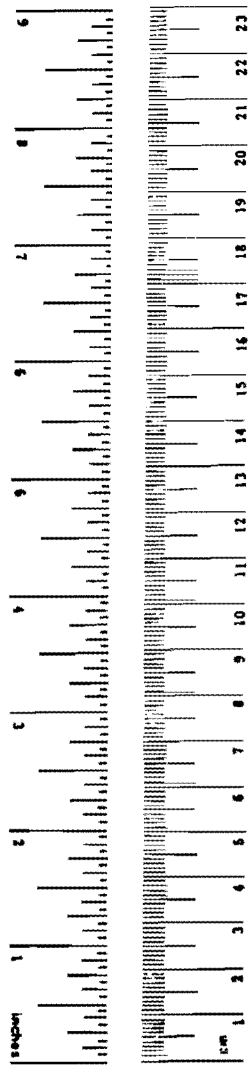
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

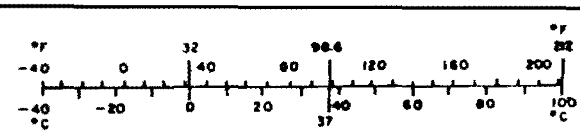
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CHAPTER 1. INTRODUCTION

BACKGROUND

The design concept for continuously reinforced pavement is to force cracks to form at relatively close intervals, thus controlling the tightness of the crack to provide good load transfer and prevent excessive water percolation. The frequency of crack occurrence and the final crack width depend on a complex interaction of environmental variables, material properties and magnitudes of applied loads. Initial cracks in the continuously reinforced concrete pavement are primarily caused by critical stresses induced by initial temperature drop and drying shrinkage of the concrete. Additional cracks may develop during application of external load when the combined stresses of the internal and external forces exceed the concrete tensile strength. Close to 90 percent of the transverse cracks occur within one month after construction. The crack pattern will eventually reach a stabilized condition when the pavement has experienced the miniumum temperature during the cold season and when most of the drying shrinkage in the concrete has occured.

The CRCP-2 computer model (Ref 1) was designed to fully simulate the mechanistic behavior of the continuously reinforced concrete pavement with respect to time and loads. The model predicts the structural responses of the CRC pavement to environmental load and static external load from the time of formation of initial cracks to the time when volumetric changes of the CRC pavement have stabilized. The final crack spacing, crack width, and steel stress appear to strongly influence the performance of the CRC pavement since major distresses common to CRC pavement are highly correlated with the above responses.

OBJECTIVES

In Research Report 177-16 (Ref 2), the relationships between the significant input variables and the structural responses predicted by the CRCP-2 model are quantified using regression techniques and expressed as a set of nomographs. This set of design charts allows graphical prediction of the final responses crack spacing, crack width and steel stress, and greatly reduces computation time and effort. The first objective of this report is to study correlations between mechanisms of major distress and structural responses as predicted from the previous report. Design criteria for each of the responses are then established to control and restrain distress which would otherwise adversely affect the performance of the continuous pavement.

SCOPE

Development of nomographs based on CRCP-2 computer models for the prediction of structural responses including crack spacing, crack width and steel stress was described in Research Report 177-16. This report (177-17) will study the design criteria for the predicted responses based on their relationships with major distress types in the continuous pavement. The two reports are intended to be used in conjunction with each other.

Chapter 2 describes the predominant distresses and some basic concepts involving continuously reinforced concrete pavement as well as the mechanistic behavior and the model simulating the structural responses in the pavement. Chapters 3, 4, and 5, establish the design criteria for crack spacing, crack width, and steel stress. Chapter 6 summarizes the criteria developed earlier, describes the procedure for their use in design and outlines a set of guidelines for the choice of values for the design parameters.

CHAPTER 2. MECHANISTIC BEHAVIOR OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

MAJOR DISTRESS TYPES ASSOCIATED WITH CRCP

A considerable amount of information concerning major distress in CRC pavements can be found in studies conducted by Darter and Barenberg (Ref 3) and McCullough et al (Ref 4). Table 2.1 prepared by Darter and Barenberg gives a list of the predominant distress types found in CRC pavement. The frequency of occurrence and the severity of the distress types were summarized. The information presented was obtained from pavements which had survived for 20 years. Results from statewide condition surveys along with the collected experience of prominent researchers were used in establishing the significant distress types and rank order. Table 2.2 shows the resulting priority ranking of distress types for the continuous pavement in decreasing order of the significance of their effect on pavement performance. Fatigue cracking, low temperature cracking, and shrinkage cracking are secondary distress types which define the spacing of transverse cracks in the continuous pavement. Secondary distresses are responsible for the development of the primary distress which leads to reduction of serviceability in the pavement. Punchout, for instance, is a primary distress type which occurs between closely spaced transverse cracks that are subsequently connected by longitudinal cracks.

CRITERIA FOR STRUCTURAL RESPONSES ASSOCIATED WITH CRCP

The primary factors to consider in the design of continuously reinforced concrete pavements are the structural responses, crack spacing, crack width, maximum steel stress, and maximum concrete stress. They play an important role in the outcome of the pavement performance and can be related to the major distresses discussed previously. These factors are also interrelated with each other. A design which forces cracks to form in either a narrow or wide spacing will affect the accumulated drag forces due to frictional restraint

TABLE 2.1. SUMMARY OF THE PREDOMINANT DISTRESS TYPES FOUND IN CRCP

		Projects	
	Distress Type* (Ref 3)	Distressed/Total	Maintained**/Distressed
1.	Surface depression	7/12	0/7
2.	Crack spalling	6/12	2/6
3.	Punchout	4/12	4/4
4.	Interconnecting cracks	4/12	2/4
5.	Longitudinal cracking	2/12	0/2
5.	Steel rupture	2/12	2/2

*Maintenance applied only to specific distress indicated.
**Rated moderate to severe.

TABLE 2.2 PRIORITY RANKING OF SIGNIFICANT DISTRESS WITH CRCP

Priority Ranking	Major Distress Type
1	Punchouts
2	Crack spacing
3	Fatigue cracking
4	Low-temperature cracking
5	Shrinkage cracking
6	Steel rupture*

* Does not usually occur in the southern United States.

from the subbase and subsequently alter the level of the responses crack width, maximum steel stress, and maximum concrete stress.

MODEL DESCRIPTION

The computer program CRCP-2 (Ref 1) models the one-dimensional changes in concrete stress, steel stress, crack width, and crack spacing occurring in a continuously reinforced concrete pavement caused by drying shrinkage of the concrete, temperature variation and wheel loads. Due to the accumulated friction and the terminal treatments used in the construction, the slab model assumes an anchorage at each end so that the pavement within the anchorages will maintain a fixed length.

The difference in the thermal coefficients of the steel and the concrete together with the drying shrinkage of the concrete enable us to determine the internal stress in the reinforced slab. Using the friction-movement characteristic of the slab and the soil as determined in laboratory experiments, the degree of restraint due to the soil frictional resistance can be estimated (Ref 1). By assuming equilibrium in the system, the stress of one material can be computed in terms of the stress of the adjacent materials. Finally, an incremental approach can be adopted to predict the formation of transverse cracks as a function of time by comparing the historical changes of the concrete stress with the strength of the concrete.

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

(1) 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.

(2) The concrete and steel properties are linearly elastic.

(3) In the fully bonded sections of the concrete slab, there is no relative movement between the steel and the concrete.

(4) The force displacement curve which characterizes the frictional resistance between the concrete slab and the underlying base is elastic.

(5) Temperature variations and shrinkage due to drying are uniformly distributed throughout the slab, and hence, a one-dimensional axial structural model is adopted for the analysis of the problem.

(6) Material properties are independent of space.

(7) The effect of the creep of concrete and slab warping are neglected.

STRUCTURAL RESPONSES OF CRCP

The transverse cracking in a continuous pavement is the result of the restraint of the pavement slab induced by internal environmental forces and external wheel load forces. Most transverse cracks occur at an early age of the pavement when most of the moisture evaporation takes place. Additional cracks may later develop if the stress, which has been increased by the wheel load application, exceeds the fatigue strength of the concrete.

Spacing of transverse cracks that occur in continuously reinforced concrete pavements is perhaps the most important variable directly affecting the behavior of the pavement (other important variables affect the crack spacing). Relatively large distances between cracks result in a higher accumulated drag force due to frictional resistance from the subgrade, thus producing high steel stress at the crack and large crack width. Closer crack spacing reduces the frictional restraint, and thus the steel stress and crack width. It is clear that the crack spacing is directly related to other responses such as steel stress and crack width. Control of one will immediately affect the behavior of the others. In general, assuming adequate foundation support, closely spaced cracks in CRC pavement are desirable because the steel stress and the crack width will be small. However, it is commonly known that the major distress observed on in-service CRC pavements is "punchout," which can be associated with the combination of closely spaced transverse and longitudinal cracking. An optimum design, therefore, calls for a balance in all of the structural responses in the continuous pavement.

Failures in CRC pavement are usually manifested as isolated areas of premature distress in different forms (according to environment) such as steel rupture, excessive spalling at the crack, edge pumping, and punchout. Among the distresses, some can be associated with poor subbase and drainage which are outside of the scope of this report, while others can be linked directly with the above pavement responses. As stated earlier, punch-out is associated with transverse crack spacing in the continuous pavement. Narrow crack spacing when combined with crack deterioration will force the beam action of the continuous pavement to act transversely instead of longitudinally. Transverse beam action will in turn cause longitudinal cracks to appear and eventual deterioration into punch-out failure. Punch-out, therefore, can be alleviated by controlling the crack spacing of the continuous pavement assuming adequate foundation support. Similarly, other failures, including spalling and steel rupture, can be controlled by tracing the origin of the distress mechanism, and by assigning design criteria to the corresponding pavement responses.

PREVIOUS DESIGN CRITERIAL

Contemporary procedures for the design of CRCP are summarized in the AASHTO Interim Guide for the Design of Pavement Structures (Ref 5) and in Vol 4 of the Texas SDHPT Operations and Procedures Manual (Ref 6). These procedures are based on early developments in the modelling of CRCP behavior, and as such they restrict steel stress to values below yield. However, they do not consider other variables which have a significant effect on performance such as crack width and spacing. More recent work (Refs 1 and 4) established newer design criteria, (again discussed in these references) for use with the computer program design approach. It is the purpose of this report to outline criteria for use in conjunction with the nomograph (regression equation) design techniques outlined in Ref 2.

CHAPTER 3. DESIGN CRITERIA FOR CRACK SPACING

CRCP can be simulated as a continuous beam resting on an elastic foundation. Transverse cracks develop as the result of frictional restraint of the slab to changes caused by shrinkage and temperature drop. Additional cracks due to bending in the longitudinal direction may develop when traffic loads are applied. As transverse crack spacing becomes relatively narrow and when load transfer at the crack deteriorates, the pavement structure no longer responds as a longitudinal beam, but as a transverse beam with stress in the transverse direction higher than that in the longitudinal direction. With further increase of fatigue loadings, longitudinal cracks crossing the transverse cracks will develop and eventually deteriorate into punchout failure. One critical crack spacing, therefore, is the spacing at which the stress in the transverse direction becomes dominant.

EFFECT OF CRACK SPACING ON TRANSVERSE AND LONGITUDINAL STRESSES

The relationship between crack spacing and stresses in the x-x and y-y directions are illustrated conceptually in Fig 3.1. Solid lines in the figure represent the relationship for the condition of zero load transfer at the crack. For crack spacing greater than B, the pavement slab acts as a longitudinal beam and the stress in the x-x direction is more critical since it becomes larger. Vice versa is true for a crack spacing less than B, since the slab acts as a transverse beam. The spacing between cracks in the continuous slab can be thought of as the span length of a rectangular plate on an elastic foundation. Increase in crack spacing or span length will result in higher σ_x and lower σ_y. The increase in bending stress will gradually diminish as we move further away from the mid-span where the load was applied. The stress in the x-x direction remains constant after reaching the maximum level. The crack spacing, B, at the intersection of the σ_x curve and the σ_y curve is, therefore, the minimum allowable crack spacing for zero load transfer at the crack.

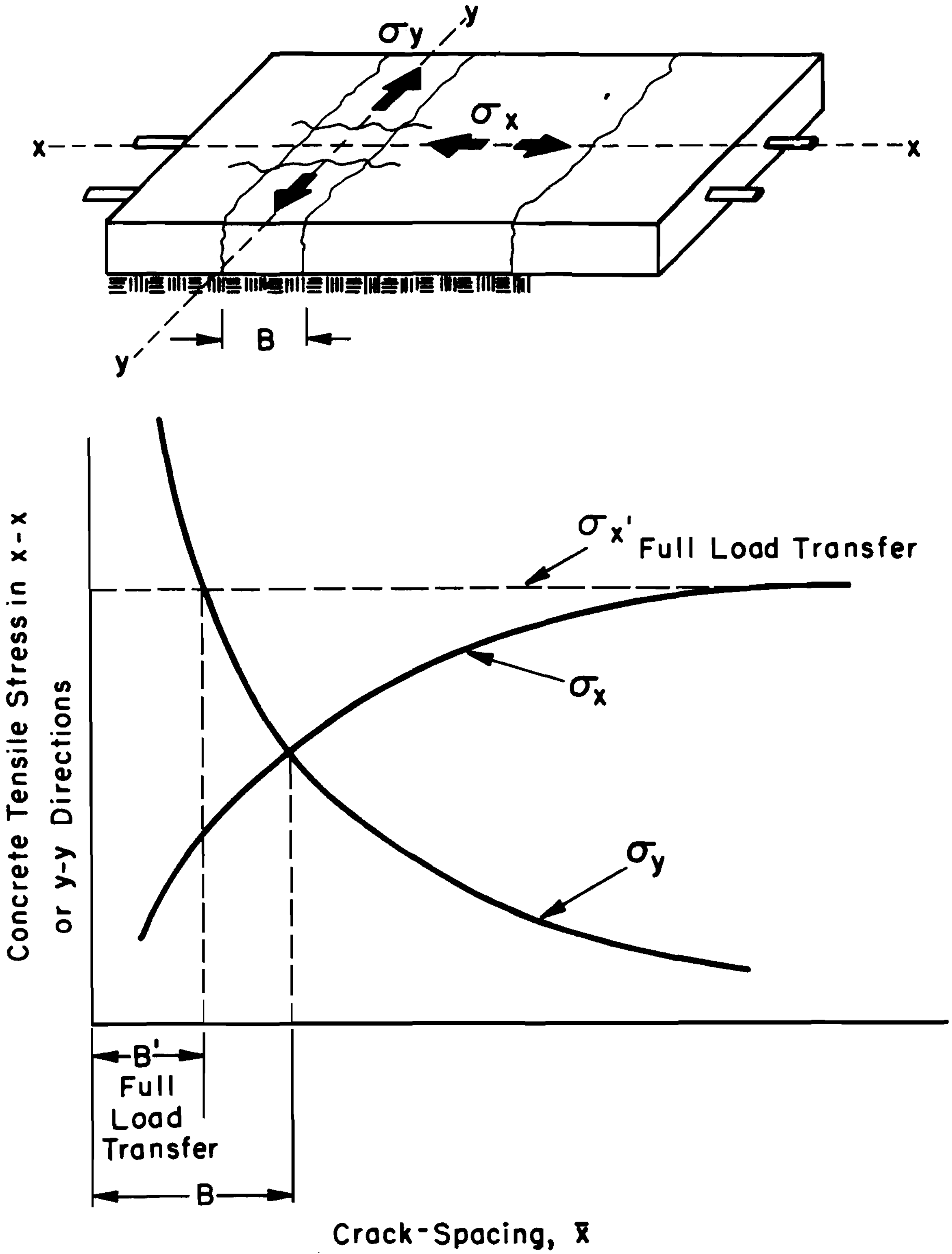


Fig 3.1. Conceptual illustration of the critical stress location as affected by crack spacing (for a given set conditions).

For full load transfer conditions, the pavement can be viewed as a continuous slab with no cracks. The σ_x' at any crack spacing under the full load transfer conditions should, therefore, be equal to the σ_x for an infinitely long slab. The horizontal dashed line in Fig 3.1 represents the stress in the x-x direction, σ_x', for an infinitely long slab or one with full load transfer conditions at the cracks. It is obtained by drawing a line tangent to the point of maximum stress occuring when the slab length no longer influences the stress. The length B' is derived from the intersection of σ_x' line and σ_y curve and it represents the minimum allowable crack width for full load transfer. Thus B' is the minimum crack spacing for full load transfer, and B is the minimum for zero load transfer. In-service CRCP has a condition between these two extremes, which is closer to full transfer after construction and decreases with repeated load application.

EFFECT OF STIFFNESS REDUCTION AT THE CRACK

Load transfer at a crack is possible through moment transfer, granular interlock, and dowel action of the steel reinforcement, assuming adequate foundation support. In field conditions, neither full load transfer nor zero load transfer at the crack are likely to be found. Theoretically, if the granular interlock and dowel action of the reinforcing bars are 100 percent efficient, half the applied load will be transfered across the crack to the adjacent slab. This is true only if the same amount of deflection occurs on both slabs and each assumes half of the applied load. However, considering a certain amount of debonding of the steel and looseness that develops in the aggregates under repeated loads, a further reduction in load transfer of between 5 and 10 percent can be assumed (Ref 7). Thus, the design load transfer due to aggregate interlock and dowel action of the steel should be 45 percent of the design load.

Under vertical load, deflection of the slab at the crack will cause the crack width to decrease. Moment transfer occurs only when the slab segments at both sides of the crack are in contact. The amount of reduction in bending stiffness at the crack depends on a combination of design variables.

Hudson and Abou-Ayyash (Ref 8) studied the effect of transverse cracks on the bending rigidity of continuous pavement. Fig 3.2 shows the result of the investigation in which percentage reduction in bending stiffness at the crack is related to the concrete compressive strength and to the percentage of longitudinal reinforcement, for a given set of environmental conditions.

CRACK EFFECT ON ALLOWABLE CRACK SPACING

Assuming that a linear relationship exists between the structural response of the slab as affected by the load transfer at the crack and the spacing between cracks, allowable crack spacing for cracks with various degrees of load transfer capacities can be predicted. As an example, if cracks in the slab can provide half the structural integrity of the uncracked section, the critical crack spacing will be at the mid-point between B and B' in Fig 3.1. Following is an example problem for the prediction of allowable crack spacing in which

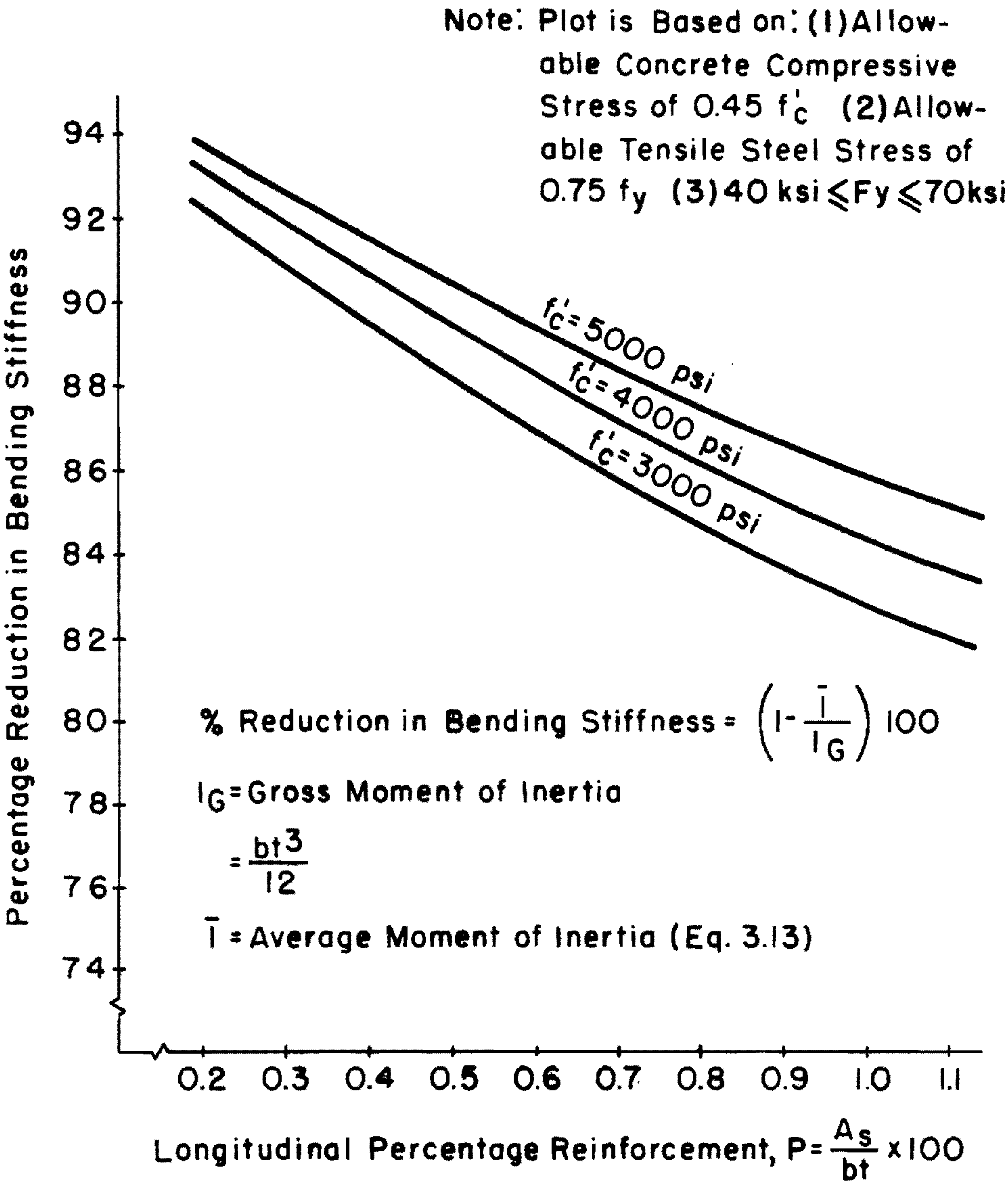
critical crack spacing for full load transfer,	B'	= 3 feet
critical crack spacing for zero load transfer at the crack,	B	= 5 feet
concrete compressive strength	f'_c	= 4000 psi
longitudinal reinforcement	p	= 0.6%

From Fig 3.2, the reduction in bending stiffness, R, at the crack is found to be 88 percent. The reduction of load transfer related to aggregate interlock and dowel action is from 1.00 to 45 percent. Total loss of load transfer can therefore be estimated to $R(1 - .45)$. The allowable crack spacing will be

$$X_{allowable} > B' + .55R(B - B')$$

$$> 3 + .55 \times .88(5 - 3)$$

$$> 4 \text{ feet}$$



1 psi = 6.895 kPa

Fig 3.2. Variation of the percentage reduction in bending stiffness at crack location with longitudinal percentage reinforcement (Ref 9).

PREDICTION OF ALLOWABLE CRACK SPACING RANGE

Minimum Allowable Crack Spacing - Concrete Tensile Stress Criterion. The Slab 49 program (Refs 9 and 10) provides an excellent analysis tool for studying the effect of crack spacing on continuity and thus provides a basis for choosing a minimum crack spacing. The logic and procedures used herein are documented in the Appendix. In order to obtain a limiting (minimum) value for crack spacing, a factorial of values of the design variables as listed in Fig 3.3 was evaluated using the Slab 49 program. The magnitudes of the variables cover a broad range of slab thicknesses, axle loads and crack spacings. Both longitudinal and transverse stresses with respect to crack spacing were computed (Appendix, Tables A3 and A4) and plotted for various axle loads and slab thicknesses (Figs 3.4 and 3.5). The minimum allowable crack spacing, B', is determined at the intersection of the σ_x' line and the σ_y curve. Allowance for reduced bending stiffness is made as indicated above using Fig 3.2.

Minimum Allowable Crack Spacing - Bond Development Length Criterion. As discussed in Ref 1, the required length for full development of the bond between the reinforcing steel and the concrete in CRCP must be kept below a value equal to one-half the crack spacing. This bond development length, however can be calculated in terms of the change in steel stress between the crack location and midspan, as we move longitudingly down the concrete slab:

i.e.,

$$\bar{x} \geq 2b$$

where

$$b = \frac{\Phi^2}{38\sqrt{f'_c}} (\sigma_{sc} - \sigma_{sm}) \qquad \text{(Ref 3)}$$

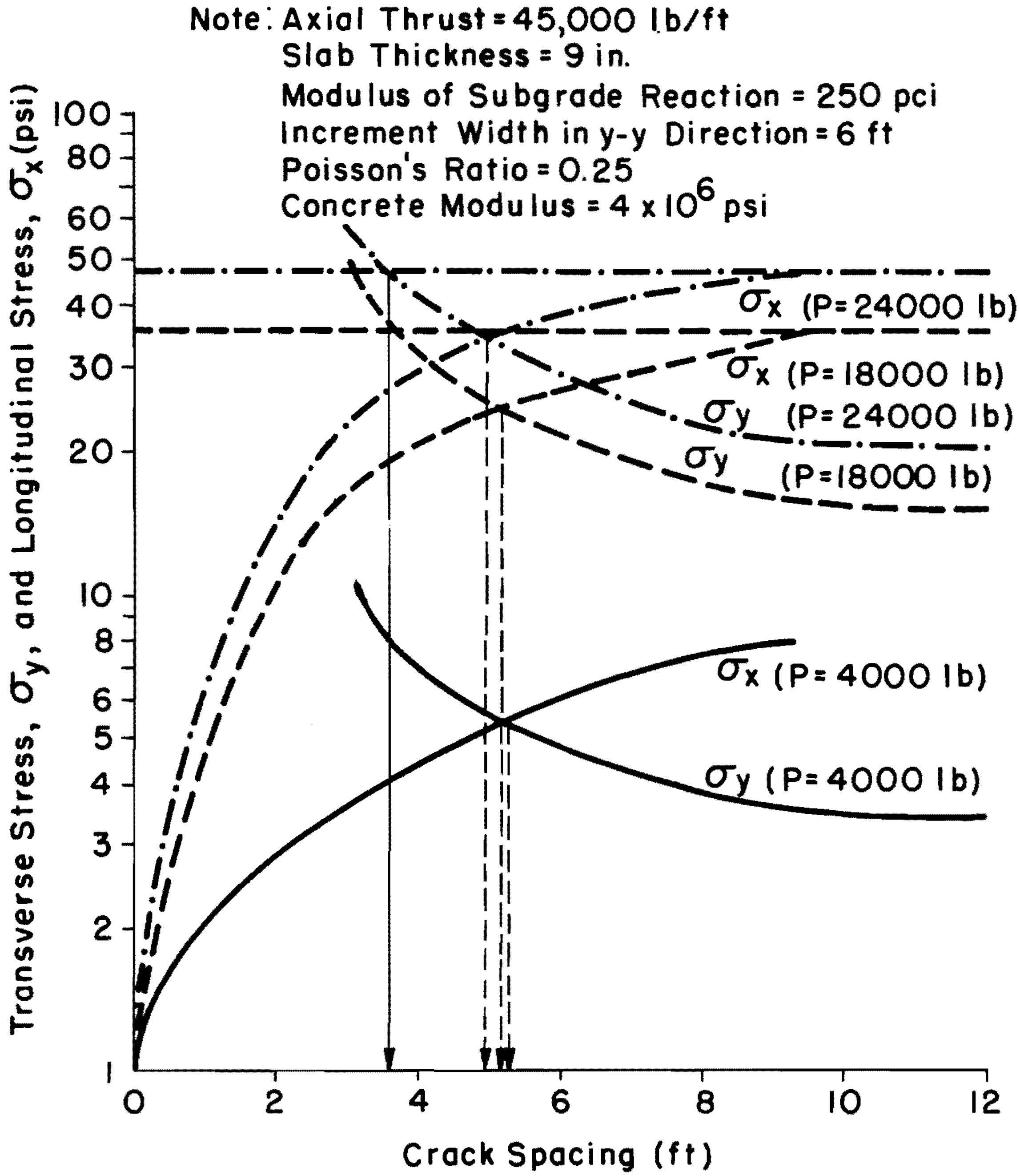
$\bar{x}$ = allowable crack spacing, ft

X Slab Problem Analyzed

Axle Load (lb)	4,000				18,000				24,000			
Crack Spacing (ft) / Slab Thickness (in.)	3	6	9	12	3	6	9	12	3	6	9	12
7					X	X	X					
8					X	X	X					
9	X	X	X	X	X	X	X	X	X	X	X	X
10					X	X	X					
11					X	X	X					
12					X	X	X					

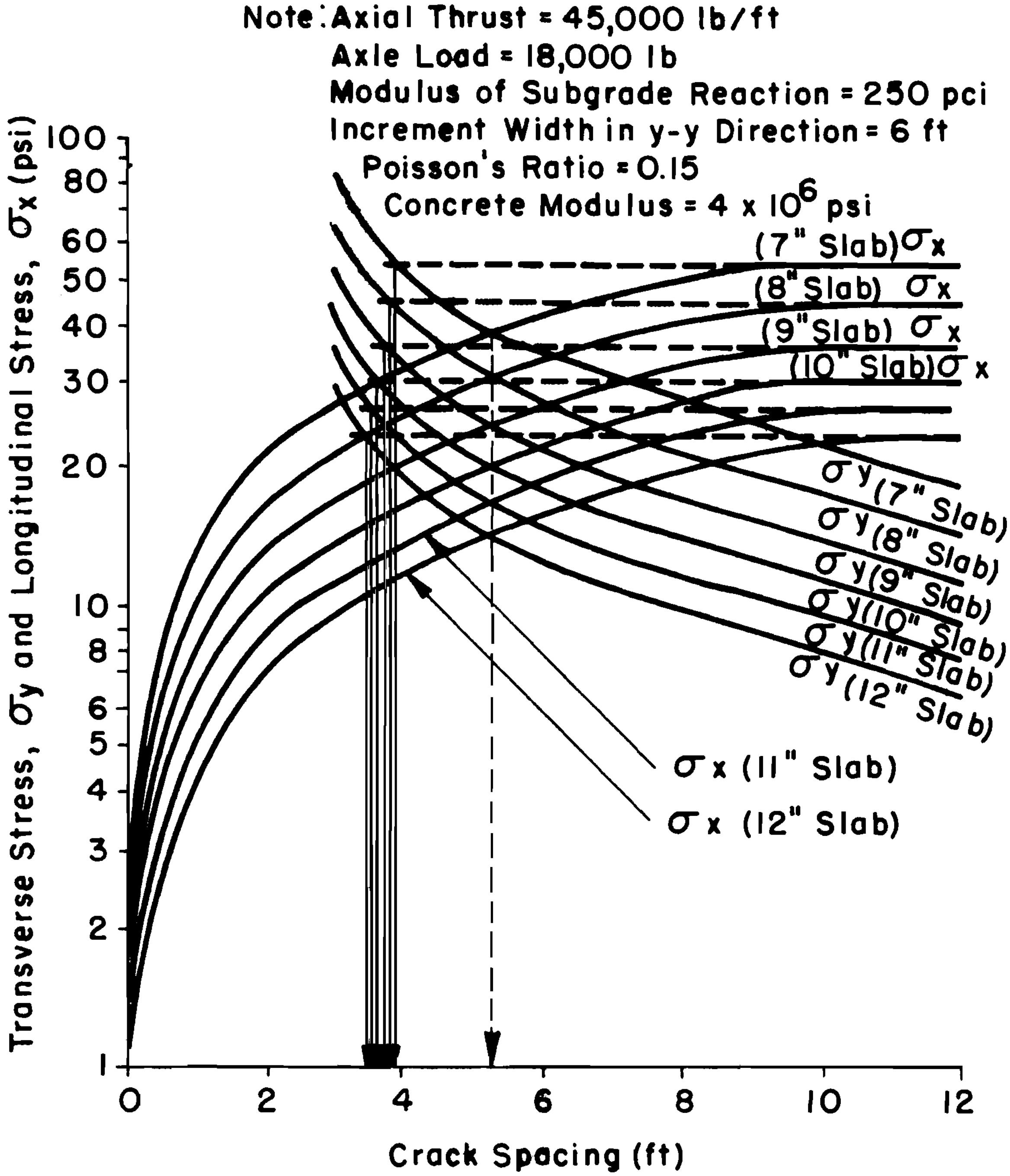
1 lbf = 4.45 N, 1 ft = 0.305 m, 1 in. = 25.4 mm

Fig 3.3. Values of variables used in determination of allowable crack spacing.



1 in.	=	25.4 mm	1 lbf	=	4.448 N
1 foot	=	0.3048 m	1 psi	=	6.895 kPa

Fig 3.4. Variation of transverse and longitudinal concrete stresses with crack spacing for various axle loads.



1 in. = 25.4 mm 1 lbf = 4.448 N
1 foot = 0.3048 m 1 psi = 6.895 kPa

Fig 3.5 Variation in transverse and longitudinal concrete stresses with crack spacing for various slab thicknesses.

and

b = required bond development length

Φ = bar diameter

f_c' = concrete compressive strength

σ_{sc} = steel stress at the crack

σ_{sm} = steel stress at mid span

The maximum required value of b likely to be encountered in practice then is that for a low strength concrete, a large bar diameter, zero steel stress at midspan, and a steel stress at the crack of just less than yield.

If

$\phi \leq 0.75$ in.,

$f_c' \geq 2{,}500$ psi,

$\sigma_{sc} \leq 60{,}000$ psi, and

$\sigma_{sm} \geq 0$,

then

$b \leq 23.7$ in.

and

$\bar{x} \geq 2 \times 23.7$ in.

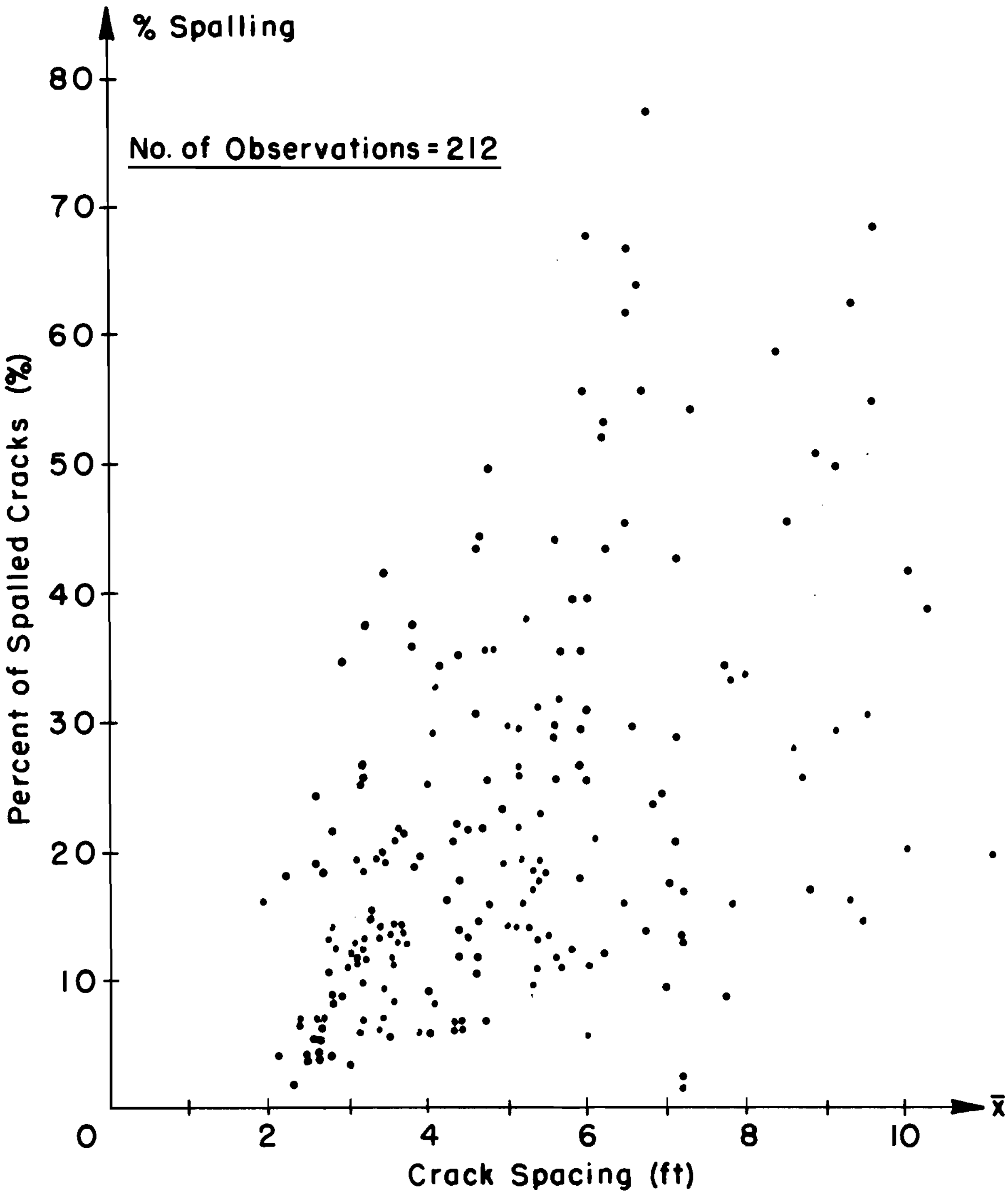
$\bar{x} \geq 4.0$ feet

In other words, since the maximum required length for full bond development is less than 2.0 feet, the minimum allowed crack spacing in this case is 4.0 feet. In general this value may be used as a lower bound on crack spacing for all CRCP designs unless excessively large reinforcing bars (0.75 in.) are used in combination with very low strength concretes ($f_c' \leq 2{,}500$ psi), which of course is very unlikely. However, in practice the designer should calculate the lower bound on $\bar{x}$ peculiar to his design situation using the procedure detailed above. In most cases this value will be of the order of three feet.

Maximum Allowable Crack Spacing - Spalling (Condition Survey) Criterion. A scattergram of "Percent Spalled Cracks" against "Crack Spacing" was plotted (Fig 3.6) using data from the 1978 Texas CRCP condition survey (Ref 11). Based upon this large sample of 212 observations taken from sections of CRCP all over the state, recommendations as to an upper bound on crack spacing can be safely made (Table 3.1). No allowance has been made here for regional or local variation since it is thought that separate estimates of the reliabilities for each district would not differ significantly from those listed in Table 3.1. From inspection of Table 3.1 it is clear that if a designer wished to restrict the fraction of spalled cracks to less than 40%, he could do so with ninety percent confidence by restricting crack spacing to no more than eight feet. However, if he wished to restrict this fraction further (say to less than 30%), his reliability drops to 84% even if he were to limit crack spacing to no more than six feet.

SUMMARY

An allowable range of crack spacing can be obtained by any CRCP designer by choosing a maximum allowable value from the spalling criterion along with a minimum allowable value from the concrete tensile stress and bond development length criteria.



1 ft = 0.305 m

Fig 3.6. Scattergram of percent spalling against crack spacing from 1978 Texas CRCP condition survey.

TABLE 3.1. EFFECT OF LIMITED CRACK SPACING ON FRACTION OF SPALLED CRACKS

Maximum Allowable Crack Spacing (feet)	Percentage (p) of Spalled Cracks (percent)	Probability That Less Than p percent of cracks will spall (percent)
10	50	92
	40	86
	30	78
	20	58
9	50	93
	40	89
	30	79
	20	58
8	50	94
	40	90
	30	78
	20	61
7	50	94
	40	90
	30	80
	20	62
6	50	98
	40	96
	30	84
	20	66

1 foot = 0.305 m

CHAPTER 4. DESIGN CRITERIA FOR CRACK WIDTH

Design criteria for crack width are established from the standpoints of controlling both water flow and spalling. In considering the water flow problem, the design criteria are developed by limiting the permanent crack width for the continuous pavement. Since permanent crack width is related to the deformation of reinforcing steel at the crack, it will be discussed together with the design criteria for steel stress.

CRACK WIDTH CRITERIA BASED ON SPALLING MEASUREMENTS

Spalling in CRC Pavement. Spalling (by which is meant "minor" or "deflection" spalling) is one of the distresses in CRC pavement. The primary causes for spalling are believed to be

(1) entrapment of road debris in cracks, which causes stress concentration when the cracks close as temperature increases,

(2) combined shear and tensile stress at joints or cracks due to horizontal temperature loading and vertical traffic loading, and/or

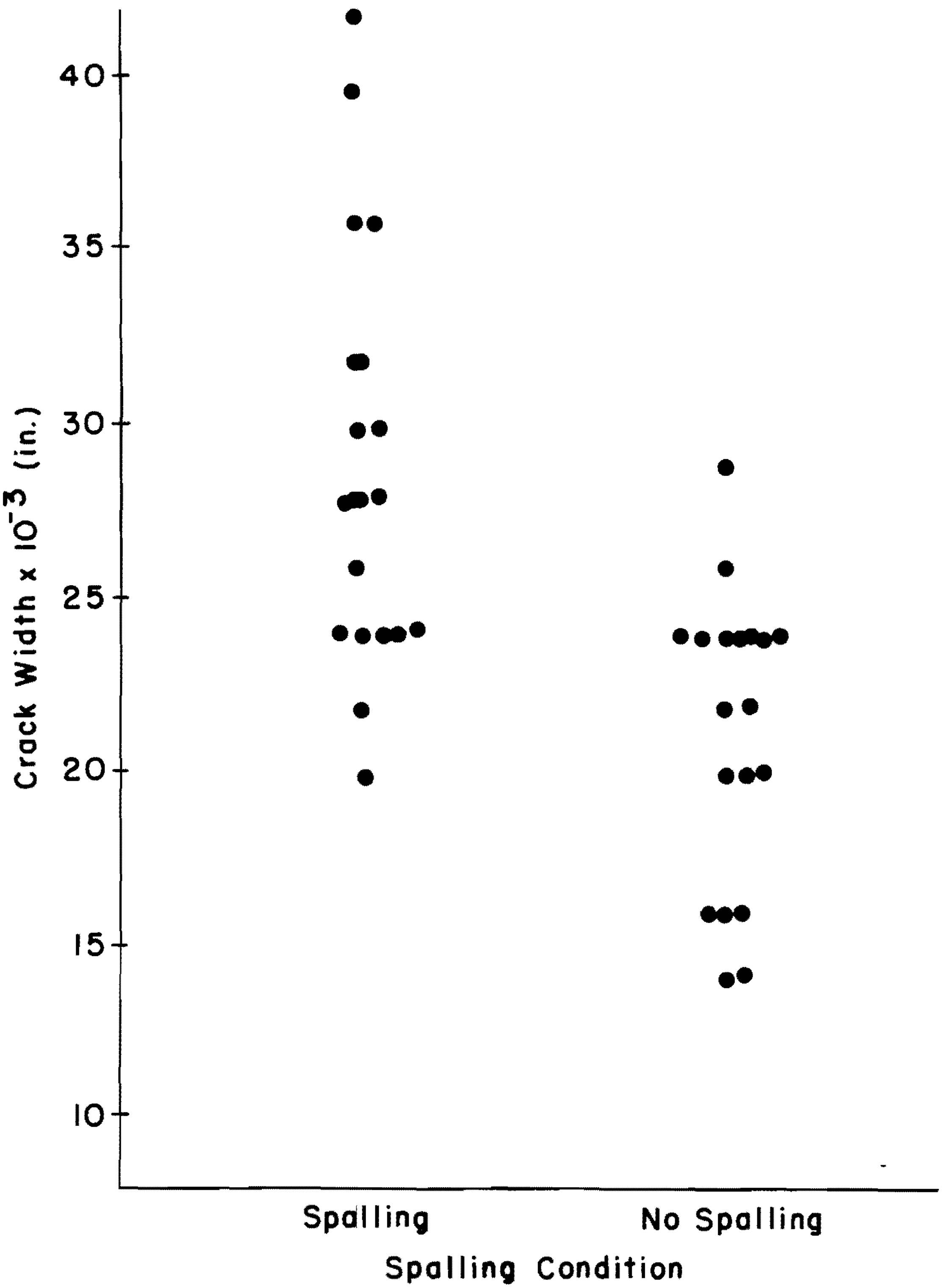
(3) poor material at surface due to overworking concrete during finishing.

Laboratory studies conducted by McCullough et al (Ref 4) indicated spalling for CRCP caused by road debris entrapment to be relatively insignificant, while the combined horizontal and vertical forces produced by repeated loading seem to be the major contributors to spalling. Darter and Barenberg's study (Ref 3) on ranking of major distresses in rigid pavements appears to corroborate McCullough's conclusion that combined horizontal and vertical forces are among the major contributors for JCP and JRCP as well as CRCP. Spalling occurred in none out of the eighteen and six out of the twelve pavements surveyed. Since the reinforcement in both JRCP and CRCP exerts horizontal forces while resisting thermal or shrinkage volume change, higher concrete stresses generally occur in these pavements than in JCP. It is believed that this contributes to stress concentrations that cause the spalling in these two types of pavements to be much more pronounced.

Correlation Between Crack Width and Spalling. Horizontal stresses developed in CRCP can be correlated with design parameters such as percent reinforcement, slab thickness, concrete modulus of elasticity, concrete strength, base friction and thermal and shrinkage coefficients. A good indicator for the amount of horizontal stress in CRCP is the crack width. In general, crack widths are directly proportional to the magnitude of horizontal stresses.

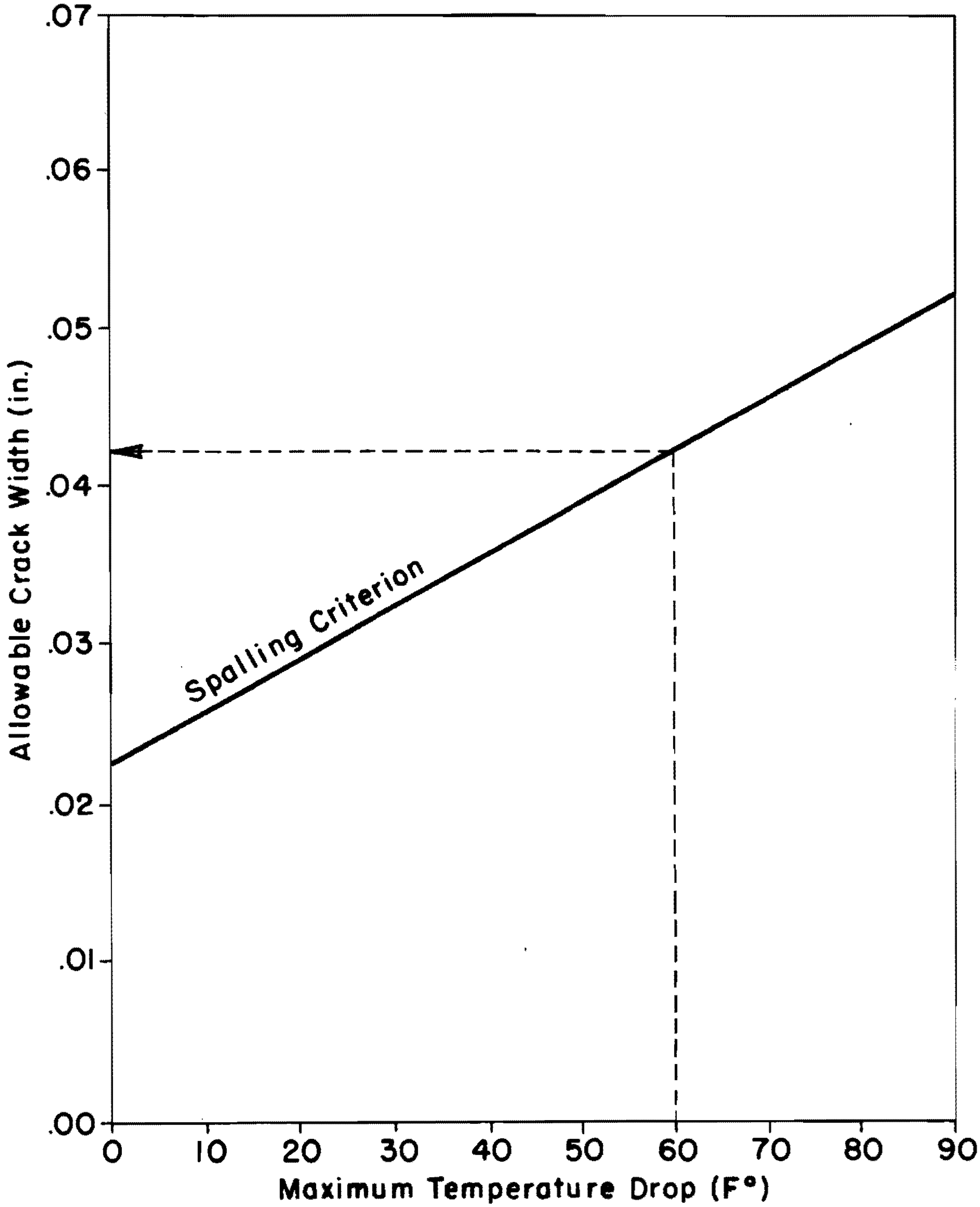
The primary spalling mechanism identified by McCullough et al was the combination of horizontal environmental stresses and stresses resulting from vertical traffic loads. Since crack width and degree of spalling both correlate highly with the horizontal stress, they should theoretically also correlate with each other. In the diagnostic study based on condition surveys of CRCP in Texas (Ref 4), crack widths were measured in the field for a temperature range of 80 - 90°F, and the results plotted with respect to the general condition of spalling. Figure 4.1 shows that spalling increases with increases in measured crack width. The mean crack width was reported to be 0.027 in. for the spalled sections and 0.021 in. for the non-spalled sections. Spalling of cracks with widths less than 0.02 in. was not observed. Similar results were obtained from a set of measurements taken in Illinois (Ref 4).

Maximum Allowable Crack Width Based on Spalling. From Fig 4.1, it is clear that the breaking point is at the level of 0.024 in. Only 5 percent of the pavements surveyed experienced spalling at crack widths less than 0.24 in. The 0.024 in. level, is therefore, used as a basis in our determination of the design criteria for crack width based on spalling. This was confirmed when a similar value was obtained following analysis of the Illinois data mentioned above. Notice that the crack widths measured in the field surveys are temperature dependent, while the spalling occurred over a long period of time during which the pavement temperature varied over a large range. Accordingly, the crack width varied over a large range of values during this period. The curve labeled 'Spalling Criterion' in Fig 4.2 characterizes this variation for the range of temperatures applicable to the surveyed pavements in Texas. Hence, Fig 4.2 must be used in the design process as described below to determine the allowable crack width for minimum temperature. First, we need to calculate the value of temperature drop in the pavement when the crack width of 0.024 in. was measured, then by back calculation, critical crack width for spalling



1 in. = 25.4 mm

Fig 4.1. Measured crack width and spalling condition for Texas CRCP sections (Ref 4).



1 in. = 25.4 mm
1 F° = 0.556 C°

Fig 4.2. Variation of allowable crack width with temperature.

under maximum temperature drop can be found. A section of CRC pavement under similar environmental conditions to those of the pavements surveyed, with a crack width equal to 0.024 in., has been used to back calculate the critical crack widths for various temperature drops. References 1 and 12 describe the theoretical approach used for the calculation of the allowable crack widths for various temperature drops for the surveyed sections, with the maximum temperature drops approximated by a mean of 60F°. Thus, the following (maximum) value of crack width in the CRC pavement is recommended to be 0.042 in., as indicated in Fig 4.2.

This value would then be compared with a value based on corrosion limiting criteria (following) and the more conservative value used in the design.

CRACK WIDTH CRITERIA BASED ON STEEL CORROSION AND SUBGRADE EROSION (PERMEABILITY)

Water Percolation in CRCP. As recognized, the purpose of the steel reinforcement is to limit the crack width to a level which will

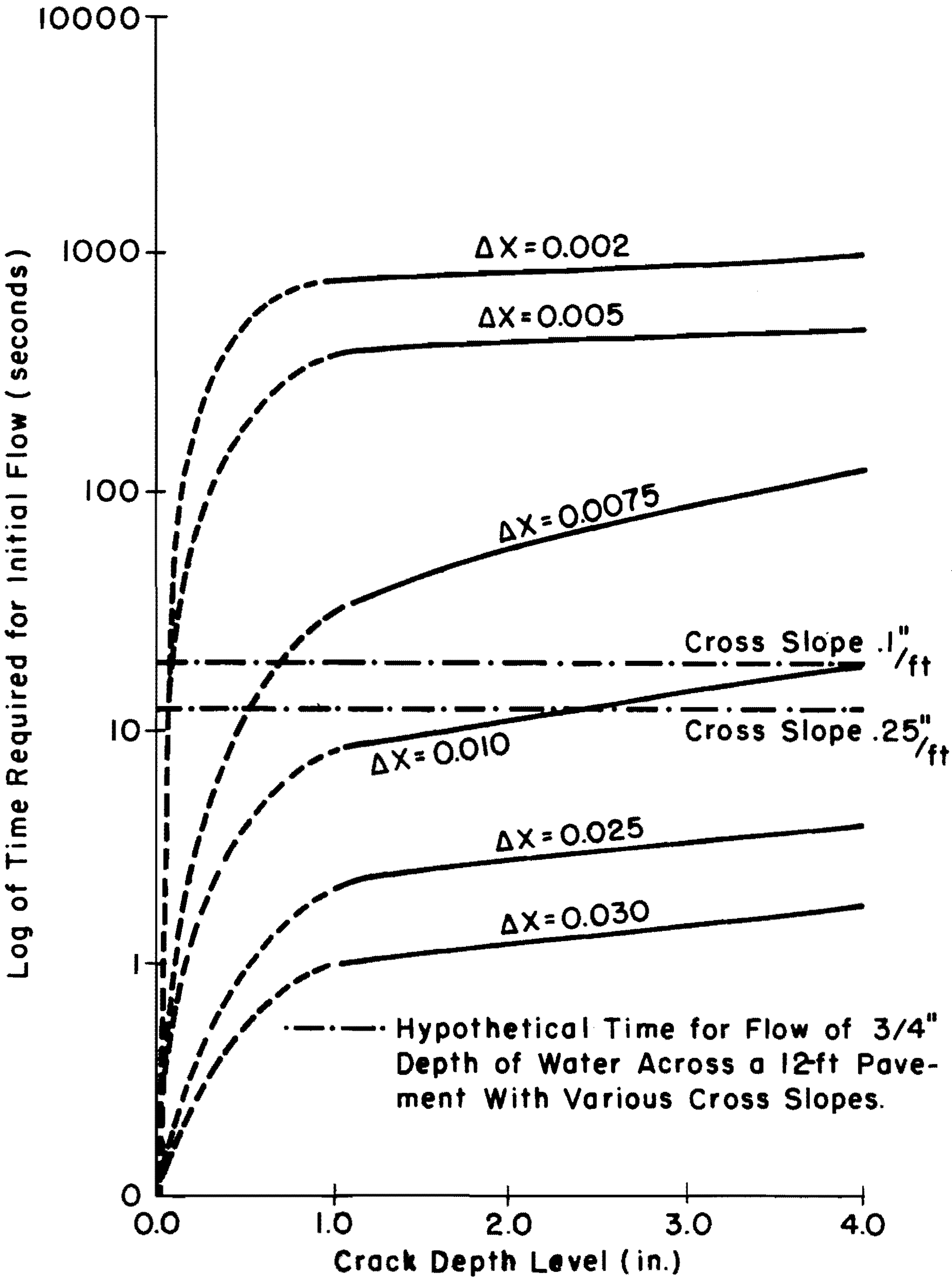
(1) provide adequate load transfer,

(2) control spalling, and

(3) avoid excessive water percolation and subsequently prevent subgrade erosion and steel corrosion.

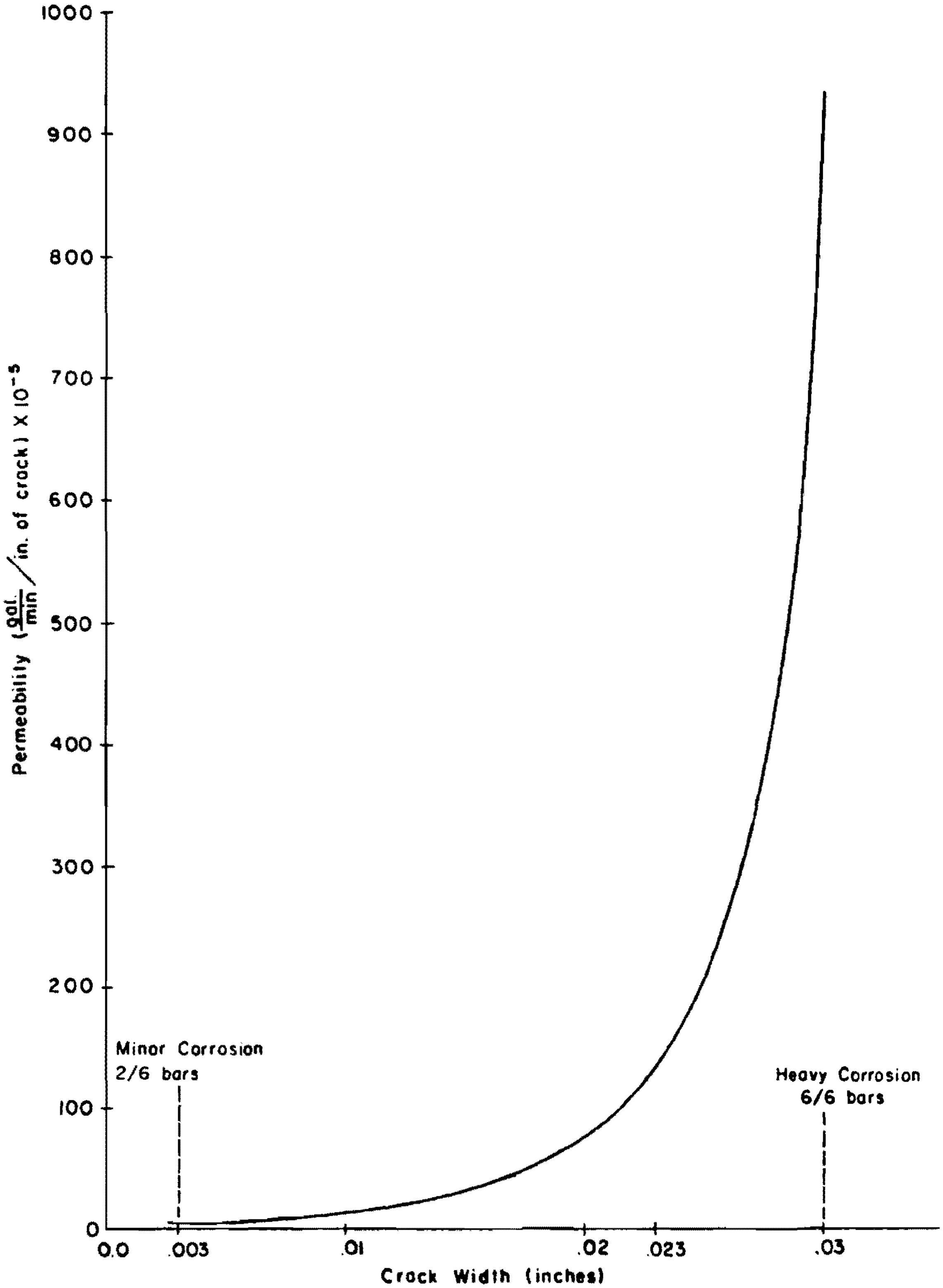
The design criterion for the crack width discussed previously has already put limits on the width of the pavements' cracks in line with objectives (1) and (2).

In considering the problem of water percolation, we may refer to the study conducted by McCullough et al (Ref 4) which presents the results of a controlled laboratory test on water percolation in CRC pavements and is summarized in Figs 4.3 and 4.4. In Fig 4.3 the relationship between various crack widths and time required for water to reach different depths in the crack was recorded. Assume a water depth of 3/4 in. and a 12-foot-wide pavement section when inspecting Fig 4.3 and 4.4. Then, the time required



1 in. = 25.4 mm, 1 ft. = 0.305 m

Fig 4.3. Water percolation rates for various crack widths (Ref 4).



1 in. = 25.4 mm, 1 gal./min./in. = $2.48 \times 10^{-6} m^3/s/mm$

Fig. 4.4. Results of water percolation permeability tests (Ref 4).

to flow across a CRC pavement section (for various cross slopes) can be calculated by using Manning's formula for open channel flow.

Thus

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

where

V = mean velocity,

n = approximately 0.016 for rough concrete,

R = hydraulic radius, and

S = slope of channel bed.

The time required for water to percolate to various depths, and the cross-pavement flow times were superimposed in Fig 4.3. It shows that a crack of width less than 0.01 in. can prevent water from reaching the subgrade. In that same study, it was found that for a crack of width less than 0.01 in., virtually no rusting of the steel developed. Similarly, a study conducted by the Illinois Department of Transportation (Ref 13) also supports this observation. It was found that a crack of width equal to or greater than 1/128 in. has a greater potential for the occurrence of significant rusting of the reinforcing steel. Ideally, based on these studies the steel reinforcement would be designed to control the crack width to a level of less than 0.01 in. under the most critical situation (when the temperature is lowest and the pavement is flooded). However, to design for such a criterion is highly impractical, since, to keep crack width at such a level will require an exorbitant amount of steel and cause excessive cracking. Also, such a restriction is unnecessarily conservative, since this most critical situation occurs for only a small fraction of each year of the pavement's life. Consequently, the designer should choose maximum crack width such that it is within a sensible range of values and yet keeps the steel corrosion caused by any water that may reach the steel, down to an acceptable level using the procedure discussed in the following paragraph.

Maximum Allowable Crack Width Based on Permeability Restriction

Figure 4.4 relates the quantity of flow of water into the crack (permeability in gallons per minute per inch of crack as determined by measured headloss in the ponded water) to crack width and to degree of steel corrosion. As is shown in Fig 4.4, the permeability of cracks below the 0.01-in. level is really quite small (resulting in minor corrosion only) while the permeability of cracks and associated corrosion between the 0.01 in. level and the 0.025 in. level is only slightly larger. However, above the 0.025 in. level, the cracks are extremely permeable, with substantial quantities of water flowing into the pavement and subsequent heavy corrosion and subgrade erosion occuring. Accordingly, for design purposes, if the pavement were to be continuously flooded and left constantly at a temperature just above freezing, crack width would have to be kept below the 0.025 in. level.

Yet, since neither of these two extreme conditions are likely to occur constantly throughout the entire life of the pavement, the value of 0.025 in. should be adjusted accordingly. Specifically, by using data representative of environmental conditions applicable to the particular pavement under consideration (Ref 20), regional modifying factors could be calculated. By examining the distribution of maximum daily temperature drops from curing temperature in any one year, the value of temperature drop from curing (in Fahrenheight degrees) which will not be exceeded a chosen fraction of the time (usually 95 percent) may be calculated. The technique for calculation of change in crack width with change in temperature, which was discussed for the spalling restriction, should be applied. This would involve preparing a chart similar to Fig 4.2. The maximum allowable design crack width whould be that value corresponding to the temperature drop from curing, which was obtained at the 95 percent confidence level as discussed above. It is important to notice that the designer should obtain climatogical data, and prepare a temperature distribution, and a crack width-temperature plot appropriate to the environment of the particular pavement which is being designed.

MAXIMUM CRACK WIDTH FOR DESIGN

The lower of the two maximum allowable crack widths as recommended by the spalling and permeability restrictions should be chosen as the design maximum crack width.

CHAPTER 5. DESIGN CRITERIA FOR STEEL STRESS

Two criteria are used to define the allowable steel stress in continuously reinforced concrete pavement. First, the steel stress must be lower than its ultimate tensile strength divided by a safety factor. This criterion is to safeguard against rupturing of the steel under high tension. Second, if the steel stress is to be greater than yield, permanent crack width associated with the permanent deformation of steel at the crack must be less than the permissable amount to avoid excessive water percolation.

CRITERIA FOR STEEL RUPTURE

To guard against rupturing of the steel, the allowable stress in steel is set to be less than ultimate strength times a safety factor of 0.75. Table 5.1 (Ref 14) shows the ultimate strength for various types of deformed bars and their allowable stress against rupture.

CRITERIA FOR PERMANENT DEFORMATION

Conventional design criteria for steel stress generally require the stress be less than the yield strength times a safety factor. Such criteria prevent the steel from undergoing plastic deformation. Based on our experience, however, we know that many miles of CRC pavements have been performing adequately while their steel stresses are predicted to be higher than yield. This prompts us to consider the adequacy of such steel stress limits as criteria by evaluating the response of steel reinforcement in the CRC pavement when stressed beyond the elastic range. Less conservative criteria can be then obtained by evaluating the maximum stress in the steel in terms of its permanent deformation which is equal to crack width at the point of maximum stress. The maximum allowable steel stress is thus calculated by keeping the crack width below some suitable value.

TABLE 5.1. MAXIMUM ALLOWABLE STEEL STRESS TO PREVENT RUPTURE IN CRCP (Ref 6)

Steel Type and Grade	Minimum Yield Strength f_y ksi	Ultimate Strength f_u ksi	Allowable Stress f_s ksi
Billet Steel			
Grade 40	40	60	52.5
Grade 60	50	90	67.5
Grade 75	100	100	75.0
Rail Steel			
Grade 50	50	80	60.0
Grade 60	60	90	67.5

1 ksi = 6.895 MPa

EVALUATION OF PERMANENT DEFORMATION OF STEEL

Plastic deformation of steel in CRC pavement can be determined by multiplying the plastic strain at the crack by a defined gauge length. The gauge length at which the steel undergoes plastic deformation can be approximated as the length of the region in the bond slip zone where steel stress is above yield. To estimate the gauge length, it is necessary to review the basic CRCP model.

Figure 5.1 shows a steel stress distribution diagram for a CRCP section under the effect of volumentric change. At the crack, since concrete provides no resistance, the steel tension is at maximum. Moving away from the crack, an increasing amount of tension force will be carried by the concrete, thus reducing the tensile stress in the steel. The rate of change in stress or the slope of the stress diagram at the bond slip zone depends on the bond strength between the steel and the concrete. The rate of change in steel stress can be determined by summing the forces acting on the steel bar. From Fig 5.1 (c) at the bond slip zone

$$F_s + dF_s = F_s + Udx$$

By combining terms and solving for U,

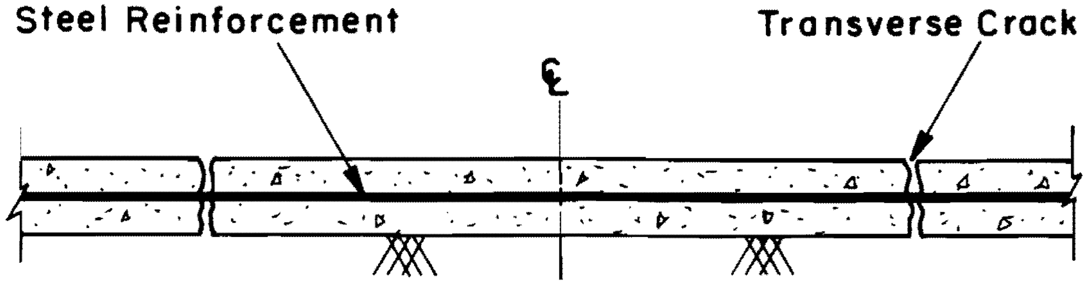
$$U = \frac{dF_s}{dx}$$

where

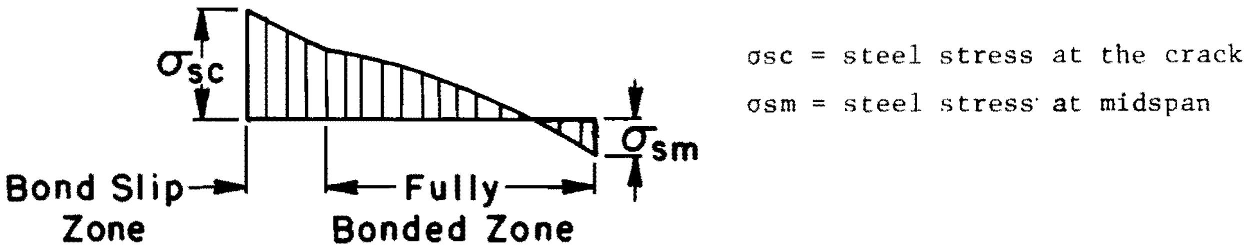
U = average bond force per unit length of bar.

The average bond force may also be expressed as

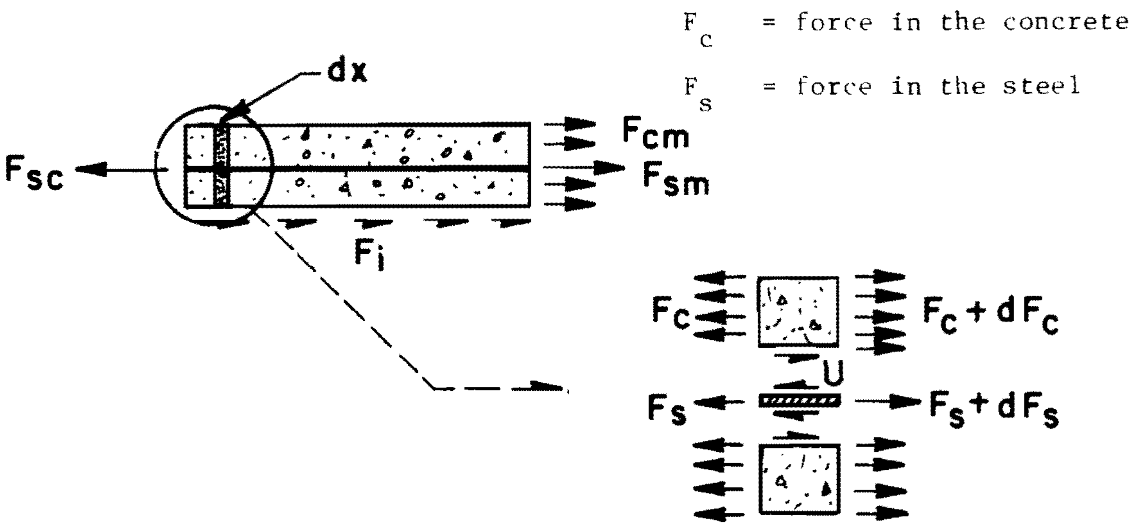
$$U = uE_o$$



(a) CRCP geometric model



(b) Conceptual Stress Diagram



(c) Free-body diagram of CRCP model and of an element in the bond slip zone

Fig 5.1. Free-body diagram and stress distribution in CRCP model.

where

$$u = \text{shear bond strength (psi)}$$

$$= \frac{9.5\sqrt{f'_c}}{\phi}$$

$$f'_c = \text{concrete compressive strength (psi),}$$

$$E_o = \text{bar perimeter (in.),}$$

and

$$\phi = \text{bar diamter (in.).}$$

By equating the two expressions for average bond force and converting to stress, the following is obtained:

$$A_s \frac{d\sigma_s}{dx} = uE_o$$

$$\frac{\pi\phi^2}{4} \cdot \frac{d\sigma_s}{dx} = u\,\pi\,\phi$$

$$\frac{d\sigma_s}{dx} = \frac{4u}{\phi}$$

Knowing the slope of the steel stress diagram at the bond slip zone, the gauge length, ℓ, can be estimated as

$$\ell = 2 \cdot \frac{(\sigma_{sc} - \sigma_{yield} \times SF)}{d\sigma_s/dx}$$

$$\ell = 2\,\frac{\phi}{4u}\,(\sigma_{sc} - \sigma_{yield} \times SF),$$

where

$$SF = \text{safety factor}$$

$$\ell = \frac{\phi^2}{19\sqrt{f'_c}} \times (\sigma_{sc} - \sigma_{yield} \times .75)$$

By approximating the plastic strain E_p to be σ_{yield/E_s}, the amount of permanent deformation $\Delta x'$ in the steel becomes

$$\Delta x = 2\ell \times \sigma_{yield/E_s}$$

$$= \frac{2\ \phi^2}{19\sqrt{f'_c}} (\sigma_{sc} - \sigma_{yield} \times .75) \frac{\sigma_{yield}}{E_s}$$

PREDICTION OF ALLOWABLE STEEL STRESS

For permanent deformation being less than 0.01 inch, the maximum allowable steel stress at the crack can be obtained by setting Δx of the above equation equal to 0.01 inch, which gives

$$0.01 = \frac{2\ \phi^2}{19\sqrt{f'_c}} (\sigma_{max} - \sigma_{yield} \times .75) \cdot \frac{\sigma_{yield}}{E_s}$$

Thus

$$\sigma_{max} = \frac{.19\ E_s \sqrt{f'_c}}{\phi^2 \sigma_y} + (\sigma_y \times .75)$$

where

σ_{max} = allowable steel stress

σ_y = steel yield stress .

Table 5.2 summarizes the allowable maximum steel stress for various bar diameters and steel yield strengths for low (f'_c < 3,500 psi) and regular (3,500 $\leq$ f'_c) strength concretes.

MAXIMUM STEEL STRESS FOR DESIGN

The limiting value on steel stress to be used in design should be chosen as the lower of the maximum allowable steel stresses recommended from Tables 5.1 and 5.2. That is, the minimum recommendation from the steel rupture and permanent deformation criteria should be used.

TABLE 5.2. MAXIMUM ALLOWABLE STEEL STRESS FOR CONTROL OF PERMANENT DEFORMATION

Steel Yield Strength f_y (psi)	Steel Bar Diameter ϕ (in.)	Maximum Allowable Steel Stress (psi)	
		Concrete Compressive Strength, f'_c (psi)	
		Low, $f'_c < 3500$ psi ($\geq$ 4.5% Air Content, or $\leq$ 4 cement sacks per cubic yard of concrete)	Regular $f'_c \geq 3500$ psi ($\leq$ 4.5% air content and $\geq$4 cement sacks per cubic yard of concrete)
40,000	0.500	60,200	64,900
	0.625	49,300	54,900
	0.750	43,400	47,300
	0.875	39,900	42,700
50,000	0.500	61,600	68,700
	0.625	53,000	57,500
	0.750	48,200	51,400
	0.875	45,400	47,700
60,000	0.500	65,100	71,000
	0.625	57,900	61,600
	0.750	53,900	56,500
	0.875	51,600	53,500
75,000	0.500	72,300	77,000
	0.625	66,600	69,500
	0.750	63,400	65,500
	0.875	61,500	63,000

1 in. = 25.4 mm

1 psi = 6.895 kPa

CHAPTER 6. USE OF LIMITING CRITERIA AND SELECTION OF INPUT VALUES FOR THE DESIGN OF CRCP

SUMMARY

This report determines the design criteria for limiting the structural response variables (crack spacing, crack width and steel stress) in the continuously reinforced concrete pavement. Previous investigations of the design criteria were reviewed and studied along with recently developed analytical models. The basic procedure used to establish the design criteria involved examination of the major distresses such as punchout, spalling and steel rupture and study of correlations between these distresses and level of the corresponding structural responses.

Design criteria for crack spacing are based on the control of punchout in the continuous pavement. As transverse crack spacing becomes relatively narrow and when load transfer at the crack deteriorates, the pavement structure no longer acts as a longitudinal beam but as a transverse beam with stress in the transverse direction higher than in the longitudinal direction. High transverse stress will cause longitudinal cracks to show up crossing the transverse crack and eventual deterioration into punchout failure. The critical crack spacing is therefore the spacing at which stress in the transverse direction becomes dominant. Criteria developed on the basis of limiting spalling and punchouts are also described, as are criteria to insure adequate bond development length. All three sets of criteria are to be considered in choosing a minimum and a maximum crack spacing.

Design criteria for crack width are established for the control of both concrete spalling and corrosion of the steel reinforcement. The combined horizontal and vertical forces are identified to be the major contributors to spalling. Since crack width and degree of spalling both correlate highly with the horizontal stress, they should also correlate with each other. Allowable crack width is determined at the level where spalling is minimal. Steel corrosion is prevented by restricting crack width to a value which will

prevent water reaching the steel. The smaller of the two minimum crack widths is then the controlling value for the design.

Design criteria for maximum steel stress at the crack for the continuous pavement are based on two separate mechanisms. First, the steel stress must be within safety limits below the level of steel rupture with consideration being given to fatigue of the metal. Second, if the steel stress is to be greater than yield, the permanent crack width associated with the permanent deformation of steel at the crack must be less than the permissible amount to avoid excessive water percolation. The lower of the allowable stresses recommended by the two criteria is chosen as the controlling value for the design.

The design criteria for crack spacing, crack width, and steel stress are thus summarized in Chapters 3, 4, and 5, respectively, where the fundamental concepts concerning the behavior and reaction of the CRC pavement under different loading conditions are discussed. It is clear from this discussion that recognition of the general tendency toward increase or decrease in crack spacing, crack width, and steel stress as affected by design variables facilitates making the decision to select higher or lower levels of various design variables.

The results from this study are therefore presented as a subsequent report to be used in conjunction with the report "Nomographs for the Design of CRCP Steel Reinforcement" as discussed in the following section.

USE OF LIMITING CRITERIA IN DESIGN PROCEDURE

In report 177-16 (Ref 2), the relationship between the significant input variables and the structural responses predicted by the CRCP-2 model are quantified using regression techniques and expressed as a set of nomographs. This set of design charts enables us to graphically predict the final responses of the pavement to the total load. Crack spacing, crack width and steel stress are predicted. It should be noticed that the CRCP-2 model only simulates the loading conditions of environmental force and bending stress under application of a single wheel load. It should also be noticed that fatigue cracking caused by the combination of repetitive wheel loads and reduction of tensile strength due to fatiguing of the concrete material was not considered in Report 177-16. However, it is proposed to treat fatigue in the design process by following the procedure for CRCP design outlined below.

(1) Determine the design slab thickness on the basis of fatigue analysis alone as shown in the following section "Guidelines for Selection of Design Input Variables."

(2) With this slab thickness and chosen values of the other trial input variables, predict the final crack spacing, crack width, and steel stress with the nomographs in Report 177-16.

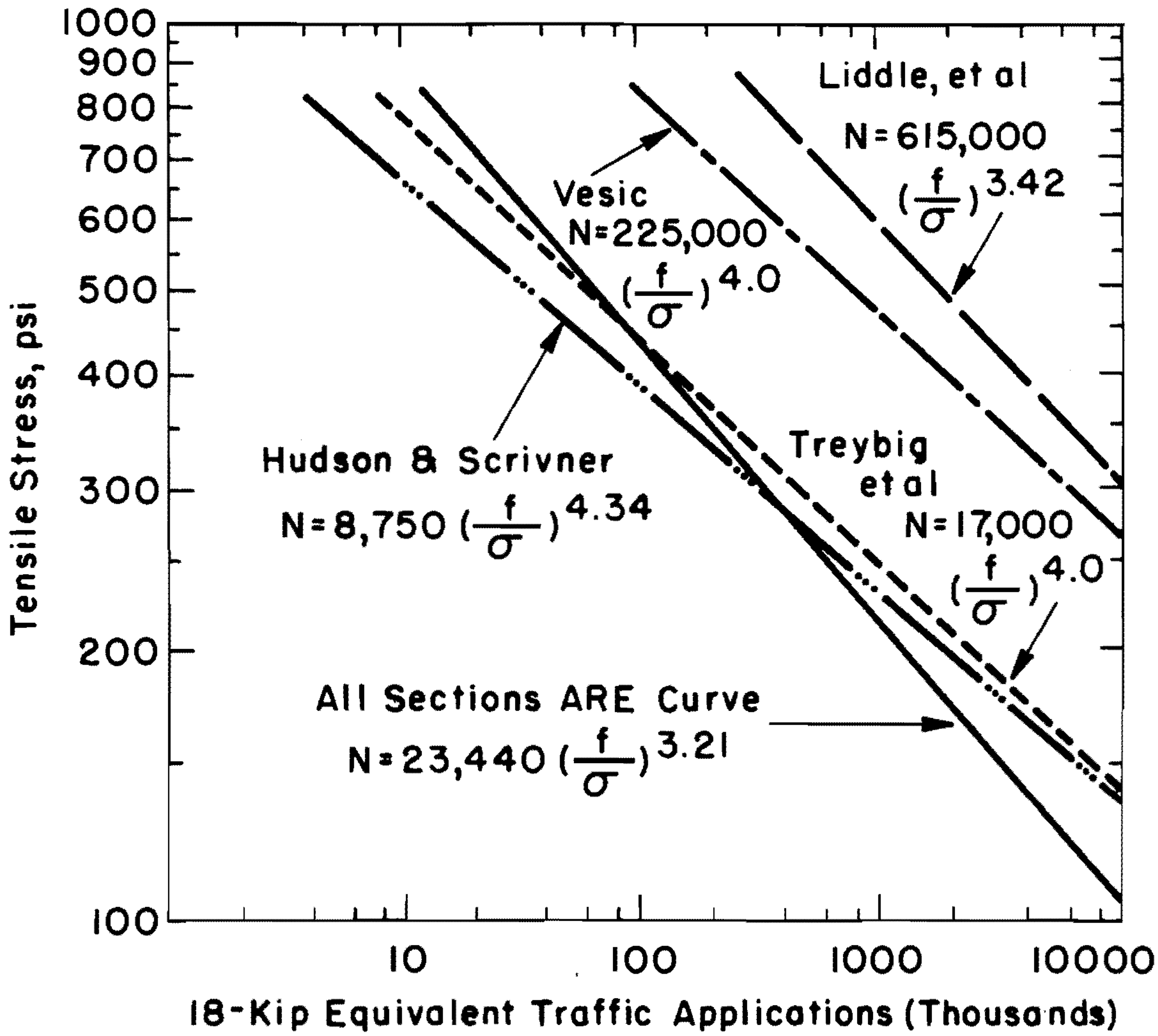
(3) Check the predicted responses with the design criteria established in Chapters 3, 4 and 5 of this report.

(4) If the predicted responses exceed the allowable criteria, the level of design variables should be lowered or raised according to the general behavior of the CRC pavement as discussed in Chapter 2 of this report.

(5) If changes in input variables involves a change in slab thickness or concrete flexural strength, Step 1 should be repeated.

GUIDELINES FOR SELECTION OF CRCP DESIGN INPUT VALUES

The fatigue analysis mentioned in Step 1 of the Design Procedure calls for selection of a fatigue relationship for the rigid pavement. Proper relationships are difficult to obtain as the information described in the literature varies widely. Figure 6.1 shows fatigue curves that have been developed from evaluations of AASHO Road Test and other data and essentially represent fatigue relationships based on performance of real slabs. The relationship designated as "All Sections ARE Curve" was developed from multiple regressions on AASHO Road Test data. Interior stresses predicted by elastic layer theory, a flexural strength of 700 psi, and Class 3 and 4 cracking as a failure criterion were used as data for the multiple regression. Development of this relationship is discussed in Reference 15. The ARE curve is believed to be the best available and the most rational at this time. Also it leads to conservative designs and this is important for prediction purposes owing to the many as yet unexplained variables affecting the behavior of the pavement.



1 psi = 6.895 kPa

1 kip = 454 kg

Fig 6.1. Comparison of A.R.E. fatigue curve and previously developed relationships (Ref 15).

(1) Selection of Slab Thickness (in.). The process of selecting a slab thickness by using the fatigue curves in Fig 6.1 is illustrated for a typical set of conditions in the following design example.

Assume a traffic volume for a representative design section of 20 million 18-kip ESALS and for a 20-year design period. Then to design the slab thickness for a continuously reinforced concrete pavement, the following data are also assumed:

concrete compressive strength	f'_c	=	4000 psi
concrete modulus of elasticity	(E)	=	4,000,000 psi
concrete flexural strength	(f)	=	700 psi
modulus of subgrade reaction	(k)	=	300 pci
Poissons ratio	(μ)	=	0.15
radius of type contact area	(a)	=	6.5 in.
maximum wheel load	(P)	=	12 kip

Based on the ARE fatigue relationship, the maximum allowable bending stress for 20 million load applications is

$$N = 23440\left(\frac{f}{\sigma}\right)^{3.21}$$

$$\sigma = \frac{700}{\sqrt[3.21]{\dfrac{20000000}{23440}}} = 86 \text{ psi}$$

By using Westergaard's interior equation for stress analysis (Ref 16), the required slab thickness is obtained as follows; using

$$\sigma = \frac{.316P}{h^2}\left[4 \log_{10}\left(\frac{\ell}{b}\right) + 1.069\right]$$

where

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)\ k}} \text{ is the radius of relative stiffness}$$

and

$$b = \sqrt{1.6\ a^2 + h^2} - 0.675\ h \text{ for } a < 1.724h$$

or

$$b = a \quad \text{for } a \geq 1.724h.$$

Then, trial one,

$$h = 8 \text{ in.}$$

gives

$$\ell = 27.6 \text{ in.}$$

and

$$\sigma = 114 > 86 \text{ psi,}$$

trial 2,

$$h = 9 \text{ in.}$$

gives $\sigma = 95 > 86$ psi,

and trial 3,

$$h = 10 \text{ in.}$$

gives $\sigma = 81 < 86$ psi.

Hence a 10-inch slab is the smallest which would satisfy the design criteria.

(2) Thermal Coefficient of Steel. Various values of coefficient of thermal expansion are listed in Table 6.1 for different steels. A value of 5×10^{-6} (in./in./F°) is commonly used.

(3) Thermal Coefficient of Concrete. The thermal expansion and contraction of concrete vary with factors such as richness of mix, water-cement

TABLE 6.1 COEFFICIENT OF THERMAL EXPANSION FOR VARIOUS METALS (Ref 17)

Material	Coefficient of Thermal Expansion (α) in./in./F° x 10^{-6}
Cast iron, ductile, as cast	7.5
Steel, 0.2 percent C, hot-rolled	6.7
Steel, 0.2 percent C, cold-rolled	6.7
Steel, 1.0 percent C, hot-rolled	7.3
Stainless steel, type 302, cold-rolled	8.9
Silver (sterling)	10.0
Steel (1020)	6.5
Steel (1040)	6.3
Steel (1080)	6.0
Steel (k8Cr-SNi stainless)	5.0

C = carbon content

1 in. = 25.4 mm

1 F° = 0.556 C°

ratio, temperature range, concrete age, and relative humidity. However, the main factor affecting the thermal properties of concrete is the mineralogic composition of the aggregate. Figure 6.2 (Ref 17) shows some experimental values of thermal coefficient of linear expansion for neat cements and for mortars and concretes with different kinds of aggregates. The coefficient appears to be very much influenced by the type of coarse aggregate, being highest for quartz, followed by sandstone, granite, basalt, and limestones. Gravel, which varies considerably in its minerological composition, may have a thermal coefficient of about five to seven millionths (in./in.F°).

(4) Drying shrinkage strain. Drying shrinkage of concrete is one of the principal causes of cracking. Upon exposure to drying conditions, moisture slowly diffuses from the interior mass of the concrete to the surface, replacing the moisture loss by surface evaporation. There are many factors that influence the magnitude of drying shrinkage, such as: water content, type of aggregate, type of cement, moisture, temperature condition, size of pavement slab, and duration of moist curing. The drying shrinkage varies commonly from 0.0002 to 0.0006 in./in.

The single largest factor that influences shrinkage is the water content and the relationship is shown in Fig 6.3 (developed by the Bureau of Reclamation (Ref 17)).

(5) Concrete tensile strength. The selection of the design value of the tensile strength of the concrete (f'_t) is a process which employs any one of several models depending upon the inference space of the proposed design. Ideally, the value of f_t to be used in design should be obtained from an indirect (or splitting) tensile test on cylindrical specimens which have been taken as samples from the concrete to be used in construction of the pavement.

Alternately, the designer could use the age-tensile strength model employed in the computer program CRCP-2 (Ref 1) which calculates the concrete-tensile strength at any given time after construction in terms of the concrete compressive strength at that time (Ref 18). It is important to allow for the gain in concrete strength after the first 28 days following construction, since the lowest temperature is likely to occur after a prolonged period of time following construction and the largest internal load will occur at this stage.

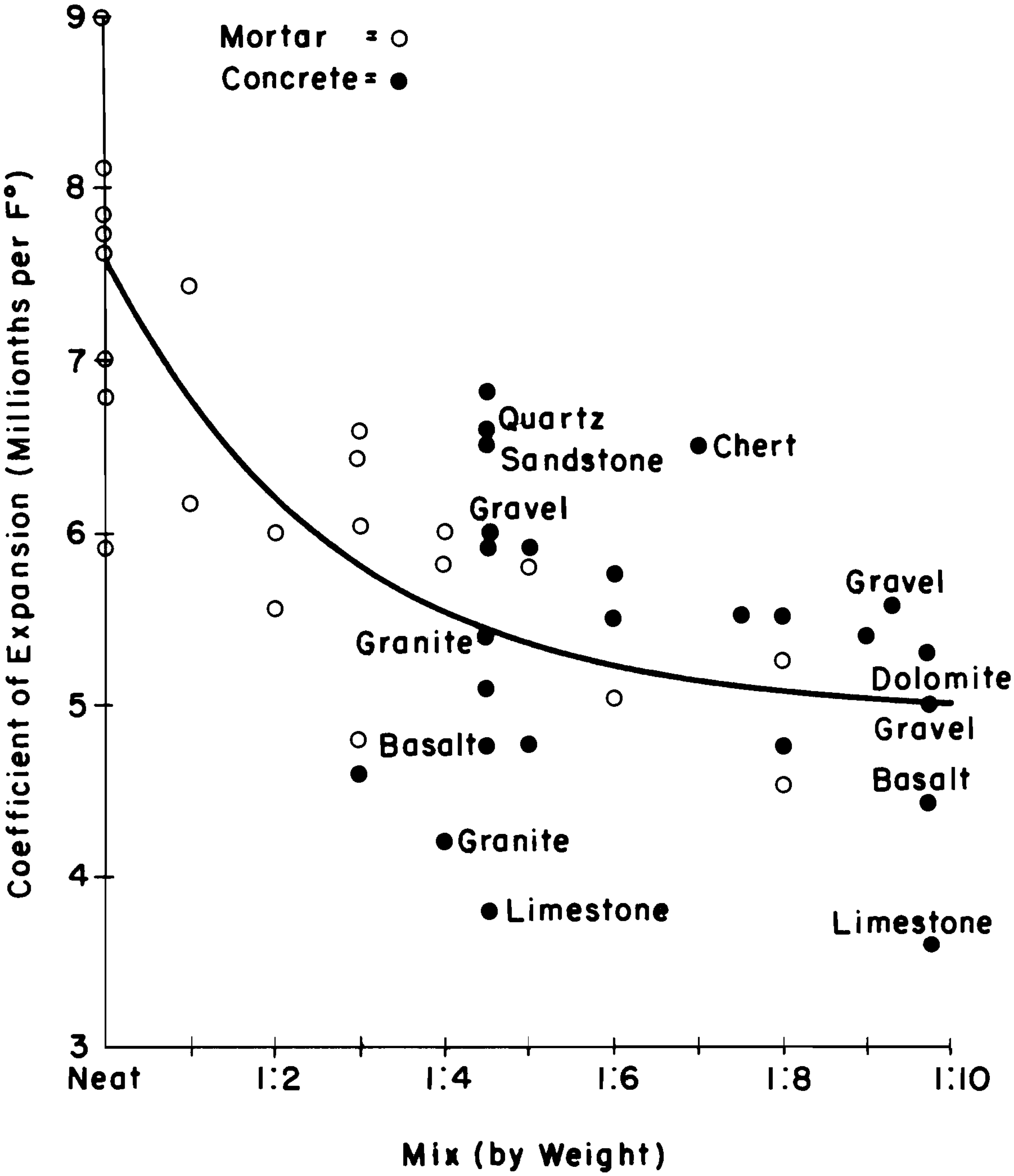
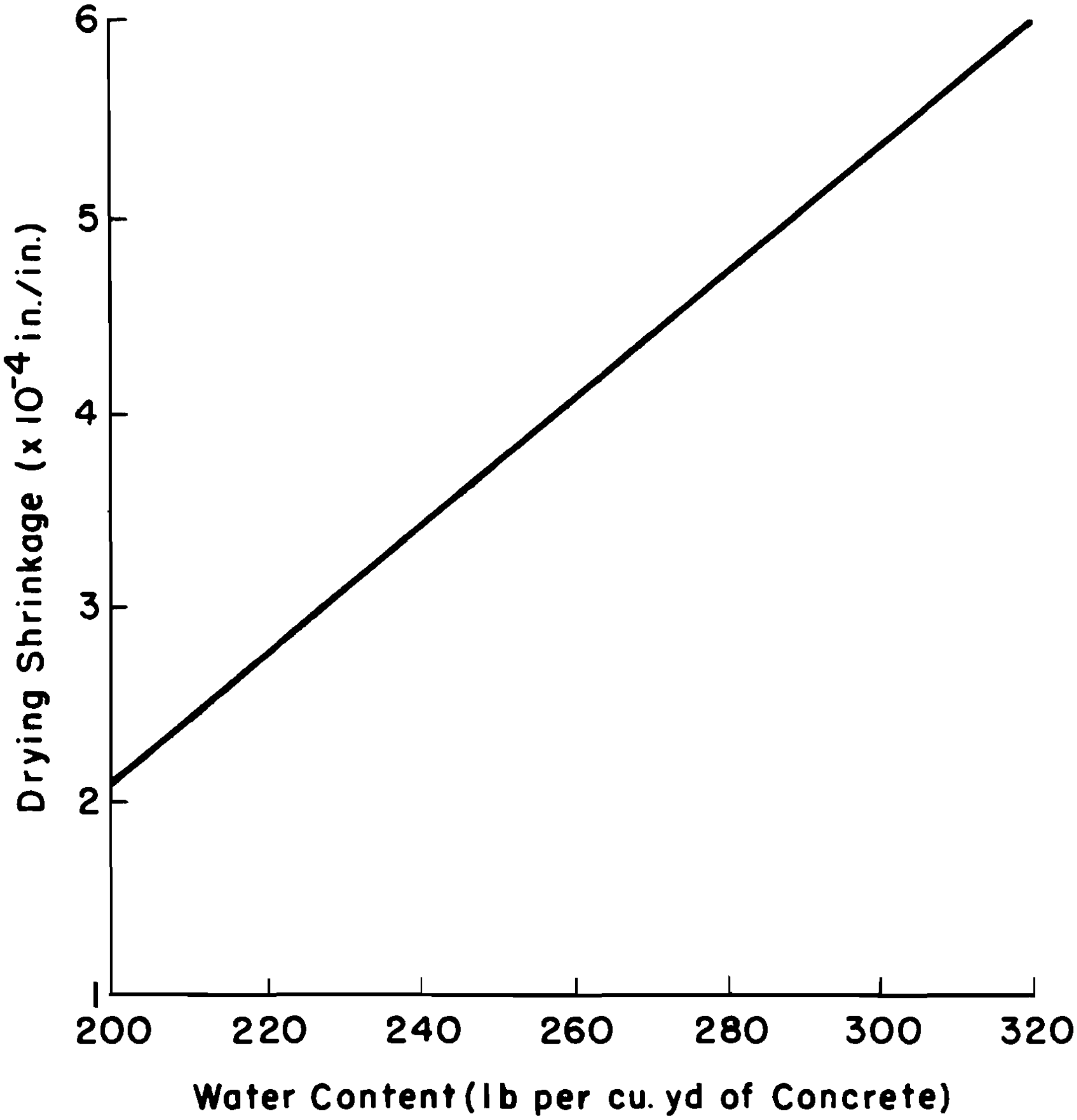


Fig 6.2. Thermal coefficients of expansion of neat cement, mortars, and concrete (Ref 17).



1 in. = 25.4 mm

1 lb/yd^3 = 0.593 kg/m^3

Fig 6.3. Typical effect of water content on drying shrinkage (Ref 17).

If

x = the number of days from the time of construction to the time at which the strength is being calculated (usually the time at which the annual minimum temperature occurs.)

$f'_{c,28}$ = concrete-compressive strength at 28 days after construction (psi),

$f'_{c,x}$ = concrete compressive strength at x days after construction (psi),

$f'_{u,x}$ = concrete flexural strength at x days after construction (psi),

$f'_{t,x}$ = concrete-tensile strength at x days after construction (psi),

and

γ = flexural-tensile factor (constant for given concrete type (psi)),

then

$$f'_{c,x} = f'_{c,28}\left[1 + 0.1972 \log \left(\frac{x}{28}\right)\right]$$

and

$$f'_{u,x} = \frac{3000}{3 + \left[\left(\frac{12000}{f'_{c,x}}\right)\right]}$$

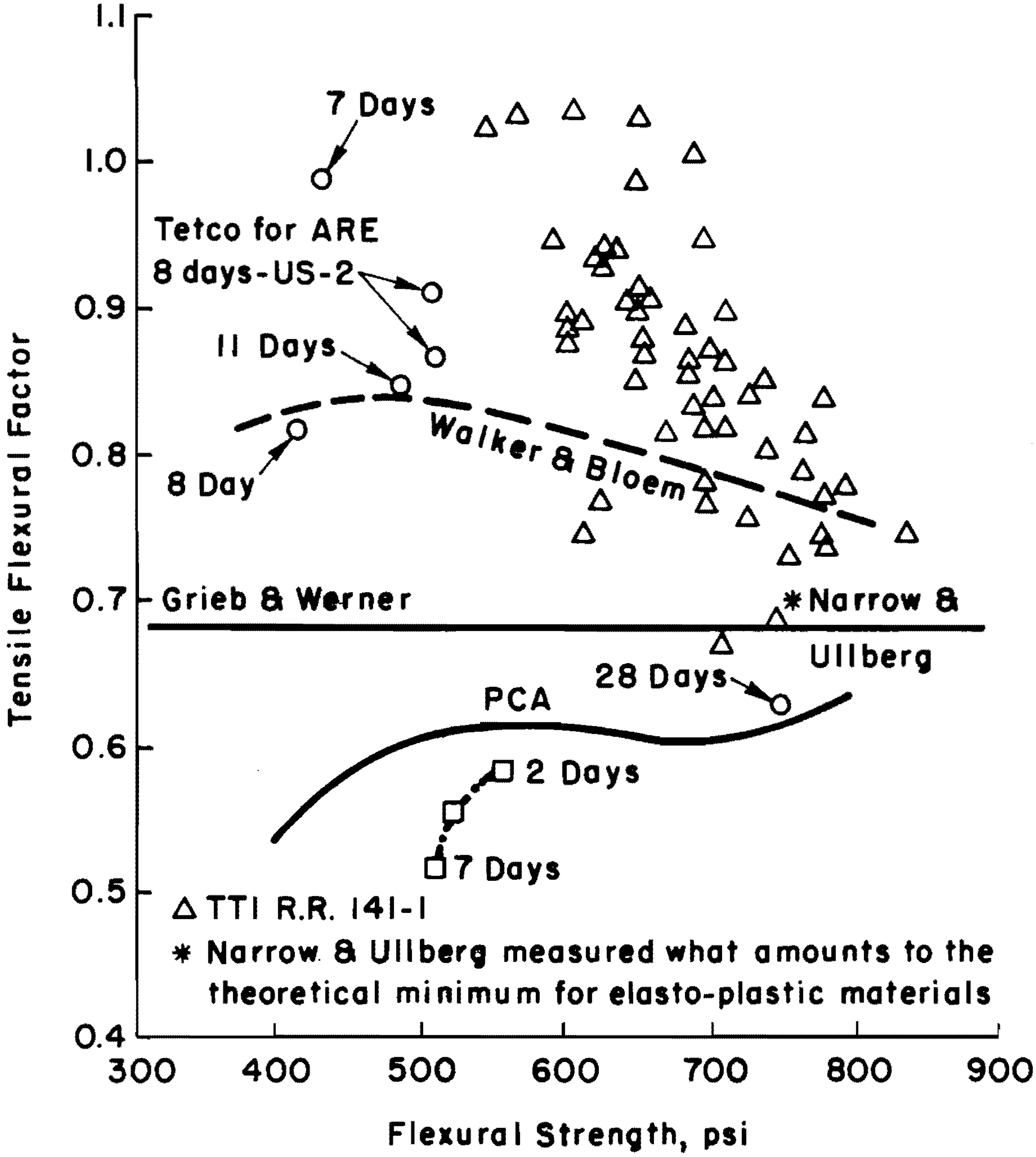
and finally,

$$f'_{t,x} = f'_{u,x} \cdot \gamma$$

where the value of γ varies from 0.5 to 1.0 and is chosen from Table 6.2, (Ref 19) or from Fig 6.4 (Ref 17) according to the type of concrete to be used. It is important to notice that values of f'_t calculated using this second approach are only applicable within the range of f'_u shown in Fig 6.4, and

TABLE 6.2. CONCRETE FLEXURAL-TENSILE STRENGTH FACTOR (Ref 19)

Concrete Type	Flexural-Tensile Conversion Factor
Gravel Aggregate	0.63
Limestone Aggregate	0.60
Light-Weight Aggregate	0.75



1 psi = 6.895 kPa

Fig 6.4. Concrete-tensile strength as a function of flexural strength (Ref 17).

any inferences made from outside these limits should use f'_t values calculated from split-tensile test results.

(6) Final Maximum Temperature Variation. The value of maximum temperature drop (ΔT_F) to be used by the designer should be calculated by subtracting the lowest temperature likely to be reached in the first year following construction from the maximum temperature likely to occur during placement of the slab. Values should be obtained from climatological data (Ref 20) for the relevant locality. Preferably, values of maximum (summer) and minimum (winter) annual temperatures at the 95 percent significance level should be calculated based on the distributions of these maximum amd minimum temperatures (respectively) over a significant period of time (for example, the previous 10 or 20 years).

(7) Wheel Load Stress. The tensile stress introduced into the bottom fiber of the slab by the application of the external, traffic (wheel) load can either be calculated using a discrete element approach (Refs 9 and 10), or, more simply, using Westergaard's equation for edge loading (Ref 16 and 21) as shown below.

If

P = applied wheel load,
h = concrete slab thickness,
a = radius of tire contact area,
k = modulus of subgrade reaction,
E = elastic modulus of concrete,
ℓ = radius of relative stiffness, and
μ = Poissons ratio,

then wheel load stress

$$\sigma_w = 0.572 \frac{P}{h^2} \left[4 \log_{10} \left(\frac{\ell}{b} \right) + 0.359 \right]$$

where

$$b = \sqrt{1.6a^2 + h^2} - 0.675h$$

for

$$a < 1.724h$$

and

$$b = a$$

for

$$a \geq 1.724h$$

and

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)k}} .$$

CHAPTER 7. RECOMMENDATIONS

(1) Having established values for input variables by the procedures discussed above, the design of the steel reinforcement for a CRC pavement can be performed by following the procedure outlined in Chapter 5 of Ref 2. Limiting criteria are also to be used in this process as outlined on page 39 of this report, and as detailed in Chapters 3, 4 and 5 of this report.

(2) The limiting criteria on crack spacing, crack width and steel stress discussed in this report represent part of the only totally rational, comprehensive, yet easy to use procedure for the design of CRCP which is available at this time. It is strongly recommended, therefore, that the entire procedure, as detailed in this report and the complementary CFHR Research Report 177-16, be incorporated into the Texas SDHPT Operations and Procedures Manual as soon as possible.

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APPENDIX

MODELING OF CRCP WITH THE DISCRETE ELEMENT COMPUTER PROGRAM

MODELING OF CRCP WITH THE DISCRETE ELEMENT COMPUTER PROGRAM

SIMULATION OF CRCP WITH DISCRETE ELEMENT MODEL

The use of discrete elements in a physical model of a rigid pavement not only gives us a straight-forward visualization of the problem, it allows freely discontinuous changes in load, bending stiffness, torsional stiffness and other parameters. The main advantages of using this model are:

(1) Freely varying loads can be applied to the slab.

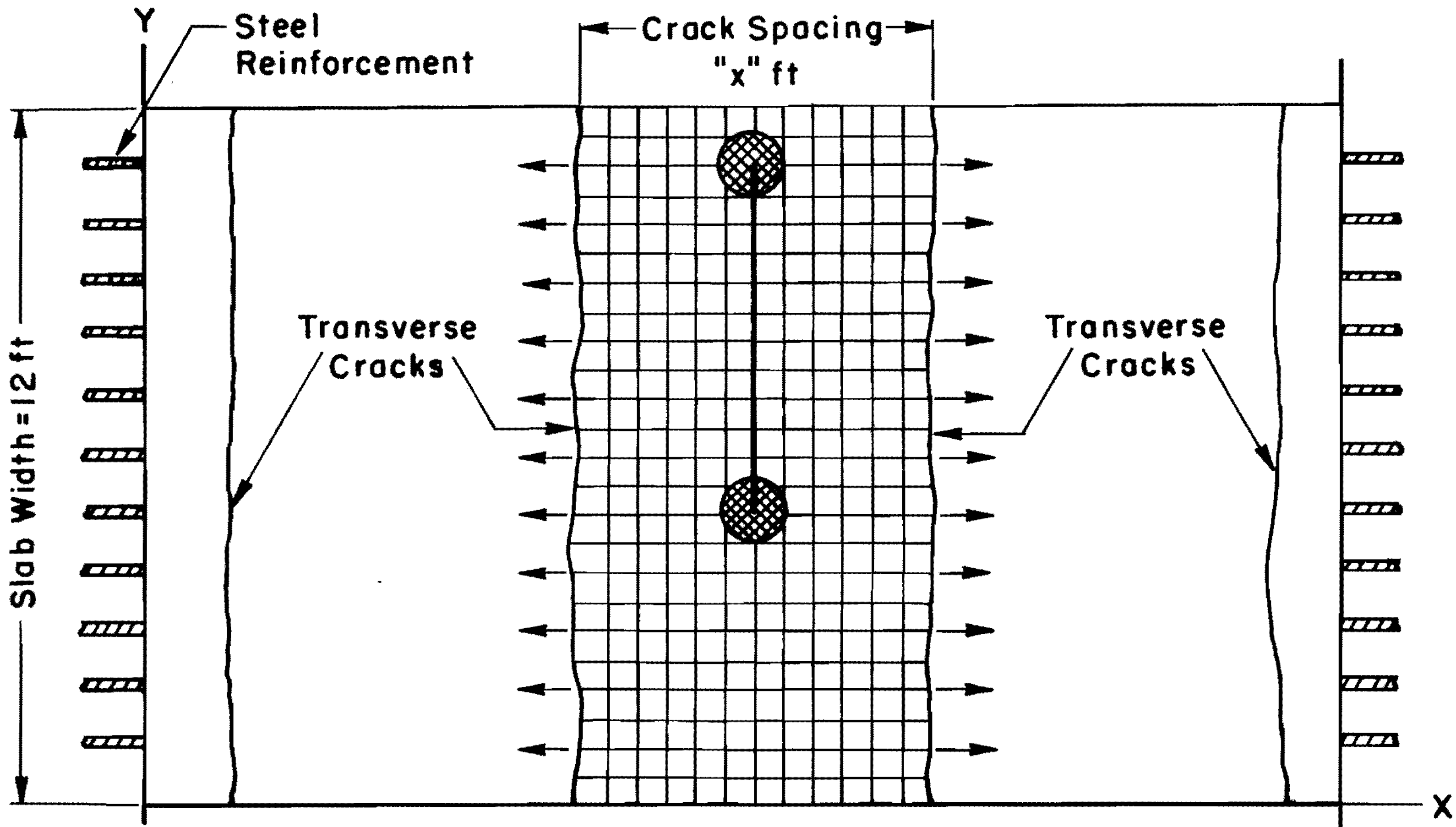
(2) Discontinuities in the slab can be handled in a routine manner and even partial discontinuities can be analyzed.

(3) Material properties of the slab can be varied from point to point.

(4) A variety of slab support conditions can be handled with the model, including non-uniform or continuous support.

(5) Horizontal forces from the action of the reinforcing bars on the continuous pavement can be modeled as in-plane axial thrusts in the discrete element model.

Figure A-1 shows a schematic modeling of CRCP using the discrete element model. The grid in the figure represents the number of increments in the X and Y direction. It covers a typical section of the continuous pavement with a crack spacing of "x" feet. The arrows along the cracks represent the horizontal force from the action of the reinforcing bars on the slab. A single wheel, single axle load is applied at mid-point between the adjacent cracks and near the edge of the pavement. Typical axle length and tire pressure corresponding to various wheel load types are used. The shaded circle in Fig A.1 represents the loading area. The input load is distributed on a per joint basis and apportioned at each joint by the contributory area loaded around each joint. The reason for choosing certain geometric configurations of loading positions, and values of material property inputs such as Modulus, Poisson's ratio for concrete, and subgrade support (k) is to use values for these variables that are thought to have minimum influence on the relative changes of σ_x and σ_y with respect to the crack spacing. This Appendix contains a detailed description of all the input variables used for the analysis. A full discussion may be obtained from Refs 22 and 23.



1 ft = 0.305 m

Fig A.1. Simulation of both vertical and horizontal load in continuously reinforced concrete pavement using discrete-element model.

SLAB BENDING STIFFNESS

For Poisson's ratio value of 0.25 and concrete modulus of 4×10^6 psi, the orthogonal slab stiffness, D^x and D^y can be solved by the following equation:

$$D^x = D^y = \frac{Et^3}{12(1 - \nu^2)} = \frac{4 \times 10^6 t^3}{12(1 - .25^2)}$$

and the twisting stiffness C^t can be described by the following equation:

$$C^t = \frac{Et^3}{12(1 + \nu)} = \frac{4 \times 10^6 t^3}{12(1 + .25)}$$

Table A.1 shows the bending and twisting stiffness solved for various slab thicknesses.

LOAD DISTRIBUTION

The input load is distributed on a per joint basis and may be apportioned at each joint by the contributory area loaded around each joint. Table A.2 shows the load distribution per increment area for a tire pressure of 80 psi. Notice that the increment widths vary with crack spacings, thus different distributed loads are assigned for different crack spacings. Support springs S are concentrated value inputs and are apportioned exactly like loads. A uniform subgrade modulus of 250 pci is used in this study.

SLAB AXIAL THRUST

The horizontal forces caused by volumetric changes of the CRC pavement can be modeled as axial thrust in the slab. The magnitude of the axial thrust is influenced by a combination of material properties used, and the environmental conditions. Forty-five thousand pounds per linear foot (22,500 lb/increment) axial force was assigned along the transverse direction of the pavement cross section. This force corresponds to approximately 70 ksi steel stress at the crack for 9 inch and 0.6 percent reinforced concrete pavement since

TABLE A.1. MESH STIFFNESS INPUTS FOR VARIOUS SLAB THICKNESSES IN THE DISCRETE ELEMENT MODEL

Slab Thickness (in.)	Discrete Element Model Inputs: Bending Stiffness, D^x, D^y (in.lb)	Twisting Stiffness C^t (in.lb)
7	121,960,000	91,450,000
8	183,000,000	136,500,000
9	259,200,000	194,400,000
10	359,600,000	266,700,000
11	473,200,000	354,900,000
12	614,400,000	460,800,000

1 in = 25.4 mm, 1 in. lb = 0.113 N

TABLE A.2. LOAD DISTRIBUTION PER INCREMENT AREA IN THE DISCRETE ELEMENT MODEL

Crack Spacing (feet)	Load Per Increment Area (lb) — Single Axle Load (lb): 4,000	18,000	24,000
3	166.7	750	1,000
6	333.3	1,500	2,000
9	1,000	4,500	6,000
12	1,000	4,500	6,000

1 foot = 0.305 m, 1 lbf = 4.448 N

$$\text{axial force/ft} = \sigma_s \times A_s$$

$$= \sigma_s (A_c \times .6/100)$$

$$= \sigma_s \times 9'' \times 12'' \times 0.006$$

Steel stress ranges from 45 ksi to 75 ksi; the 70 ksi chosen above is considered to be conservative.

PREDICTION OF BENDING STRESSES

The stresses in X-X and Y-Y directions are obtained with the moment-curvature equation in which

$$\sigma_{x-x} = \frac{M_{x-x} C}{I}$$

where

σ_{x-x} = stress in the x-x direction,

M_{x-x} = bending moment in x-x direction,

C = distance between the neutral axis and extreme fiber,

I = moment of inertia,

= $b_y h^3/12$,

h = slab thickness,

b_y = increment width in Y-Y direction.

Therefore,

$$\sigma_{x-x} = \frac{M_{x-x}\,(h/2)}{b_y h^3/12}$$

Similarly,

$$\sigma_{y-y} = \frac{M_{y-y}\,(h/2)}{b_y h^3/12}$$

Solutions of the discrete element model analysis for various slab thicknesses and wheel loads are summarized in Tables A.3 and A.4.

TABLE A.3. MAXIMUM BENDING MOMENT AND STRESSES IN X-X AND Y-Y DIRECTIONS FOR VARIOUS SLAB THICKNESSES

Slab Thickness (in.)	Crack Spacing (ft)	Increment width in x-x Direction, b_x (in.)	Increment width in Y-Y Direction, b_y (in.)	Bending Moment in X-X Direction, M_{x-x} (in. - lb)	Stress in X-X Direction $\sigma_{x-x} = \frac{6M_{x-x}}{b_y h^2}$ (psi)	Bending Moment in Y-Y Direction, M_{y-y} (in. - lb)	Stress in Y-Y Direction $\sigma_{y-y} = \frac{6M_{y-y}}{b_x h^2}$ (psi)
7	3	3	6	1355	27.7	2024	82.6
	6	6	6	2097	42.8	1707	34.8
	9	9	6	2656	54.2	1843	25.1
	12	12	6	2513	51.3	1771	18.1
8	3	3	6	1361	21.3	2076	64.9
	6	6	6	2138	33.4	1733	27.1
	9	9	6	2798	43.7	1875	19.5
	12	12	6	2718	42.5	1811	14.2
9	3	3	6	1365	16.9	2109	52.1
	6	6	6	2166	26.7	1747	21.6
	9	9	6	2903	35.8	1894	15.6
	12	12	6	2899	35.8	1834	11.3
10	3	3	6	1370	13.7	2132	42.6
	6	6	6	2188	21.9	1757	17.6
	9	9	6	2989	30.0	1907	12.7
	12	12	6	3063	30.6	1861	9.3
11	3	3	6	1370	11.3	2144	35.4
	6	6	6	2197	18.2	1758	14.5
	9	9	6	3040	25.1	1908	10.5
	12	12	6	3182	26.3	1870	7.7
12	3	3	6	1372	9.5	2153	29.9
	6	6	6	2206	15.3	1759	12.2
	9	9	6	3084	21.4	1910	8.8
	12	12	6	3288	22.8	1879	6.5

1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 in.lb. = 0.113 N.m,
1 psi = 6.895 kPa.

TABLE A.4. MAXIMUM BENDING MOMENT AND STRESSES IN X-X AND Y-Y DIRECTIONS FOR VARIOUS VERTICAL LOADS

Single Axle Load (lb)	Crack Spacing (ft)	Increment Width in X-X Direction, b_x (in.)	Increment Width in Y-Y Direction, b_y (in.)	Bending Moment in X-X Direction, M_{x-x} (in. - lb)	Stress in X-X Direction, $\sigma_{x-x} = \frac{6M_{x-x}}{81\ b_y}$ (psi)	Bending Moment in Y-Y Direction, M_{y-y}	Stress in Y-Y Direction, $\sigma_{y-y} = \frac{6M_{y-y}}{81\ b_x}$ (psi)
4,000	3	3	6	303.4	3.7	468.7	11.6
	6	6	6	481.2	5.9	388.2	4.8
	9	9	6	645.2	8.0	420.8	3.5
	12	12	6	644.2	8.0	408.5	3.4
18,000	3	3	6	1365.0	16.9	2109.0	52.1
	6	6	6	2166.0	26.7	1747.0	21.6
	9	9	6	2903.0	35.8	1894.0	15.6
	12	12	6	2899.0	35.8	1838.0	15.1
24,000	3	3	6	1820.0	22.5	2811.0	57.5
	6	6	6	2887.0	35.6	2329.0	28.8
	9	9	6	3871.0	47.8	2525.0	20.8
	12	12	6	3865.0	47.7	2451.0	20.2

1 lbf = 4.448N, 1 ft. = 0.305m, 1 in. = 25.4 mm, 1 in.lb = 0.113 N.m, 1 psi = 6.895 kPa.

THE AUTHORS

B. Frank McCullough is a Professor of Civil Engineering at The University of Texas at Austin. He has interests in pavements and pavement design and has developed design methods for continuously reinforced concrete pavements currently used by the State Department of Highways and Public Transportation, U.S. Steel Corporation, and others. He has over 250 published technical papers and reports to his credit.

James Ma earned his B.S. and M.S. degrees in Civil Engieering at The University of Texas at Austin in 1974 and 1977. In 1977, he joined the Austin Research Engineers, Inc. doing analysis of jointed reinforced concrete pavement, and jointed concrete pavement and pavement design for guideways. He is well versed in the latest analytical methods for rigid pavements including (1) finite elements, (2) discrete element methods, (3) JCP (FHWA method), and (4) CRCP methods developed in NCHRP Project 1-15. While pursuing his Master's degree, Mr. Ma worked as an Assistant Research Engineer at The University of Texas at Austin, during which time he prepared a report for the Center for Highway Research on improved computer methods for analysis of continuously reinforced concrete pavement.

Christopher S. Noble is an Assistant Professor of Civil Engineering at The University of Texas at Austin. He gained experience in the design of composite, prestressed concrete and steel box girder bridges as well as other reinforced and prestressed concrete structures with the New South Wales Public Works Department in Australia. His research interests include applications of probabilistics, statistics, and decision analysis to civil engineering in general and pavement design in particular. He is presently concerned with research in the areas of pavement design and rehabilitation management systems, economic modelling and design, and distress prediction models for continuously reinforced concrete pavements.