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ZERO-MAINTENANCE PAVEMENTS: RESULTS OF FIELD STUDIES ON THE PERFORMANCE REQUIREMENTS AND CAPABILITIES OF CONVENTIONAL PAVEMENT SYSTEMS

April 1976

Interim Report

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16. Abstract

This report presents results achieved during the first year's effort to determine performance requirements and capabilities of heavily trafficked conventional pavements to provide zero-maintenance performance. Five types of pavements are under consideration in this study: plain jointed concrete, reinforced jointed concrete, continuously reinforced concrete, flexible, and composite pavements. An extensive field survey of over 60 pavements carrying high traffic volumes was conducted and comprehensive analytical analyses were performed. Results presented herein include (1) an evaluation of the adequacy of commonly used design procedures to provide maintenance-free performance, (2) the identification of types of major distress, their causes, and maintenance performed for the five pavement types under consideration, (3) limiting design criteria for zero-maintenance design, and (4) a determination of the maximum maintenance-free lives of each pavement subjected to different environments. Recommendations are also made as to the most promising pavement types for consideration in zero-maintenance design. A wealth of basic data has been accumulated for the development of zero-maintenance pavement design procedures during the next phase of the project.

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PREFACE

This investigation was conducted at the Transportation Research Laboratory, Department of Civil Engineering, University of Illinois at Urbana-Champaign. The project is sponsored by the Federal Highway Administration, U. S. Department of Transportation. Dr. Michael I. Darter and Dr. Ernest J. Barenberg served as co-principal investigators of the project.

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LIST OF ABBREVIATIONS

Pavement Section

AC	Asphalt Concrete
ATB	Asphalt Treated Base
BTS	Bituminous Treated Sand
CJS	Contraction Joint Spacing
COMP	Composite Pavement
CRCP	Continuously Reinforced Concrete Pavement
CTB	Cement Treated Base
EJS	Expansion Joint Spacing
FLEX	Flexible Pavement
GR.B.	Granular Base or Subbase
JCP	Jointed Concrete Pavement (Non-reinforced)
JRCP	Jointed Reinforced Concrete Pavement
LTB	Lime Treated Base
LTD	Load Transfer Device
MAC	Asphalt Penetration Macadam
PCC	Portland Cement Concrete
SM	Select Material
SS-C	Scarified Subgrade - Compacted to 100% Proctor

- -

Maintenance

CF	Crack Filling
CS	Crack Sealing
DCP	"D" Crack Patching
JF	Joint Filling
JS	Joint Sealing

LIST OF ABBREVIATIONS (Continued)

Maintenance (Continued)

P	Patching with AC or PCC
SJ	Slab Jacking
SH	Shoulder Repair or Replacement
SR	Slab Replacement

Distresses

JF	Joint Faulting
JS	Joint Spalling
CF	Crack Faulting
CS	Crack Spalling
TC	Transverse Cracking
B	Blowups
CC	Corner Cracking
LC	Long Cracking
D	"D" Cracking
PU	Pumping
DC	Diagonal Cracking

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

MULTIPLY	BY	TO OBTAIN
inches	2.54	centimeters
feet	0.3048	meters
square inches	6.4516	square centimeters
square yards	0.83612736	square meters
knots	0.5144444	meters per second
pounds	0.45359237	kilograms
kips	0.45359237	metric tons
pounds per cubic foot	16.018489	kilograms per cubic meter
pounds	4.448222	newtons
kips	4.448222	kilonewtons
pounds per square inch	6.894757	kilopascals
pounds per cubic inch	2.7144712	kilopascals per centimeters
gallons (U. S. liquid)	3.785412	cubic decimeters
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F-32). To obtain Kelvin (K) readings, used: K = (5/9)(F-32) + 273.15.

CHAPTER 1

INTRODUCTION

This report presents results obtained during the first year effort on the project entitled "Zero-Maintenance Pavement: Performance Requirements and Capabilities of Conventional Pavements." The major objective of this report is to document results obtained concerning the following:

1. Types and causes of distress and maintenance applied on heavily trafficked pavements;
2. Adequacy of commonly used design procedures to obtain maintenance-free pavements;
3. Limiting criteria for use in designing maintenance-free pavements, and
4. Maximum maintenance-free lives of conventional pavements.

These results serve as a basis for the development of design procedures for high traffic volume, zero-maintenance pavements during the next phase of the project.

Many highways in urban and suburban areas are being subjected to heavy traffic volumes which cause rapid deterioration and premature failure of pavements. Hence, considerable maintenance is required, but scheduling of remedial and preventive maintenance is almost impossible without closing lanes and producing massive traffic jams, accidents, and delays to the traveling public. Often routine maintenance is completely neglected, thus causing even more accelerated deterioration of pavements. When maintenance is performed, it is usually during off-peak hours or at nights.

Under such conditions, repairs are often rushed and/or performed with inadequate equipment and with inadequate room to maneuver. The repairs are often inefficiently done because of logistics, traffic interference, and workers toiling under hazardous conditions. The cost of traffic control is a major item in any maintenance budget, but especially under the conditions described, and the cost of delays to the motorist because of lane closure or detours from the expressway for maintenance operations accumulates at a fantastic rate.

The fundamental question which underlies this research is "how to design and build conventional pavements (or optimized conventional pavements in which inherent weaknesses are eliminated) to serve exceptionally heavy traffic without requiring maintenance and providing satisfactory 'rideability' for twenty to forty years?"

The term "zero-maintenance" as used in this study is restricted to the structural adequacy of the pavement system. Activities such as mowing, snow removal, guardrail repair, sweeping, opening of drains, slope stability, channel maintenance, bridge and bridge approach repair, maintenance of signs/lighting, striping, and providing skid resistance are not related to the structural adequacy of the pavement system, and therefore are outside the scope of this study. Also, geometric obsolescence, subsequent widening to increase capacity, and wear from studded tires are not of concern in this study.

The research approach to develop basic data and information from which the zero-maintenance design procedures will be ultimately developed relies heavily upon information gained from extensive field visits, analytical evaluations, and prior research findings.

1. Field visits were made to 11 state highway and transportation departments and other agencies where many heavily trafficked pavements were surveyed, extensive data collected, and interviews held with engineers responsible for the design, construction, and maintenance of the pavements.

2. Analytical studies were conducted using the data and information obtained from the field studies, previous research results, and theoretical and empirical pavement models.

3. The field surveys and analytical studies were used to determine the adequacy of existing design procedures, types and causes of distress, maintenance performed, limiting design criteria, and maximum maintenance-free lives for five types of pavements: plain jointed concrete, reinforced jointed concrete, continuously reinforced concrete, flexible, and composite pavements.

Results of the first year's effort provide a wealth of information on many aspects of high traffic volume pavement design, construction, and maintenance. An identification of the significant distress types that require maintenance along with their causes is presented. Currently used design procedures are found to be deficient in many respects and do not provide maintenance-free pavements for periods greater than 10 to 20 years. Limiting design criteria in terms of terminal serviceability and allowable amounts of particular distresses were determined. The maximum maintenance-free life of typical conventional pavements under heavy traffic volumes were found to range from 5 to 25 years with different pavement types showing varying maintenance-free lives in different environmental regions. An extensive amount of information was obtained to assist in the development of reliable design procedures during the next phase of the study.

The results of this study are presented in the following sequence:

Chapter 2 - Describes the field survey and project data obtained.

Chapter 3 - Identifies types of distress, causes, and maintenance performed on heavily trafficked pavements.

Chapter 4 - Evaluates commonly used design procedures for their applicability to the design of zero-maintenance pavements.

Chapter 5 - Describes limiting criteria for design of zero-maintenance pavements.

Chapter 6 - Provides estimates for maximum maintenance-free life of conventional high traffic volume pavements.

Chapter 7 - Summarizes results and gives recommendations for most promising pavement types.

CHAPTER 2

BRIEF DESCRIPTION OF FIELD SURVEY

2.1 GENERAL

A field survey was conducted on 68 in-service, heavily trafficked pavements located in 13 states. The results from this survey and interviews with state highway and transportation personnel provided considerable information which will be extremely helpful in the successful accomplishment of various objectives of this project. Hence, a brief description of the projects surveyed and the data collected is presented and will be referred to throughout the report.

The purposes of the field survey on the in-service pavements were as follows:

1. To obtain information about their performance characteristics including types, amounts, severity, and causes of distress, actual maintenance-free life, pavement structure composition, and traffic loading.

2. To obtain information about the structural maintenance practices including types and amounts of maintenance, and types and severity of distress before such maintenance is applied.

3. To obtain current views and design practices of state highway and transportation department engineers with regard to design, construction, and maintenance of heavily trafficked pavements.

4. To document the performance of selected, in-service, heavily trafficked pavements. Such documentation is the basis for detailed

analyses, including

a. Determination of the adequacy of commonly used design procedures,

b. Determination of limiting design criteria for designing zero-maintenance pavements, and

c. Determination of the maximum maintenance-free lives of conventional pavements, and thus verifying the existing models and developing new predictive models for designing maintenance-free pavements.

2.2 PROCEDURE

Contacts were established with 13 state highway and transportation departments, the Ontario Ministry of Transportation and Communications, and three turnpike commissions, as follows:

1. State highway or transportation departments

   Arizona Department of Transportation

   California Department of Transportation

   Colorado Department of Highways

   Georgia Department of Transportation

   Illinois Department of Transportation

   Minnesota Department of Highways

   Michigan Department of Highways and Transportation

   New Hampshire Department of Public Works and Highways

   New Jersey Department of Transportation

   New York Department of Transportation

Texas Highway Department

Utah Department of Highways

Washington Department of Highways

2. Other agencies

Ontario Ministry of Transportation and Communications

New Jersey Turnpike Commission

Dallas-Fort Worth Turnpike Commission

New York Thruway Authority

Attempts were made to select projects for each pavement type in a variety of climatic regions so that each pavement type could be evaluated over a variety of climates. The general climatic categories are defined as follows:

1. Wet/Dry: A wet region is defined as a region where the annual percipitation equals or exceeds the potential evapotranspiration of moisture (143). A dry region is where the reverse is true.

2. Frost/No-Frost: A frost region is defined as a region where significant freezing temperatures results in pavement frost heave or spring thaw damage. The extent of frost action has been defined by the Corps of Engineers (28) as a Freezing Index. Regions which have a freezing index of 100 degree days or more are categorized as frost areas, those with less degree days are designated as no-frost areas.

A summary of annual precipitation, annual potential evapotranspiration, and freezing indices for the several regions is shown in Table 2.1.

Table 2.1. Annual Precipitation, Evapotranspiration, and Freezing Index of Field Project Regions.

Project Region	Annual[1] Precipitation (ins.)	Precipitation[2] Minus Evaporation (ins.)	Freezing[3] Index
I. WET/FREEZE AREA			
A. Manchester, NH	34	+15*	750
B. Minneapolis, MN	22	0	1750
C. Ottawa, IL	30	+ 4	700
D. Chicago, IL	30	+ 5	700
E. Detroit, MI	29	+ 4	500
F. Toronto, ONT	31	+ 8	750
G. Albany, NY	32	+15	600
H. Yardville, NJ	39	+10	100
II. DRY/FREEZE AREA			
A. Salt Lake City, UT	13	-20**	250
B. Denver, CO	17	- 8	300
III. WET/NO FREEZE AREA			
A. Atlanta, GA	43	+12	0
B. Seattle, WA	30	+30	0
C. Houston, TX	44	0	0
IV. DRY/NO FREEZE AREA			
A. San Francisco, CA	17	-10	0
B. Los Angeles, CA	10	-30	0
C. Phoenix, AZ	5	-65	0
D. Fort Worth, TX	22	-30	0
E. Dallas, TX	28	-24	0

* plus means precipitation is more than evaporation
** minus means precipitation is less than evaporation

1. Ref. 143
2. Ref. 143
3. Ref. 28

Various types of pavements are included in the project survey, including:

1. Flexible (FLEX)
2. Composite (an asphalt concrete surface over portland cement concrete base) (COMP)
3. Continuously Reinforced Concrete (CRC)
4. Plain Non-Reinforced Jointed Concrete (JCP)
5. Reinforced Jointed Concrete (JRCP)

The symbols FLEX, COMP, CRC, JCP, and JRCP are used in this report to designate each type of pavement. A complete list of abbreviations and symbols is given at the beginning of the report.

The goal of locating one or more projects in each major climatic region was only partially successful. A complete summary showing all projects, the city or state where they are located, and the general climate for each is given in Figure 2.1.

Projects were selected within the various climatic regions using the following guidelines:

1. Age - Longevity was the most important factor which severely limited the number of available projects. In general, only those projects which were greater than 10 years old were selected for this study.

2. Maintenance - Only those projects which have not been overlayed or seal coated were selected. Past routine maintenance was not a consideration and two types of pavements were selected, those which have been maintenance-free and pavements that have required maintenance.

Moisture Condition / Frost Condition / Pavement Type	WET		DRY	
	F-T	No F-T	F-T	No F-T
FLEX	NH, IL (4) MN (4), NJ	GA(2),WA(2)	UT	AZ,CA-LA CA-SF (2)
COMP	MI, CAN (3) IL , NJ	WA		
JCP	IL (4), NJ	WA, GA	UT CO (2)	AZ, CA-LA CA-SF, TX-D
JRCP	IL(5),CAN(2) IL-CHIC ,NJ NY(3) MI(2)	TX-HOU		TX-D
CRCP	IL (2) MN, MI, NJ	TX-HOU		TX-D/FW (5)

Abbreviations given are the official U. S. Post Office abbreviations for the States. Letters after the state symbol represent a specific city where a state has cities in 2 areas, i.e., CA-SF is San Francisco, California. The numbers in parentheses are the number of projects in that category if more than one.

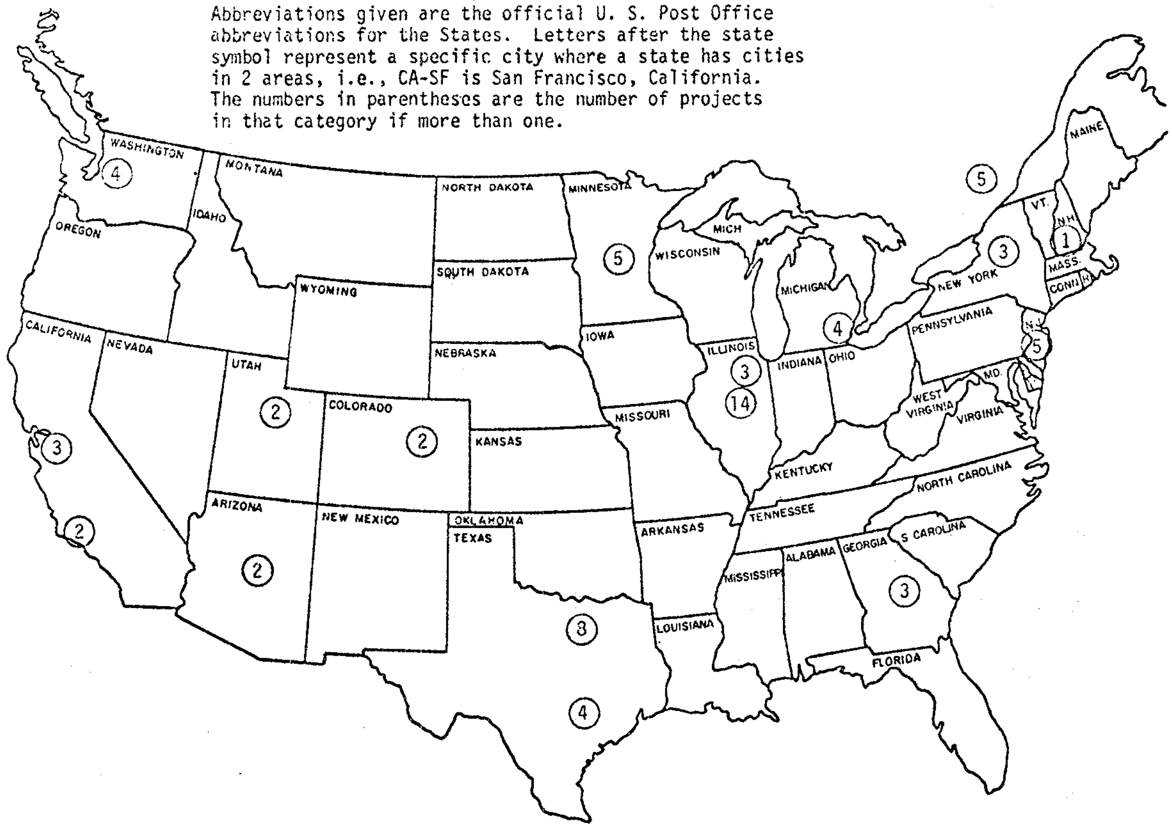


Figure 2.1. Summary of projects included in field survey.

3. Traffic - Only those pavements that have relatively high traffic volume were given preference in selection. In a few cases, projects with relatively low traffic volume were also selected for special reasons (i.e., high truck percentage or an unusual type of design). Most of the projects, however, are high volume urban and rural freeways.

4. Pavement Type - Each area visited usually had one predominant type of pavement. One or two pavements of such type were always selected, and in addition, where possible, other types of pavements were also selected in an attempt to complete the experiment factorial. Only partial success was achieved, however, due to the limited funding available for the field work and the large staff effort required to survey the pavements and obtain necessary data.

2.3 SURVEY PROCEDURE

Discussions with agency personnel and condition surveys were conducted in each area visited as described below:

1. Detailed discussions were held with administration, design, construction, and maintenance personnel of the agency during the visit, whenever possible. Subjective information was thus gathered for pavement performance, types of distress, recommended design practices, critical design limits, and construction and maintenance practices in the area. Some of this information is contained in subsequent chapters of this report; the additional information will be documented in the final project report.

2. Surveys were conducted on several projects in each area, some of which were later selected for inclusion in the project analyses. Attempts were made to collect the data from each project as follows:

a. A general surface condition survey was conducted over a typical portion of the included pavement about 2000 ft in length. The outer traffic lane was surveyed in most cases.

b. Present serviceability rating was estimated by the project staff member(s), or when available, the Present Serviceability Index was obtained from the agency.

c. A general drainage evaluation was made.

d. The pavement was photographed both with 35 mm camera and with a Super-8 movie camera.

e. The date of opening to traffic and the original pavement cross section were obtained from the agency. Any available material data for various layers and the subgrade measured during or after the construction of the project were also obtained.

f. Traffic data including ADT, percent trucks, axle load distributions, lane distributions, directional distribution, and average axles per truck were obtained from the agency.

g. Climatic data were obtained from published Weather Bureau and other sources.

h. Opinions of the local engineers as to the reasons for pavement distress, if any, were obtained.

i. Previous maintenance performed on the section was determined.

Attempts were made to obtain the above information for each project. All agencies were very cooperative and helpful in collecting and providing the requested data, but some data were not available. Considerable time was spent by the project staff and by each agency in collecting this data which are of tremendous benefit to the project.

Categorized summaries of basic project information are shown in Tables 2.2, 2.3, 2.4, 2.5, and 2.6 for each pavement type.

The following statements are relative to the sample of projects that were surveyed and analyzed in this study:

(1) The 68 projects represent only a small sample of the hundreds of heavily travelled pavements in the U. S. and hence the sample is probably biased in various ways (i.e. it does not completely represent the performance of all heavily travelled pavements). It is difficult to clearly see exactly what the biases might be since the projects represent such a widely diverse selection. An attempt was made to include both "successes" and "failures." Approximately 57 percent of the projects had shown maintenance-free performance and 43 percent had received structural maintenance (i.e. patching, crack filling, slab replacement, overlay, etc.).

(2) Many of the projects selected are representative of several others of identical design and performance that are located in the given area. For example, JRCP-5 in New Jersey is one of many projects of the same design and performance that have been constructed for many years. JCP-7 in Los Angeles is typical of many pavements in that area that have been constructed for many years. The eight sections denoted as JCP-1, 2, 3, 4 and JRCP-6, 7, 8, 9 are actually representative of about 50 sections on I-80 in Illinois. Other projects that generally represent others of similar design and performance include JRCP-11, 12, 13, 14, and 18; JCP-5, 6, 7, 9, 10, 13, and 14; CRCP-3, 4, 5, 6, 7, 8, 9, and 10; FLEX-1, 3, 4, 5, 6, 11, 12, 13, 17, and 19.

(3) Several projects are essentially "one of a kind" that were constructed in an area. For example, FLEX-2, the New Jersey Turnpike (southern portion), is the only known asphalt penetration McCadam type pavement in the area under heavy traffic. COMP-6 in New Jersey is the only composite of this type constructed, and FLEX-10 is the only flexible freeway pavement constructed in Los Angeles. Other projects that fall into this category include: JRCP-4 and 17; JCP-8 and 11; CRCP-2, 11, and 12; FLEX-2, 14, 15, 16, and 18; COMP-1, 4, 5, 6, and 7.

(4) Several pavements were selected that were part of road tests under regular mixed traffic. These sections include JRCP-6, 7, 8, 9, and 10; JCP-1, 2, 3, and 4; CRCP-12; FLEX-7, 8, 9, and 10; and COMP-2, 3, and 5.

(5) Whenever possible, examples of both good and poor performance projects that had roughly the same design were included. For example, JRCP-13 and 14 in Detroit were of essentially similar design but gave different performance. Other projects are JCP-13 and 14, CRCP-7 and 8, and CRCP 9 and 10.

(6) Pavements that showed distress within a few years were included (such as JRCP-18, JCP-13, CRCP-1, FLEX-10, and COMP-2), as well as pavements that performed over more than 14 years without any structural distress (such as JRCP-5, JCP-7, CRCP-11, FLEX-2, COMP-4).

These results indicate that the 68 projects represent a wide variety of conditions that seem not to be too heavily based in any given direction.

Table 2.2 Summary of Information for Jointed Reinforced Concrete Pavement (JRCP) Projects Included in the Field Survey

Project No.	City, State	Route	Date Opened	ADT		No. of Lanes	Million 18-kip ESAL *	Pavement Section					Maintenance** Performed
				Average over life	1974			Surface	Base	Subbase	Subbase	Subgrade	
JRCP 1	Houston, Texas	I-45 Gulf Freeway	1950	71,700	95,100	6	3.79	9" pcc EJS:30'6"	6" GR.B.	6" SS-C	--	--	J.F. S.H.
JRCP 2	Houston, Texas	I-45	1952	14,375	23,650	4	1.76	9" pcc EJS: 45' CJS: 15'	6" Sand & Shell	6" L.T.B.	--	--	J.F. S.H.
JRCP 3	Dallas, Texas	US 77 Hines Blvd.	1941	17,940	21,000	4	7.04	8"-6"-8" pcc EJS: 100' CJS: 20'	6" GR.B.	--	--	--	J.F. P. S.H.
JRCP 4	Dallas, Texas	DFW Turnpike	1957	51,000	86,960	6	5.78	10" pcc CJS: 30'	6" GR.B	12" S.M.	--	--	J.F. S.R. S.J. S.H C.F.
JRCP 5	Yardville, New Jersey	Rt. 130	1951	13,200	17,100	4	26.57	10" pcc EJS: 78'	5" GR.B	7-19" GR.B.	--	--	J.F. S.H.
JRCP 6	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	9.54	8" pcc CJS: 40'	6" GR.B	--	--	A-6	J.F. P.
JRCP 7	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	16.85	9.5" pcc CJS: 40'	6" GR.B	--	--	A-6	J.F. P.
JRCP 8	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	17.42	11" pcc CJS: 40'	6" GR.B	--	--	A-6	J.F. P.
JRCP 9	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	18.54	12½" pcc CJS: 40'	6" GR.B	--	--	A-6	J.F.
JRCP 10	Ottawa, Ill.	I-80	1962	11,100	18,900	4	7.73	10" pcc CJS: 40'	8.5" GR.B	--	--	A-6	J.F.

*Heaviest traveled lane, one direction

(continued)

Table 2.2 JRCP Continued

Project No.	City, State	Route	Date Opened	ADT Average over life	ADT 1974	No. of Lanes	Million 18-kip ESAL *	Pavement Section Surface	Base	Subbase	Subbase	Subgrade	Maintenance Performed
JRCP 11	Toronto, Canada	Hwy 401	1964	Note 1	187,000	12	24.5	9" pcc CJS: 56'	9" GR.B.	--	--	Clay	None
JRCP 12	Toronto, Canada	Hwy 401	1959	37,000	63,500	4	10.5	9" pcc CJS: 99'	6" GR.B	6" GR.B	--	Clay	J.F. P.
JRCP 13	Detroit, Michigan	Mich. Hwy. 39	1961	56,500	73,000	6	4.5	10" pcc CJS: 100	3" GR.B	10" GR.B	--	--	P, S.R., C.F.
JRCP 14	Detroit, Michigan	I-75	1964	40,000	66,250	6	9.1	10" pcc CJS: 100' EJS: 400'	3" GR.B.	10" GR.B.	--	--	None
JRCP 15	Albany, New York	I-90/ I-87	1954	17,750	28,875	4	6.98	9" pcc EJS: 100'	12" GR.B.	--	--	--	J.F. S.H. P. S.R.
JRCP 16	Syracuse New York	I-81	1955	41,500	80,000	4	9.13	9" pcc EJS: 60'-10"	12" GR.B.	--	--	--	J.F. S.H. P. S.J. S.R. C.F.
JRCP 17	Albany, New York	I-87	1960	32,375	46,250	6	4.67	9" pcc EJS: 100'	12" GR.B.	--	--	--	J.F.
JRCP 18	Chicago, Illinois	I-55	1960	44,000	11,400	6	11.54	10" pcc CJS: 100'	6" GR.B	--	--	--	J.F. C.F. P. S.H.

*Heaviest traveled lane, one direction

Note 1: Average ADT: 11,200 in Design Lane
1974 ADT: 14,500 in Design Lane

Table 2.3 Summary of Information for Plain Jointed Concrete Pavement (JCP) Projects Included in the Field Survey

Project No.	City, State	Route	Date Opened	ADT Average over life	ADT 1974	No. of Lanes	Million 18-kip ESAL *	Pavement Section: Surface	Base	Subbase	Subbase	Subgrade	Maintenance** Performed
JCP 1	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	9.54	8" pcc CJS: 15'	6" GR.B.	--	--	A-6	J.F. P.
JCP 2	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	16.85	9½" pcc CJS: 15'	6" GR.B.	--	--	A-6	J.F
JCP 3	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	17.42	11" pcc CJS: 15'	6" GR.B.	--	--	A-6	J.F.
JCP 4	Ottawa, Ill. AASHO Test Site	I-80	Oct. 1958	11,100	18,900	4	18.54	12-1/2" pcc CJS: 15'	6" GR.B.	--	--	A-6	J.F.
JCP 5	Tacoma, Wash.	I-5	1962	36,750	57,250	6 in '62 8 in '71	4.32	9" pcc CJS: 15'	2" GR.B.	7" GR.B	--	--	J.F.
JCP 6	San Francisco, California	I-80	1955	35,750	55,000	6	12.4	8" pcc CJS: 15'	4" C.T.B.	12" GR.B.	--	--	J.F. P. C.F.
JCP 7	Los Angeles, California	I-5	1954-1958	67,500		4	32.7	8" pcc CJS: 15'	4" C.T.B.	4" GR.B.	8" Sand	--	None
			1959-1974	113,500	129,000	6							
JCP 8	Dallas, Texas	I-35	Dec. 1960	12,000	19,300	4	2.8	10" pcc CJS: 15'	6" GR.B.	4" A.C. old Pavement	--	Austin Chalk	J.F.
JCP 9	Salt Lake City, Utah	I-15	1965	76,500	102,000	8	4.08	9" pcc CJS: 12-18' (Skewed)	4" C.T.B.	2" GR.B.	--	--	None

*Heaviest traveled lane, one direction

(Continued)

Table 2.3 JCP Continued

Project No.	City, State	Route	Date Opened	ADT		No. of Lanes	Million 18-kip ESAL *	Pavement Section					Maintenance Performed
				Average over life	1974			Surface	Base	Subbase	Subbase	Subgrade	
JCP 10	Phoenix, Arizona	I-17	1960	41,500	75,000	6	27.59	9" pcc CJS: 15' Skewed	4" GR.B.	6" S.M.	--	Sand-Gravel	S.H
JCP 11	New Jersey	Rt. 130	1949	12,850	17,100	4	29.38	10" pcc CJS: 15'	12" GR.B.	6" S.M.	--	--	S.H.
JCP 12	Atlanta, Georgia	I-57 & I-85	1950	100,000	170,000	6	17.0	8" pcc CJS: 30' EJS: 600'	6" GR.B.			--	J.F.
JCP 13	Denver, Colorado	I-70	1964	37,250	50,800	6	8.66	8" pcc CJS:12-19 Skewed	6" GR.B	14" GR.B		Sand-Gravel	C.F.
JCP 14	Denver, Colorado	I-25	1959	47,500	75,000	6	13.08	8" pcc CJS: 15' Skewed	6" GR.B.			A-1-b(0)	None

*Heaviest traveled lane, one direction

Table 2.4. Summary of Information on Continuously Reinforced Concrete Pavement (CRCP) Projects Included in the Field Survey

Project No.	City, State	Route	Date Opened	ADT Average over life	ADT 1974	No. of Lanes	Million 18-kip ESAL *	Pavement Section Surface	Base	Subbase	Subbase	Subgrade	Maintenance Performed **
CRCP 1	Chicago, Illinois	I-94 Dan Ryan Expway	Dec. 1962	194,500	266,300	12	35.66 to 1971	8" CRC Rehab. 1971	6" GR.B.	--	--	Clay	P. S.H.
CRCP 2	Chicago, Ill. (Stevenson Exp.)	I-55	Oct. 1964	55,000	71,000	6	10.30	10" CRC	6" GR.B	--	--	Clay	S.H.
CRCP 3	East St. Paul, Minnesota	I-694	1969	6,630	13,000	4	1.85	9" CRC	6" GR.B.	--	--	--	None
CRCP 4	Detroit, Michigan	I-75	1965	32,750	40,500	6	7.71	9" CRC	4" GR.B	10" Sand	--	--	None
CRCP 5	Houston, Texas	I-610	1966	119,500	180,000	8	6.24	8" CRC	6" CTB	--	--	--	None
CRCP 6	Houston, Texas	I-45	1962	85,700	120,000	8	1.96	8" CRC	6" GR.B.	--	--	--	P
CRCP 7	Dallas, Texas	I-35E	Feb. 1962	88,000	145,000	8	2.57	8" CRC	6" GR.B	--	--	12" Embankment with P.I. 6-12	None
CRCP 8	Dallas,	U.S. 183	May 1961	45,400	60,000	6	3.98	8" CRC	6" GR.B.	12" S.M.	--	--	P S.H.
CRCP 9	Fort Worth, Texas	I-20 (old I-820)	1961	17,000	32,000	4	3.25	8" CRC	6" L.T.B.	--	--	--	None
CRCP 10	Fort Worth, Texas	I-20	1966	14,300	18,500	4	2.42	8" CRC	6" L.T.B.	--	--	--	P
CRCP 11	Fort Worth, Texas	I-30/ I-20	1951	61,100	80,000	6	5.96	9" CRC 1/2" AC 1970	8" GR.B.	6" SS-C	--	--	None
CRCP 12	New Jersey	Rt. 130	1947	12,500	17,100	4	31.99	10" CRC	12" GR.B.	--	--	--	P S.H.

* Heaviest traveled lane, one direction.

Table 2.5 Summary of Information for Flexible Pavement (FLEX) Projects Included in the Field Survey.

Project No.	City, State	Route	Date Opened	ADT Average over life	ADT 1974	No. of Lanes	Million 18-kip ESAL *	Pavement Section Surface	Base	Subbase	Subbase	Subgrade	Maintenance Performed
FLEX 1	Manchester, New Hampshire	I-93	1960	19,800	35,200	4	7.26	3" A.C.	2-1/2" A.C.	12" GR.B.	12" GR.B.	--	C.F.
FLEX 2	New Jersey	N.J. Turnpike	Nov. 1951	21,600	32,800	4	12.42	4-1/2" A.C.	7'1/2" MAC	6-1/2" GR.B.	5-1/2" S.M.	12" S.M. / Sand	P. S.H.
FLEX 3	West St. Paul, Minnesota	TS #21 Hwy. 55	1961	9,500	15,600	4	2.01	4" A.C.	6" GR.B.	12-1/2" GR.B.	--	A-6 Clayey-Loam	C.F. P.
FLEX 4	West St. Paul, Minnesota	TS #22 Hwy. 55	1961	9,500	15,600	4	1.91	4" A.C.	3" A.T.B.	6" GR.B.	16" GR.B.	A-4 (CL)	C.F.
FLEX 5	North St. Paul, Minnesota	TS #23 Rt. 36	1961	8,000	12,100	4	1.69	5-1/2" A.C.	1-1/2" A.T.B.	9" GR.B.	12" GR.B.	A-2-4 Sandy Loam	C.F.
FLEX 6	South St. Paul, Minnesota	TS #109 Hwy.101	1966	10,200	14,100	4	5.21	1-1/2" A.C.	5" A.T.B.	3" B.T.S.	--	A-3	C.F.
FLEX 7	Ottawa, Illinois	I-80	1962	11,000	18,900	4	4.70	4-1/2" A.C.	8-1/2" GR.B.	23" GR.B.	--	A-6	P.
FLEX 8	Ottawa, Illinois	I-80	1962	11,100	18,900	4	4.82	4-1/2" A.C.	8" A.T.B.	4" GR.B.	--	A-6	None
FLEX 9	Ottawa, Illinois	I-80	1962	11,000	18,900	4	4.72	4-1/2" A.C.	10" A.T.B.	4" GR.B.	--	A-6	None
FLEX 10	Ottawa, Illinois	I-80	1962	11,100	18,900	4	5.59	4-1/2" A.C.	12" C.T.B.	4" GR.B.	--	A-6	P.

* Heaviest traveled lane, one direction.

(continued)

Table 2.5 FLEX Continued.

Project No.	City, State	Route	Date Opened	ADT		No. of Lanes	Million 18-kip ESAL *	Pavement Section					Maintenance Performed
				Average over life	1974			Surface	Base	Subbase	Subbase	Subgrade	
FLEX 11	Seattle, Wash.	I-90	1960	10,400	13,400	4	1.66	7" A.C.	3" GR.B.	18" GR.B.	--	Laminated Silts, Sands and Clays	None
FLEX 12	Seattle, Wash.	SR 101	1963	2,450	2,925	2	0.35	8" A.C.	2" GR.B.	Scarified old Pavt.	--	Gravels, Silts, Clays	None
FLEX 13	South San-Francisco, California	SCL 101	1955	21,250	28,000	4	8.81	5" A.C.	8" CTB	8" GR.B.	-- --	-- --	P. C.F.
FLEX 14	San Francisco, California	CC-4	1965	19,000	28,000	4	2.27	7" A.C.	8" GR.B.	15" GR.B.	--	--	None
FLEX 15	Los Angeles, California	I-405	1965	69,000	138,500	8	3.70	7" A.C.	8" CTB	10" GR.B.	--	--	None
FLEX 16	Salt Lake City, Utah	I-80	1966	21,000	19,125	4	2.75	4-1/2" A.C.	5" CTB	4" GR.B.	--	--	None
FLEX 17	Pheonix, Arizona	I-17	1964	11,100	17,000	4	4.27	3" A.C.	6" GR.B.	10" S.M.	--	--	C.F. P.
FLEX 18	Atlanta, Georgia	Rt. 78	1951	14,200	11,000	4	4.02	11" A.C.	10" GR.B.	--		Clay	None
FLEX 19	Atlanta, Georgia	Rt. 78	1968	14,000	20,000	4	2.38	3-1/2" A.C.	9" CTB	7-1/2" GR.B.	--	Clay	None

* Heaviest traveled lane, one direction.

Table 2.6. Summary of Information for Composite Pavements (COMP) Projects Included in the Field Survey.

Project No.	City, State	Route	Date Opened	ADT		No. of Lanes	Million 18-kip ESAL *	Pavement Section					Maintenance Performed
				Average over life	1974			Surface	Base	Subbase	Subbase	Subgrade	
COMP 1	Detroit, Michigan	I-94	1965	13,500	102,000	6	7.73	11" A.C.	3" A.C. Exist.	9" PCC Exist. Conc. C.J.S.-30	--	--	S.H.
COMP 2	Toronto, Canada	Hwy. 401	1959	37,000	63,000	4	12.42	3 1/4" A.C.	8" PCC	6" GR. B.	6" GR. B.	Clay	C.F. P.
COMP 3	Toronto, Canada	Hwy. 401	1959	37,000	63,000	4	12.42	3 1/4" A.C.	8" PCC CJS= 100'	6" GR. B.	6" GR. B.	Clay	C.F. P.
COMP 4	Ontario, Canada	QEW 10	1960	66,000	87,500	6	27.93	6" A.C.	8 1/2" PCC	6" GR.B.	--	--	None
COMP 5	Ottawa, Illinois	I-80	1962	11,100	18,900	4	4.83	3" A.C.	8" PCC	6" GR. B.	--	A-6	None
COMP 6	New Jersey	Rt. 3	1963	88,500	96,500	8	8.54	3 1/2" A.C.	5" Stone Macadam	3" GR. B.	8" PCC C.J.S.-15'	6" GR.B. Silt, Clay, and Sand	None
COMP 7	Seattle, Washington	I-5	1966	35,000	43,500	6	1.19	4" A.C.	6" PCC	2" GR.B.	7" GR.B.	Glacial Till	None

*Heaviest traveled lane, one direction.

CHAPTER 3

DISTRESS TYPES, CAUSES, AND MAINTENANCE PERFORMED

3.1 GENERAL

The major distress types, their causes, and maintenance applied on high traffic volume pavements are presented in this chapter. The results presented are largely based upon an extensive field survey of 68 high traffic volume pavements as described in Chapter 2 and an extensive of previous research presented in the references cited. Results are presented for each pavement type beginning with jointed plain concrete (JCP) and jointed reinforced concrete (JRCP), followed by continuously reinforced concrete (CRC), flexible (FLEX), and composite (COMP) pavement.

3.2 JOINTED RIGID PAVEMENT

3.2.1 General

Jointed concrete pavements (plain or reinforced) are the most common type of high traffic volume pavement used in the United States. Twelve[1] jointed plain concrete pavements (JCP) are included in the field survey and are located in 8 states with widely ranging environments. Of the 18 jointed reinforced concrete pavements (JRCP), 16 are located in 5 states and 2 in one Canadian province. Traffic volume for JCP ranged from 17,000 two-directional ADT to 170,000, and total 18-kip ESAL applications in the heaviest traveled lane from 2.8×10^6 to 32.7×10^6. JRCP two-directional

[1] Two jointed concrete pavements, JCP-13 and JCP-14, in Denver, Colorado, were surveyed later in 1975 and not included as a part of this analysis.

ADT ranged from 17,000 to 187,000 and total 18-kip ESAL application in the heaviest traveled lane from 1.8×10^6 to 26.6×10^6.

The twelve JCP vary in age from 9 to 25 years and 10 of these projects showed maintenance-free performance up to the time of this survey; while the 18 JRCP vary in age from 10 to 33 years and only 7 showed maintenance-free performance. In addition to these projects, other pavements in the various regions visited were observed and the general performance characteristics of jointed rigid pavements in the region were discussed with resident pavement engineers.

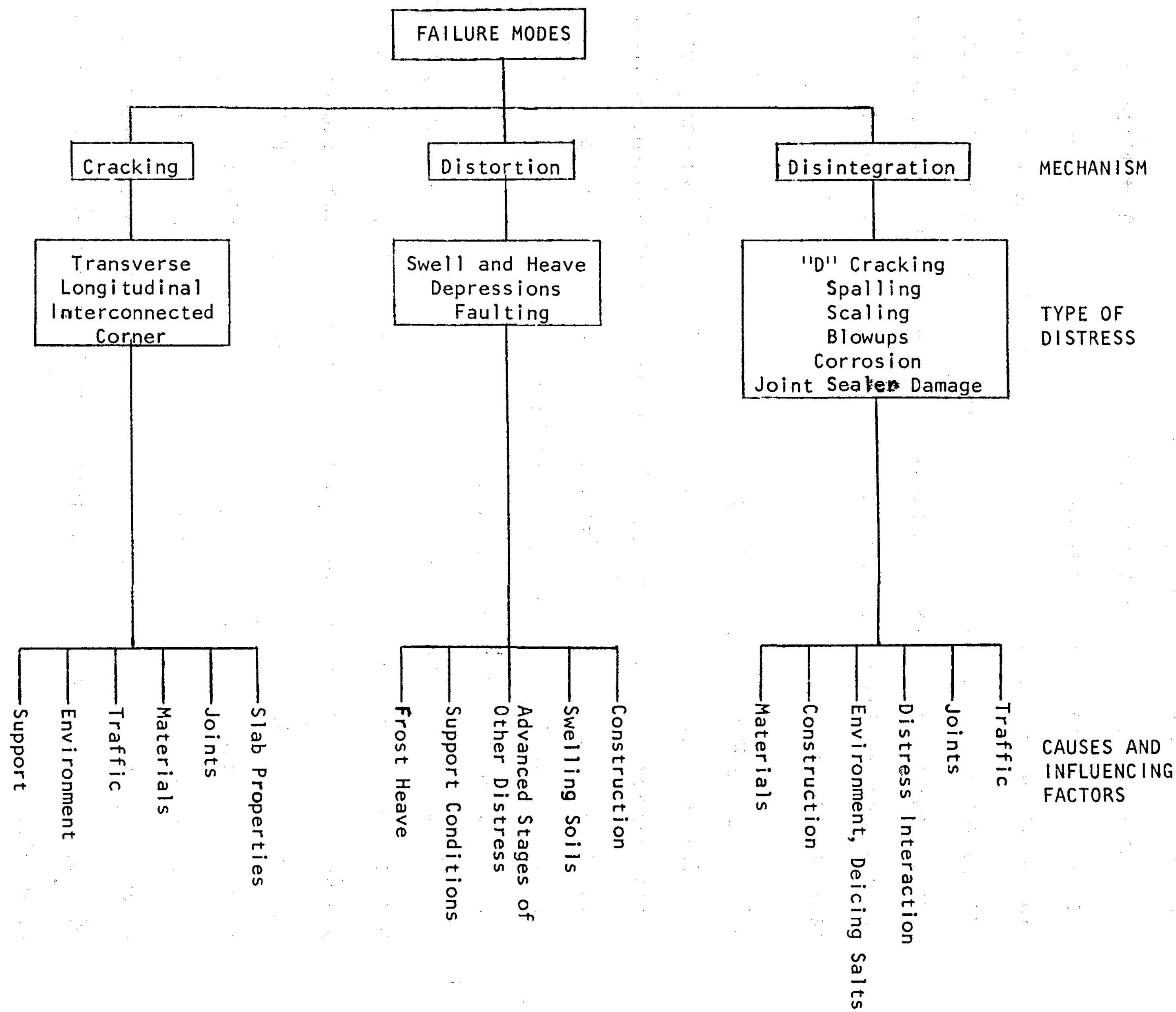
3.2.2 Types and Causes of Distress

The major types of distress occurring on high traffic volume jointed rigid pavements are summarized in this section. Specific causes of these distresses are also identified based on the opinions of the engineers in the various regions visited, the project staff, previous research studies, and also upon some analytical analyses.

The predominant types and general causes of distress in jointed rigid pavements are summarized in Figure 3.1. The various distress types can be grouped into the three main categories of cracking, surface distortion, and concrete disintegration. The general primary causative and contributing factors of distress in jointed rigid pavements are also shown in Figure 3.1.

Distress Types. Previous research studies have identified and defined numerous types of distress that occur in jointed concrete pavements (13, 59,110, 111). Most of these distresses were found on high traffic volume pavements.

Figure 3.1. Modes and Causes of Failure of Jointed Concrete Pavements



Distress observed in jointed rigid pavements during the field survey was classified according to type and severity. Although most severity ratings were subjective, some guidelines were used to provide a uniform basis for rating by the project staff. Furthermore, the ratings were reviewed by the entire staff after viewing the slides and 8 mm movies taken of each project. Severity was rated subjectively on a scale of of zero to five with ratings of none (0-1), minor (1-2), moderate (2-3), major (3-4), and severe (4-5). The general definitions of the ratings given in Table 3.1 are used in subsequent discussions.

The distresses and their level of severity that were found in the field study for JCP and JRCP are summarized in Tables 3.2 and 3.3.

Major Distress Types.

The distress types occurring on high traffic volume jointed rigid pavements which were assigned a severity rating of "moderate" to "severe" for JCP and JRCP are summarized in terms of the number of occurrences in Tables 3.4 and 3.5, respectively. Certain of the distress types are more critical than others insofar as requiring maintenance is concerned. Maintenance performed as related to distress types is also summarized in Tables 3.4 and 3.5.

1. JCP Projects

Joint filler extrusion: All JCPs surveyed showed considerable amounts of sealer stripping and some extrusion with an average rating of "major". The field survey indicates very little relationship between structural maintenance activity and the severity of joint sealer damage. Based

Table 3.1 SUMMARY OF GUIDELINES FOR SEVERITY OF JCP AND JRCP

Distress Type	0 None 1 Minor	Distress Severity 2 Moderate 3 Major	4 Severe 5
Transverse Cracking	cracking tightly closed little or no spalling	cracks somewhat opened to 1/4 in. some spalling	most cracks opened and spalled and faulted
Longitudinal Cracking	less than 5% of slab length	5 to 25% of slab length	greater than 25% of slab length
Spalling	less than 10% of cracks - spalls small and shallow	10 to 25% cracks spalled - some spalls large enough to affect ride	more than 25% of cracks with spalls large enough to affect ride
Faulting	< 0.10"	0.10"-0.20"	> 0.20"
"D" Cracking	occasional evidence of "D" cracking	evidence of "D" cracking but no spalling associated with cracks	Considerable "D" cracking and spalling along cracks
Joint Sealer Extrusion and/or Stripping	less than 5% extruded or stripped	5 to 25% extruded or stripped	greater than 25% extruded or stripped

TABLE 3.2

DISTRESS TYPE AND SEVERITY RATING ON JCP

Distress Type	Distress Severity*					Total No. of Projects Rated
Numerical Rating	0-1	1-2	2-3	3-4	4-5	
Subjective Rating	None	Minor	Moderate	Major	Severe	
Joint Sealer Extrusion and/or Stripping			3**	1	7	11***
Faulting of Joints	2	6	1	1	2	12
Joint Spalling	1	10	1			12
Pumping (edge)	6	2				8****
Pumping (joints,cracks)	10	2				12
Corner Cracking	8	4				12
Diagonal Cracking	11	1				12
Longitudinal Cracking	9	3				12
Transverse Cracking	9	1	1		1	12
Interconnecting Cracking	9	1	2			12
"D" Cracking	9		1	2		12
Blowups	12					12
Scaling	12					12
Repair Patches (joints, slabs)	7	4	1			12
Punchout	12					12
Shoulder Distress****	2	2	2	2		8
Surface Depression	5	5	2			12

* As defined in Table 3.1

** Each number represents the number of projects in this distress category

*** One section had no filler during its entire life

**** Sections had gravel shoulders and were not rated

TABLE 3.3

DISTRESS TYPE AND SEVERITY RATING ON JRCP

Distress Type Numerical Rating Subjective Rating	Distress Severity* 0-1 None	 1-2 Minor	 2-3 Moderate	 3-4 Major	 4-5 Severe	Total No. of Projects Rated
Joint Sealer Extrusion and/or Stripping	2**	5	2	4	5	18
Faulting of Cracks	1	6	6	2	3	18
Joint Spalling	2	7	4	3	2	18
Pumping (edge)	4	6	1	1		12***
Pumping (joints,cracks)	7	8	2	2		18
Corner Cracking	7	5	4	2		18
Diagonal Cracking	9	6	2	1		18
Longitudinal Cracking	7	8	3			18
Transverse Cracking		4	7	2	5	18
Interconnecting Cracking	8	7	3			18
"D" Cracking	16		2			18
Blowups	12	2	4			18
Scaling	17	1				18
Repair Patches (joints, slabs)	3	8	3	3	1	18
Punchout	18					18
Shoulder Distress***	1	2	4	1	1	9****
Surface Depression	6	7	5			18

* As defined in Table 3.1
** Each number represents the number of projects in this distress category
*** Sections had gravel shoulders and were not rated
**** Sections had gravel shoulders or reconstructed shoulders and were not rated

TABLE 3.4

SUMMARY OF DISTRESS TYPES ON JCP
RATED AS MODERATE TO SEVERE

Type of Distress	Projects Distressed/Total	Projects Maintained/Distressed*
1. Joint Filler Extrusion and/or Stripping	11/11	2/11
2. Shoulder Distress	4/8	1/4
3. Faulting Joints	4/12	2/4
4. "D" Cracking	3/12	1/3
5. Interconnecting Cracking	2/12	0/2
6. Surface Depression	2/12	0/2
7. Transverse Cracking	2/12	2/2
8. Joint Spalling	1/12	0/1

* Maintenance applied to only specific distress indicated.

TABLE 3.5

SUMMARY OF DISTRESS TYPES FOUND ON
JRCP RATED AS MODERATE TO SEVERE

Types of Distress	Projects Distressed/Total	Maintained/Distressed*
1. Transverse Cracking	14/18	9/14
2. Paved Shoulder Distress	6/8	4/6
3. Faulting of Cracks	11/18	6/11
4. Joint Filler Stripping	11/18	5/11
5. Joint Spalling	9/18	7/9
6. Surface Depressions or Swell	4/18	4/4
7. Corner Cracking	6/18	5/6
8. Longitudinal Cracking	3/18	2/3
9. Blowups	4/18	4/4
10. Diagonal Cracking	3/18	1/3
11. "D" Cracking	2/18	2/2

* Maintenance applied to only specific distress indicated.

on these results, the importance of the presence of joint sealer material is somewhat questionable (for JCPs) and at least one state (California) does not require sealing material even at the time of initial construction, in other than mountainous areas.

Joint faulting: Of two projects surveyed that faulted enough to be rated as "severe" (> 0.2" fault), only one received maintenance. One project faulting was rated "major" but had not received maintenance. The data in Table 3.4 show that faulting (unless it is "moderate" to "severe") does not usually receive maintenance. However, based on discussions with pavement engineers during the field visits, projects having "severe" faulting normally do receive maintenance in terms of an overlay or grinding. A photo showing "serious" faulting is presented in Figure 3.2.

"D" cracking: The data on this type of distress indicate that the presence of "D" cracking does not necessarily require maintenance even at "major" (3.0 - 4.0) severity level. It is noted, however, that "D" cracking distress normally continues to propagate and can often be the cause of other types of distress, such as blowups, which do require immediate maintenance. A photo of "D" cracking on JCP-3 which was rated as "major" is shown in Figure 3.3.

Transverse cracking: Only two of the JCP projects surveyed had moderate to severe transverse cracking. One project rated as "moderate" (cracks closed and some spalling), and one project rated as "severe" (most cracks opened and spalled). Transverse cracking in both projects had received maintenance, while other projects with transverse cracking rated as "none" to "minor" had not received maintenance. While the data base

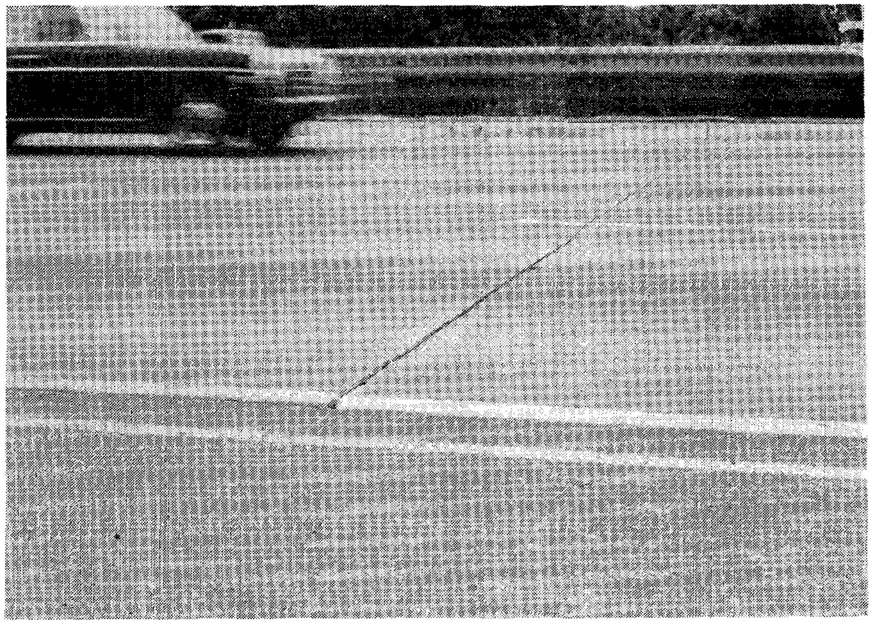


Figure 3.2. Illustration of joint faulting rated "severe" on a jointed concrete pavement (JCP-7-CA)

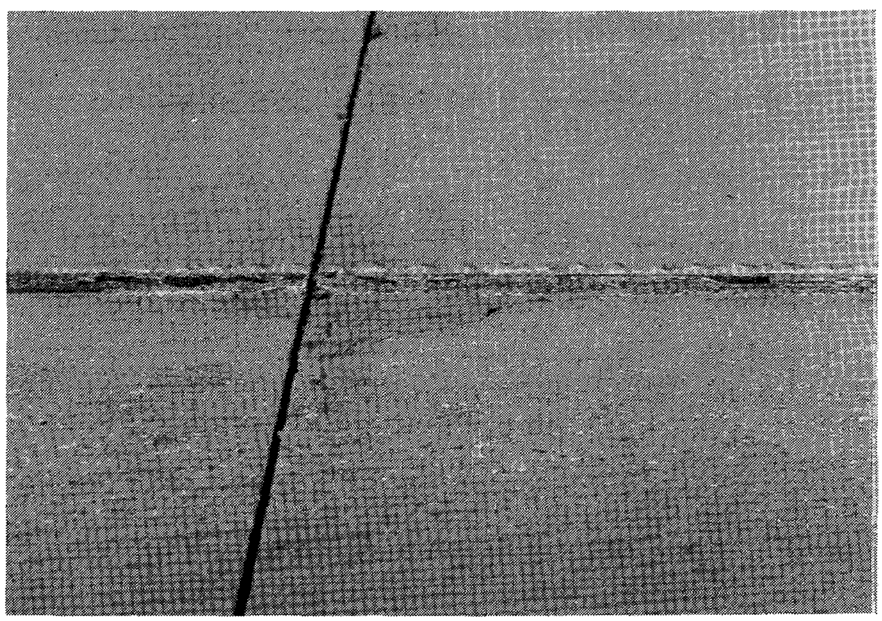


Figure 3.3. Illustration of "D" cracking rated "major" on a jointed concrete pavement (JCP-3-IL).

is very limited, it shows a high correlation between required maintenance and the severity rating of transverse cracking. Several other projects were observed to have transverse cracking during the field visits. A photo of transverse cracking rated as "severe" in JCP-1 is shown in Figure 3.4.

Based upon these data from the field survey, the major distress types occurring in high traffic volume JCP's that require or receive maintenance and their level of severity are as follows:

(a) Transverse cracking: occurs transverse to the longitudinal axis of the roadway at approximately mid-slab, and is load-associated distress, with severity level of moderate to severe (cracks opened and spalled).

(b) Faulting: occurs at transverse joints without dowels and transverse cracks with severity levels of $\geq$ 0.2 in. fault.

2. JRCP Projects

Transverse cracking: All projects with transverse cracking rated as "severe" (most cracks opened and spalled) required maintenance, while 4 out of 9 rated as "moderate" to "major" received maintenance. The data show that transverse cracking almost always received maintenance when its severity reached a level of "moderate" and above. Typical transverse cracking distress is shown in Figures 3.5 and 3.6.

Joint sealer extrusion/stripping: No relationship was found between maintenance activity and the severity of joint sealer extrusion/stripping. Eleven of the eighteen JRCP had "moderate" to "severe" joint sealer stripping but only five of these eleven had received maintenance. Conversely, five of the nine pavements with joint sealer stripping rated as "none" to "minor" also had maintenance performed. Thus, there appears to be no consistent pattern between the severity of joint sealer stripping and maintenance.

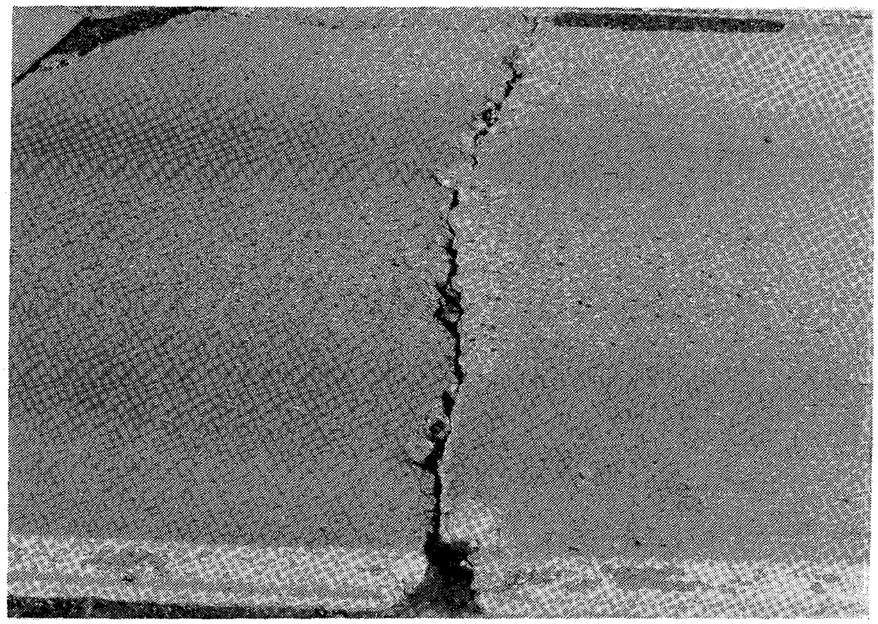


Figure 3.4. Illustration of transverse cracking rated "severe" on a jointed concrete pavement (JCP-1-IL).

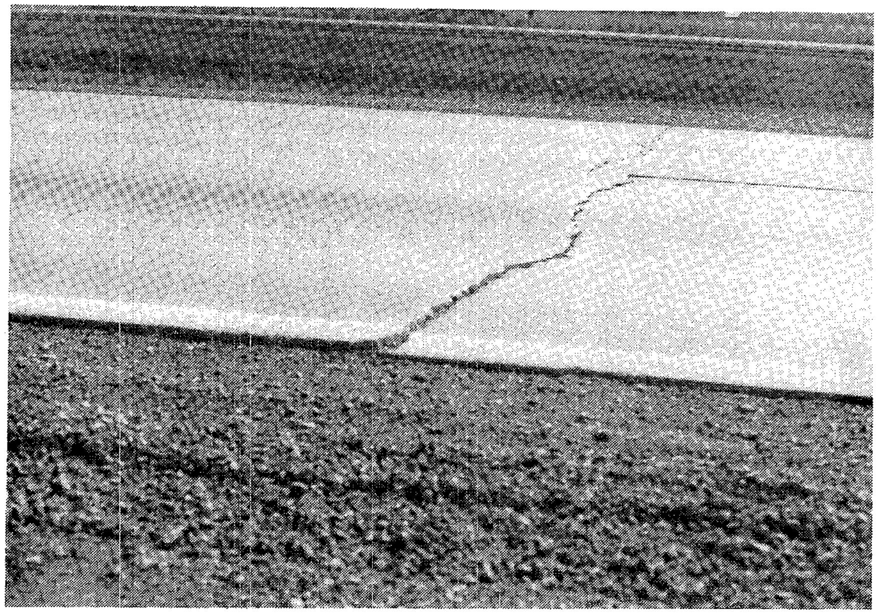


Figure 3.5. Illustration of transverse cracking rated as "severe" with steel rupture and crack faulting (JRCP-12-CAN).

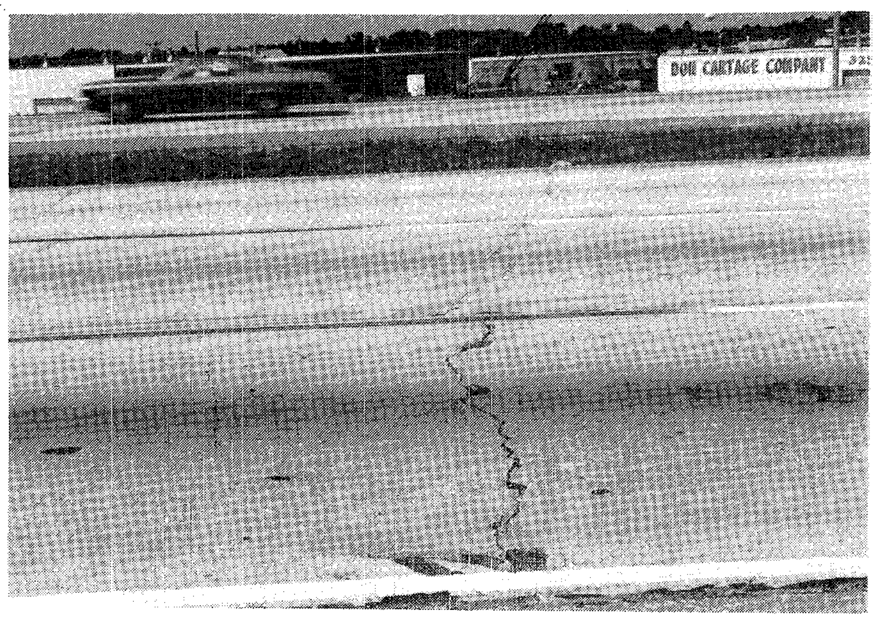


Figure 3.6. Illustration of transverse cracking rated as "major". (JRCP-18-IL).

Joint spalling and crack faulting: There is a strong correlation between maintenance activities and joint spalling and crack faulting as the severity of each type of distress increases from "moderate" to "severe." All JRCP with joint spalling and crack faulting rated as "severe" had maintenance performed. Typical joint spalling distress rated "severe" is shown in Figure 3.7.

Based on these data from the field study, the major distress types occurring in high traffic volume JRCP projects that require or receive maintenance and the acuteness of the distress from the standpoint of maintenance needs are as follows:

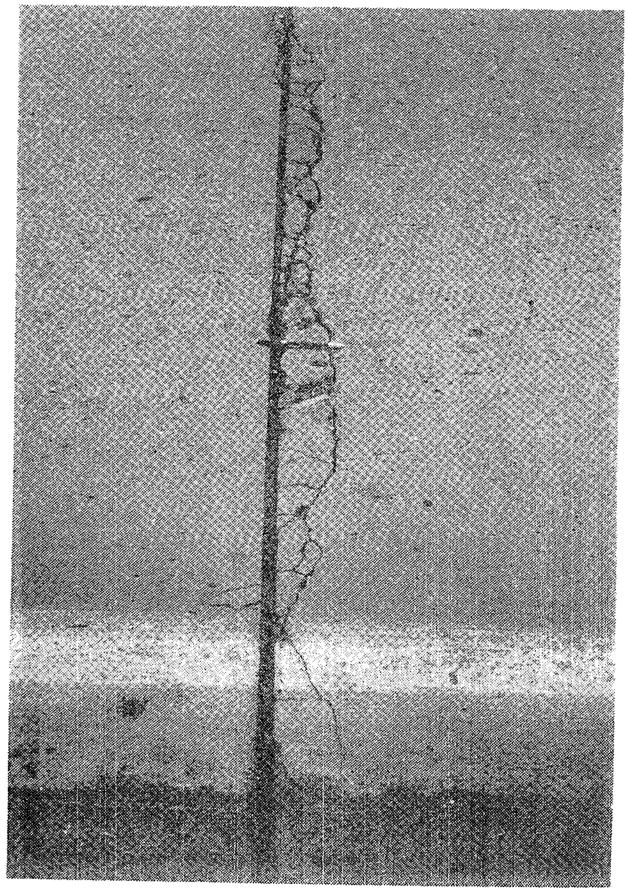
(a) Transverse cracking occurs transverse to the longitudinal axis of the roadway. Transverse cracking may result in steel rupture, crack faulting, and crack spalling under heavy traffic. This is a major factor in pavement maintenance activity levels.

(b) Joint spalling is a very significant distress if it becomes moderate to severe in requiring maintenance.

(c) Crack faulting occurs at transverse cracks, probably after reinforcement rupture, requires increasing maintenance effort as it becomes more severe.

(d) Steel rupture when it occurs at transverse cracks tends to accelerate crack deterioration, and hence the maintenance activity.

(e) Joint lockup occurs at transverse joints when load transfer devices restrain normal movement of the joint and may result in rupture of reinforcing steel between joints with subsequent crack faulting and spalling.



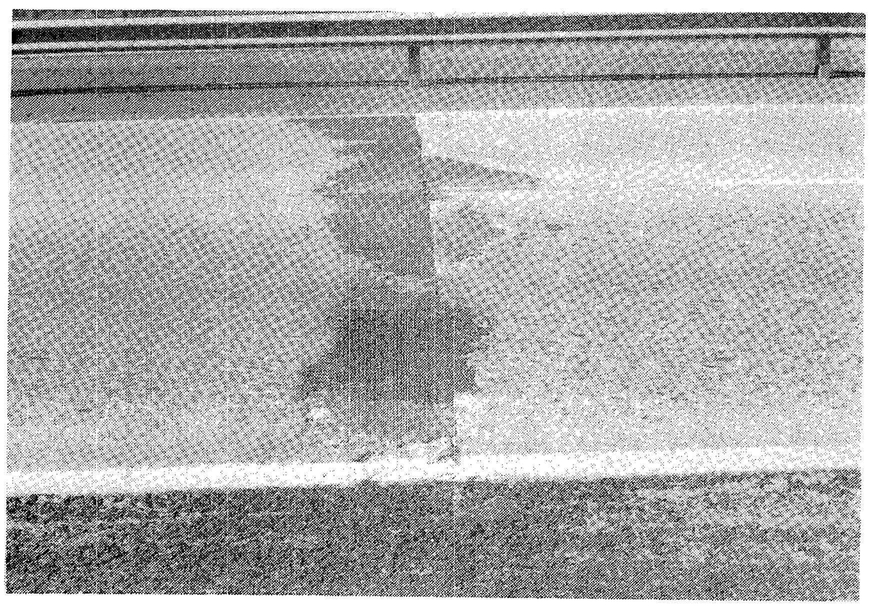


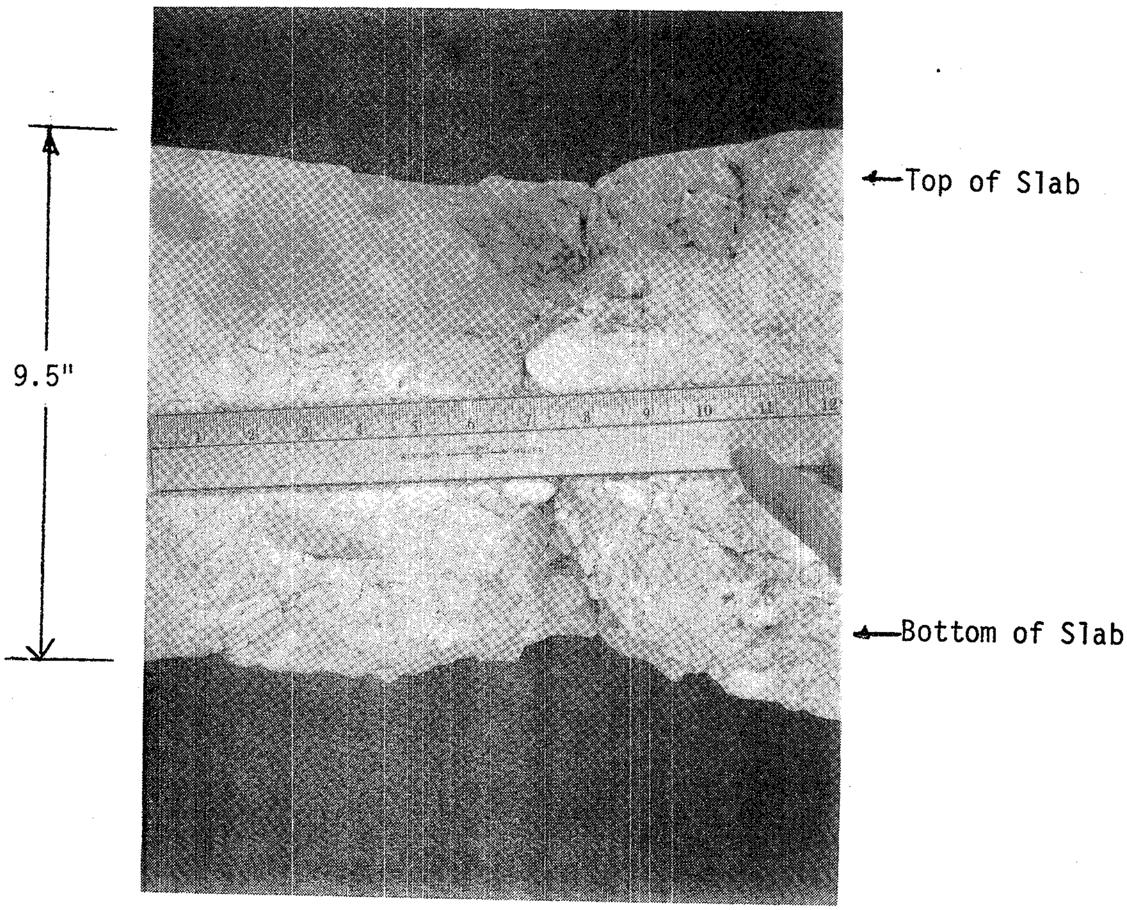
Figure 3.7. Illustration of joint spall rated as "severe" on a jointed reinforced concrete pavement (JRCP-16-NY).

Other Distresses. Besides the major distresses previously identified on the projects surveyed, there are other distresses that occur in varying amounts and degrees depending on construction, foundation soil, materials, environment, etc. These distress types are as follows:

1. Concrete Deterioration: Including deterioration of concrete due to freeze-thaw, deicing salts, and "D" cracking. Several states in wet-freeze regions reported observing concrete deterioration. This can be seen in Figure 3.8 which shows a typical joint cut out from JRCP-7 (ILL-AASHO Road Test Site) and examined. As seen in Figure 3.8, considerable deterioration had taken place in the lower portion of the slab near the joint and may have been caused by a combination of the above listed factors. This pavement has received approximately 15 tons/lane mile of salt annually. Also, concrete wear due to studded tires was serious in several northern states. This distress is not within the scope of this project, however, but cannot be neglected in actual design. This was a major concern for maintenance engineers in these regions.

2. Swelling or Heaving Foundations: Certain regions of the country have serious problems with swell and heave of the subgrade soils. A typical example of the resulting cracking due to heave of swelling soils is illustrated for JRCP-4 (TX) in Figure 3.9.

3. Shoulder Distress: Several shoulders on jointed rigid pavements showed serious distress, but this type of distress was particularly noted for those pavements located in wet-freeze regions. This subject is discussed under a separate heading.



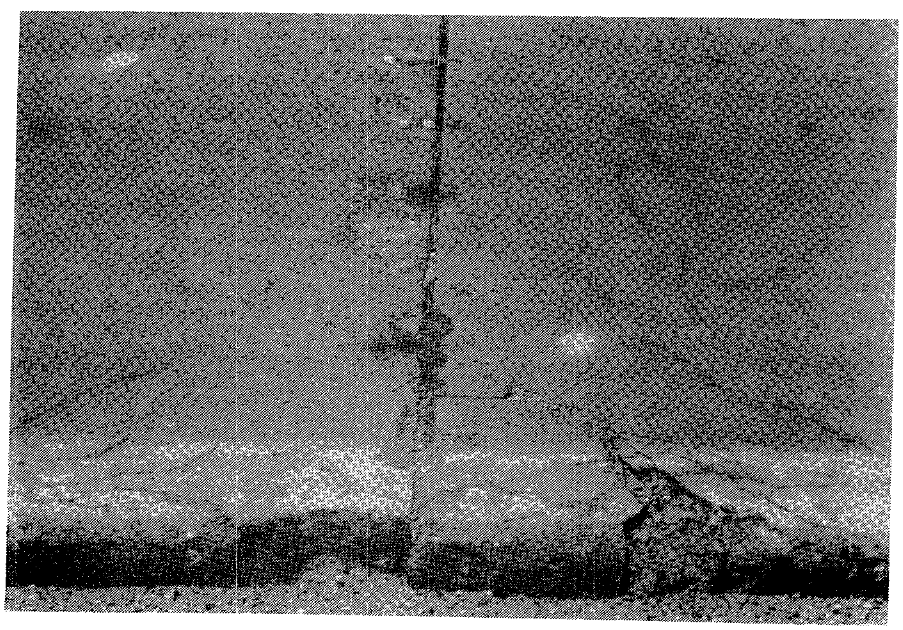


Figure 3.8. Joint cut from 17-year concrete pavement (JRCP-7-IL) showing serious concrete deterioration.

4. Blowups: Blowups were found in four JRCP projects. They were rated as "moderate" in 3 sections and "severe" in another section. A typical blowup is illustrated in Figure 3.10.

Causes of Distress. The known causes are summarized for each of the major distress types identified in the field survey for JCP and JRCP projects.

1. Transverse Cracking: Cracking occurs when the tensile stresses in the concrete slab exceed the strength of the concrete. The tensile capacity of the concrete slab is a function of concrete strength and slab thickness. Tensile stresses are the result of:

(a) Traffic loadings.

(b) Restrained curling and warping of concrete slabs due to the temperature and moisture changes, respectively.

(c) Restraint of expansion and contraction of the concrete slab due to friction between slab and its supporting foundation and from adjacent slabs and/or structures.

(d) Partial or full joint lockup.

(e) Loss of foundation support.

Factors influencing transverse crack distress on JCP and JRCP sections are shown in Tables 3.6 and 3.7, respectively. Of 12 JCP projects with joint spacing of 12 to 18 ft, and one with 30 ft, only one section had transverse cracking rated as "severe" and one rated as "moderate," while others had ratings of "none" to "minor". However, of the 18 JRCP projects with joint spacing of 15 to 100 ft, 14 sections had transverse cracks rated as "moderate" to "severe." This difference in cracking incidence of JCP and JRCP sections

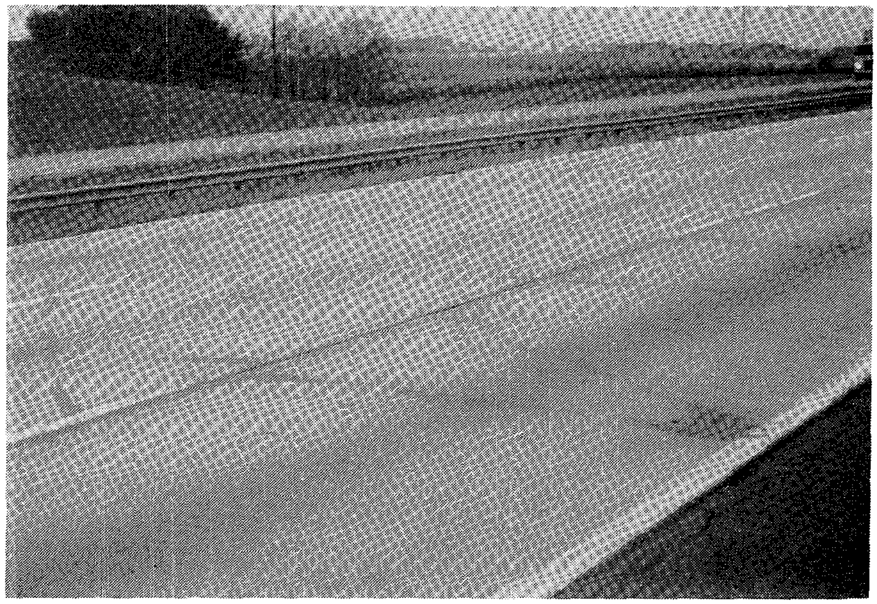


Figure 3.9 . Illustration of the effect of swelling clays on a jointed reinforced concrete pavement (JRCP-4, Dallas, Texas).

Figure 3.10. Illustration of a joint blowup on a jointed reinforced concrete pavement (JRCP-18, Chicago, Illinois).

Table 3.6 FACTORS INFLUENCING TRANSVERSE CRACK DISTRESS ON JCP SECTIONS

JCP SECTION NO.	STATE	DESIGN FACTORS			STRESS FACTORS			DISTRESS RATING		MAINTENANCE REQUIRED
		SLAB THICKNESS	CONCRETE FLEXURAL STRENGTH (6)	JOINT SPACING	SUSPECTED JOINT LOCKUP	(3) TRAFFIC	(4) ENV.	TRANSVERSE CRACKING	(5) CRACK SPALLING FAULTING	
1	IL	8	690 psi	15'	yes	9.5	WF	severe	severe	yes
2	IL	9.5	690 psi	15'	yes	16.8	WF	minor	minor	no
3	IL	11	690 psi	15'	yes	17.4	WF	none	none	no
4	IL	12.5	690 psi	15'	yes	18.5	WF	none	none	no
5	WA	9	>650 psi	15'	no	4.3	W	none	none	no
6	CA	8	625 psi (1)	15'	no	12.4	D	moderate	minor	yes
7	CA	8	625 psi (1)	15'	no	32.7	D	minor	minor	no
8	TX	10	687 psi	15'	no	2.8	D	none	none	no
9	UT	9	690 psi (1)	12-18'	no	4.1	DF	none	none	no
10	AZ	9	690 psi (1)	15'	no	27.6	D	none	none	no
11	NJ	10	707 psi	15'	no	29.4	WF	none	none	no
12	GA	8	690 psi (1)	30' (2)	no	17.0	W	minor	minor	no

(1) Estimated

(2) This section has expansion joints at 600 ft.

(3) Millions 18 kip ESAL

(4) Environmental Condition, W = Wet/No Freeze-Thaw, D = Dry/No Freeze-Thaw, WF = Wet With Freeze-Thaw, DF = Dry With Freeze-Thaw

(5) Severe > 0.2", moderate-major 0.1-0.2", none-minor < 0.1".

(6) 28 day and 3rd point loading.

Table 3.7 FACTORS INFLUENCING TRANSVERSE CRACK DISTRESS ON JRCP SECTIONS

JRCP SECTION NO.	STATE	DESIGN FACTORS: SLAB THICKNESS	DESIGN FACTORS: CONCRETE FLEXURAL STRENGTH	DESIGN FACTORS: PERCENT REINF. REQUIRED (3)	STRESS FACTORS: JOINT SPACING & TYPE	STRESS FACTORS: SUSPECTED JOINT LOCKUP	STRESS FACTORS: (5) TRAFFIC	STRESS FACTORS: (7) ENV.	DISTRESS: TRANSVERSE CRACKING (6)	DISTRESS: (6) CRACK SPALLING FAULTING	MAINTENANCE REQUIRED
1	TX	9"	550-700	187	30'6" exp	no (4)	3.8	W	moderate	minor	no
2	TX	9"	575-700	67	15' cont (9)	no	1.7	W	minor	minor	no
3	TX	8"-6"-8"	(1)	1310	20' cont (10)	no	7.0	D	minor	none	yes
4	TX	10"	(1)	174	30' cont	no	5.8	D	moderate	minor	yes
5	NJ	10"	530-810 (2)	89	78' exp	no	26.6	WF	minor	none	no
6	IL	8"	690	89	40' cont	yes	9.5	WF	severe	severe	yes
7	IL	9.5"	690	137	40' cont	yes	16.8	WF	severe	major	yes
8	IL	11"	690	159	40' cont	yes	17.4	WF	major	moderate	yes
9	IL	12.5"	690	142	40' cont	yes	18.5	WF	moderate	minor	no
10	IL	10"	690	209	40' cont	no	7.7	WF	major	minor	no
11	CAN	9"	550	164	56' cont	no	24.5	WF	moderate	minor	no
12	CAN	9"	550	92	99' cont	yes	10.5	WF	severe	severe	yes
13	MI	10"	580-730 (2)	83	100' cont	yes	4.5	WF	severe	moderate	yes
14	MI	10"	650-750 (2)	83	100' cont (11)	no	9.1	WF	minor	minor	no
15	NY	9"	640 (2)	48	100' exp	yes(4)	7.0	WF	moderate	severe	yes
16	NY	9"	640 (2)	80	60'10" exp	yes(4)	9.1	WF	moderate	moderate	yes
17	NY	9"	640 (2)	48	100' exp	no	4.7	WF	minor	minor	no
18	IL	10"	(1)	83	100' cont	yes	11.5	WF	severe	moderate	yes

(1) Not available

(2) $\sigma_{flex} = 10\sqrt{\sigma_c}$

(3) As determined by AASHO eq. $A_s = \frac{LFW}{2f_s}$

(4) Proprietary lugs used as LTD.

(5) Millions 18 kip SAL equivalents

(6) From condition surveys

(7) Environmental Condition, W = Wet/No Freeze-Thaw, D = Dry/No Freeze-Thaw, WF = Wet With Freeze-Thaw, DF = Dry With Freeze-Thaw

(8) exp = expansion, cont = contraction

(9) plus 45' exp./(10) plus 100' exp./ (11) plus 400' exp.

is thought to be due partially to slab length. The JRCP projects with the longer slabs are subjected to much higher warping and frictional stresses.

The effect of slab thickness on severity of transverse cracking is shown in Table 3.8 for both JCP and JRCP pavements. Thicker slabs clearly reduce the severity of transverse cracking. Thicker slabs would exhibit less deflection under load, and hence a smaller amount of slab bending at the crack, causing less rupture of the steel reinforcement. Reinforcement is placed in JRCP to restrict the width of the crack and to assure load transfer through aggregate interlock. Once the steel ruptures, however, the crack soon spalls, the pavement often faults, and usually requires maintenance under heavy traffic. Although only eight pavement sections are shown in Table 3.8, they represent the performance of approximately 50 sections.

Joint lockup is a factor that apparently contributes significantly to transverse cracking, and is more critical for longer joint spacings. Joint lockup is caused by corrosion of load transfer devices, high frictional resistance between dowels and concrete (lack of bond breakers), or misalignment of dowels. Based upon the results of this and other studies (163, 176), lockup appears to be a normal occurrence where plain steel dowels or other malleable iron LTD's are used, and deicing salt is applied in significant amounts. Joint lockup was suspected because of apparent lack of joint movement on 9 JRCP sections out of 18 surveyed, and all 9 sections were rated "moderate" to "severe" on transverse cracking, crack faulting, and spalling (Table 3.7).

Two 6-foot long sections of doweled joints (one half a lane width containing six dowels) were cut from JRCP-7 and 10 in Illinois. Both were subjected to joint pull-apart tests to determine the extent of lockup. The maximum force necessary to open the joint to 0.1 inches was 28,000 lbs for JRCP-7 (17 years old), and 14520 lbs for JRCP-10. The dowels were

Table 3.8 EFFECT OF SLAB THICKNESS ON TRANSVERSE CRACK AND JOINT FAULTING SEVERITY OF JCP AND JRCP PROJECTS.*

Project	Slab Thickness-in.	Transverse Cracking	Joint Faulting
JCP - 1 - IL	8	severe	severe (> 1/4")
JCP - 2 - IL	9.5	moderate	moderate (1/8")
JCP - 3 - IL	11	none	none
JCP - 4 - IL	12.5	none	none
JRCP - 6 - IL	8	severe	severe (1/4")
JRCP - 7 - IL	9.5	severe	minor (1/16")
JRCP - 8 - IL	11	major	none
JRCP - 9 - IL	12.5	moderate	none

*All sections are 16 years old and originally part of Loop 6 of the AASHO Road Test but have served since as part of I-80. Project JCP-1 and JRCP-6 had traffic of approximately 10×10^6 kip ESAL and all others had approximately 20×10^6 kip ESAL.

Although only eight pavement sections are indicated in this table, they are representative of approximately 50 sections that are 16 years old.

severely corroded and steel reinforcement rupture had taken place in the slab on either side of the joint. These results indicate extensive lockup had occurred (179). Transverse cracking and subsequent crack faulting and spalling result from one or more of the following:

(a) Inadequate structural thickness of concrete slab and inadequate reinforcing steel for the traffic load applied.

(b) Subbase pumping which greatly accelerates crack initiation and deterioration.

(c) Long joint spacing which results in high curling and frictional stresses.

(d) Joint lockup which restrains longitudinal movements.

(e) Improper joint type and design.

2. Joint Spalling: Several factors have been identified as causing spalling including: (1) infiltration of incompressibles, (2) misalignment of dowels, (3) use of unusual load transfer devices which prohibit joint movement, (4) too long of joint spacing, (5) joint forming methods, (6) joint sealing practices, (7) excessive joint deflection, and (8) concrete deterioration.

Joint spalling severity and related factors are shown in Tables 3.9 and 3.10 for the JCP and JRCP projects. None of 12 JCP sections had joint spalling rated "moderate" to "severe." However, 7 out of the 18 JRCP projects had joint spalling rated "moderate" to "severe." The following observations and conclusions were drawn from the field surveys and related interviews.

(a) All JRCPs having approximately 100 ft contraction joint spacing showed "major" to "severe" joint spalling (JRCP-12-CAN, 13-MI, 18-IL). This distress was also observed during the field visits on other projects not included in the survey having the same joint configuration design.

Table 3.9 FACTORS INFLUENCING TRANSVERSE JOINT DISTRESS ON JCP

JCP Section No.	STATE	DESIGN FACTORS		DETERIORATION FACTORS			DISTRESS FACTORS		
		CONTRACTION		ENVIRONMENT					
		SPACING	LTD	CLIMATE(2)	AGE (YEARS)	TRAFFIC(4)	SUSPECTED LOCKUP	JOINT SPALLING(5)	FAULTING(6)
1	IL	15'	dowels	WF	16	9.54	yes	minor	severe
2	IL	15'	dowels	WF	16	16.85	yes	minor	moderate
3	IL	15'	dowels	WF	16	17.42	yes	minor	none
4	IL	15'	dowels	WF	16	18.54	yes	minor	none
5	WA	15'	none	W	12	4.3	no	minor	minor
6	CA	15'	none	D	19	12.4	no	minor	moderate
7	CA	15'	none	D	20	32.7	no	minor	severe
8	TX	15'	none	D	14	2.8	no	minor	minor
9	UT	12'-18'(3)	none	DF	9	4.1	no	minor	minor
10	AZ	15'(3)	none	D	14	27.6	no	minor	minor
11	NJ	15'	none	WF	25	29.4	no	none	major
12	GA	30'(1)	none	W	24	17.0	no	minor	minor

1. This section has expansion joint at 600 ft
2. W=Wet No Freeze-Thaw, D=Dry No Freeze-Thaw, WF=Wet With Freeze-Thaw, DF=Dry With Freeze-Thaw
3. Skewed joints.
4. Millions 18 kip SAL equivalents application
5. Taken from field visit condition survey
6. Severe ($\geq$ 0.2"), major-moderate (0.1-0.2"), minor-none (< 0.1")

Table 3.10 FACTORS INFLUENCING TRANSVERSE JOINT DISTRESS ON JRCP

PROJECT		DESIGN FACTORS				DETERIORATION FACTORS				DISTRESS FACTORS		
		CONTRACTION		EXPANSION		ENVIRONMENT		AGE		SUSPECTED	JOINT 5	
NO.	-STATE	SPACING	LTD	SPACING	LTD	CLIMATE 2	AASA 3	(YEARS)	TRAFFIC 4	LOCKUP	SPALLING	FAULTING
1	TX	none		30'6"	proprietary	W	none	24	3.8	no	minor	minor
2	TX	15'	dowels	45'	dowels	W	none	22	1.7	no	minor	minor
3	TX	20'	dowels	100'	dowels	D	none	34	7.0	no	moderate	none
4	TX	30'	dowels	none		D	none	17	5.8	no	minor	minor
5	NJ	none		78'	dowels(7)	WF		23	26.6	no	minor	minor
6	IL	40'	dowels	none(1)		WF	60	16	9.54	yes	minor	severe
7	IL	40'	dowels	none(1)		WF	60	16	16.85	yes	moderate	major
8	IL	40'	dowels	none(1)		WF	60	16	17.42	yes	none	moderate
9	IL	40'	dowels	none(1)		WF	60	16	18.54	yes	none	minor
10	IL	40'	dowels	none(1)		WF	60	16	7.73	no	minor	minor
11	CAN	56'	dowels	none		WF	20-25	13	24.5	no	minor	minor
12	CAN	99'	dowels	none		WF	20-25	15	10.5	yes	major	minor
13	MI	100'	dowels	none		WF	20-30	13	4.5	yes	severe	minor
14	MI	100'	dowels	400'	dowels	WF	20-30	10	9.1	no	minor	minor
15	NY	none		100'	proprietary	WF		20	6.9	yes	major	moderate
16	NY	none		60'10"	proprietary	WF		19	9.1	yes	severe	moderate
17	NY	none		100'	dowels(7)	WF		14	4.7	no	minor	minor
18	IL	100'	dowels	none		WF		14	11.5	yes	major	moderate

1. AASHO Road Test Sections - 240' experimental sections
2. W=Wet No Freeze-Thaw, D=Dry No Freeze-Thaw, WF=Wet With Freeze-Thaw, DF=Dry With Freeze-Thaw
3. AASA = Average Annual De-Icing Salt Applications (total tons salt/lane mile range from 5 to 20 per year)
4. Millions 18 kip SAL Equivalents Application
5. Taken from Field Visit Condition Surveys
6. Faulting At Transverse Cracks Are Rated (See Table 3.9).
7. Stainless Steel Coated

(b) "D" cracking was responsible for moderate spalling on one project (JRCP-7-IL) and minor spalling on 5 others.

(c) Two projects in wet-freeze regions (JRCP-15-NY, 16-NY) had unusual proprietary load transfer devices and both showed major to severe spalling. One project (JRCP-1-TX) had a proprietary load transfer device that did not show spalling, but probably because the pavement was located in a wet/non-freeze region with 30 ft joint spacing which likely resulted in very little required joint movement.

(d) Five projects had expansion joints with dowel LTDs. None of these projects showed major or severe spalling, and only one showed moderate spalling (this project, JRCP-3-TX, was a 33 year old project, however). Several others were observed in New Jersey ranging in age from 20 to 30 years that showed no distress and were maintenance-free.

(e) None of the projects with short joint spacing (i.e., < 30 ft) showed any significant spalling.

(f) All pavements, except one, with moderate to severe joint spalling were located in wet-freeze regions.

3. Joint Faulting: This type of joint distress is produced by traffic, probably in combination with water. As a load crosses the joint, a downward thrust is applied suddenly to the leave slab as compared to the more gradual rate of application on the approach slab. It was noted in California (120, 121), for JCPs, that there was particle movement counter to the direction of traffic when water collected below the concrete, even though the bases were stabilized with cement. It was concluded in these studies that faulting is caused by buildup of loose materials under the approach slab near the joints rather than a depression of the leave slab. The source of the buildup is found to be the surface layer of the cement treated base and

untreated shoulder materials. In another study in Georgia (50), it was concluded that heavy axle loads, existence of free water and loose base and shoulder materials have great effect on faulting of jointed rigid pavements. These studies are made on non-doweled joints. It is also believed that with non-stabilized bases, consolidation of the base material can also lead to joint faulting.

Lack of proper load transfer across the transverse joints permits faulting to occur. The effect of load transfer on faulting of jointed rigid pavements was studied by the State of Florida (125). In this study, it was noted that the faulting of doweled joints is significantly less than with non-doweled joints. Other studies have also shown this result (177).

Of 12 JCP sections, 4 had dowels as load transfer across the transverse joints and only one of the doweled joints had joint faulting rated "severe," one had "moderate" faulting and the other 2 had no faulting. These were all under heavy traffic (i.e., 10-20 x 10^6 18-kip ESAL). The faulting of these sections is summarized in Table 3.8. However, all 8 JCP sections without dowels had joint faulting rated "minor" to "severe." (Table 3.9). Furthermore, it appears that severity of faulting on non-doweled sections is a function of traffic. JCP-7-CA and 11-NJ which had highest traffic (32.7 and 29.4 x 10^6) were rated "major" to "severe" joint faulting. It is important to note, however, that there are other factors such as joint skewing, slab thickness, joint spacing, subbase stabilization, and environment that affect faulting of joints without dowels (50). Two out of 12 JCP projects (9-UT and 10-AZ) which had skewed joints were rated as "minor" on joint faulting, and one of these (JCP-10-AZ) had very heavy traffic (27.6×10^6 18-kip ESAL). Only one of the three (JCP-9-UT) had a stabilized base and is located in a dry/freeze region.

4. <u>Blowups</u>: These occur at joints or cracks in concrete pavements when high temperature and moisture in the concrete produce expansions which are greater than available expansion space, and expansion forces are greater than the slab capacity. The weakest joint of a series offers stress relief to the compression built up in the pavement by shattering or by buckling upward.

Accumulated infiltration of hard, relatively incompressible materials into joints over a period of years, high temperature and moisture, long joint spacing, concrete deterioration, and joint lockup are major factors contributing to blowups. Some observations on blowups are summarized in the NCHRP synthesis on joint design (92) as:

> "Most blowups occur during the spring or early summer after a significant hot spell combined with recent rain, and usually occur late in the afternoon.
>
> Although blowups do occur in growing concrete caused by chemical reactions (such as an alkali-aggregate reaction), the extent of such growth is not very prevalent across the United States. Most blowups occur in chemically stable concrete where physical lengthening is caused by debris infiltration at the cracks and joints.
>
> Blowups seldom occur where joint spacings are less than 20 ft (6 m) (with no intermediate expansion joints), even where joints are not sealed.
>
> Blowups occur at various frequencies; the maximum observed is about one blowup per mile per year (0.6 per km per year).
>
> Blowups usually occur at joints or cracks in the pavement and the concrete at the blowup appears to be weak or deteriorated at that point.
>
> These observations fit a logical pattern when they are considered in light of the following:
>
> Pavements with long slabs have greater joint opening than do those with short slabs. A 100-ft (30 m) slab will have a joint opening of approximately 3/4 in. (19 mm), and it is difficult to maintain effective seals. Areas where deicing grit is used have a copious supply of material available to fill the joints. Most of these areas also used long slabs.

Good drainage and chemically stabilized bases and shoulders restrict the amount of loose materials that can infiltrate joints. Slab ends deflect more on soft bases and produce pumping action that forces debris upward into the joints.

Joints are particularly susceptible to being the focal point for blowups because:

(a) Often poor concrete is placed at joints during construction.
(b) Salts, moisture, and infiltration debris can cause concrete deterioration.
(c) Because of the saw cuts, only about 80 percent of the depth of the concrete is in contact.
(d) Warping/curling and loads can cause spalling at the top or bottom to further reduce the effective cross section.
(e) Excessive faulting and deflection at joints can also reduce the area of the faces in contact.

During a cold winter, much debris can infiltrate the open joints. Subsequent warming in the spring can cause expansion and blowups. Once the pavement has blown up, it has relieved itself. If no exceedingly warm weather follows, no subsequent blowups will occur that summer. However, the cycle begins again the following winter."

Of 12 JCP projects with short joint spacing (from 12 to 18 ft and one with 30 ft), none had blowups. However, of 18 JRCP projects, three (JRCP-12-CAN, 15-NY, and 18-IL) showed moderate to severe blowups. All projects had 100 ft joint spacings and were located in wet/freeze-thaw areas. One project (JRCP-15-NY) had expansion joints with proprietary LTDs that "froze" and two others had contraction joints only.

5. "D" Cracking: It is generally agreed that "D" cracking is an environment-induced deterioration of the concrete produced by freezing and thawing and is related to the aggregate type and soundness and is strongly affected by the degree of saturation of the concrete.

The "D" cracking deterioration type of distress is noted to be more prevalent in geographical areas where the freezing index ranges from 200 to 1000 (38). Freezing index in this range is usually associated with a large number of freeze-thaw cycles each season. For example, Chicago has a freezing index of approximately 650 and 12 freeze-thaw cycles per year at a depth of 4 inches below the pavement surface (170).

A few agencies, primarily from the regions where "D" cracking occurs, have research programs in progress. Recent reports indicate some success in retarding the development of "D" cracking distress in concrete through the use of smaller maximum particle size for the aggregates (171, 172, 173, 174). Aggregate source is the largest cause of "D" cracking, and only reduction in the size of the coarse aggregate fraction has been effective in reducing this distress.

Of the projects surveyed, only those located at the site of the AASHO Road Test showed "D" cracking. However, this distress was observed by the project staff and others in many pavements in the areas surrounding the projects.

6. Salt Deterioration: Use of salt as a deicing agent in freeze regions may be the cause of or contribute significantly to many types of pavement distress (16). Joint spalling, dowel bar and reinforcing steel corrosion which results in joint lockup and steel rupture, transverse cracking, crack faulting, crack spalling, and concrete deterioration are all associated with the application of significant quantities of deicing salts.

The results of heavy salt applications which were found to range from 5 to 20 tons salt/lane mile per year in freeze regions were a major concern of maintenance engineers visited in northern states.

3.2.3 Maintenance Activities

The maintenance activities which are of concern in this report are those which deal with maintenance of the pavement structural system. This includes the pavement slab and shoulders, but excludes maintenance of bridges and approach slabs. Maintenance activities such as mowing, snow and litter removal, etc., are not considered here. Other exclusions are maintenance or major construction operations which are constructed to increase the traffic capacity of the pavement systems or improve the skid resistance of pavement surface.

The most common maintenance activity on jointed rigid pavements is joint and crack sealing. Initial sealing of contraction joints was performed on all jointed rigid pavements that were investigated in the field study with the exception of JCP-6 and 7 in California.

The stated purpose of joint sealer materials is to prevent incompressible materials from infiltrating the joints, and to seal the surface and protect the subbase and subgrade from surface water intrusion. However, the idea of effectiveness of sealants for protecting pavement from surface water intrusion is questionable (23, 164).

Several types of joint sealers have been used; however, none of them had reliable performance over a long period of time. Preformed neoprene joint sealers have reportedly functioned well (27, 49), but expected life of this type joint sealer is still limited to 5-10 years of service life. Other sealants, such as polysulfides, hot-poured rubber asphalt, latex combined with extender, rubberized asphalt, etc., had little success for longer than 1-5 years of service.

The time period of resealing operations varies from 6 month intervals to 5 years, but yearly intervals are the most common. Hot-pour asphalts are the most commonly used material for resealing.

It is also reported (23, 49, 72, 99, 113) that crack and joint sealing operations do not always prevent surface water from entering the lower parts of the pavement system. In many cases, sealants were effective for only a very short period after application. These studies suggest that provisions for adequate internal drainage must also be made to minimize damage from surface water infiltration. Implementation of good sealing and drainage practices such as recommended in NCHRP project 14-3 could significantly increase the maintenance-free life of jointed pavements.

Prevention of infiltration of incompressible material cannot be totally achieved by the use of joint sealers because small incompressible fragments become trapped in the liquid sealers and become embedded by the action of traffic. Also, granular particles work up into the joints from below.

Patching is another common maintenance activity on jointed rigid pavements. Patching is used to correct transverse and/or longitudinal joint and crack spalling and faulting, blowups, corner breaks, or any other surface roughness. Both asphalt concrete and portland cement concrete have been used for full depth patching, however, in the past, only asphaltic concrete has been used for partial depth patching and to correct joint faulting and minor spallings. More recent technology, however, makes it possible to place partial depth rigid patches made from concrete or resins. Full depth patches are used to correct joint spalling, corner cracks, punchouts, diagonal cracks, and blowups. These are usually made from concrete or other rigid materials.

Maintenance activities performed on JCP and JRCP projects included in the field study are summarized in Tables 3.11 and 3.12, respectively. Of 12 JCP sections, only 2 (JCP-1-IL and 6-CA) were patched, whereas 9 of the 18 JRCP projects were patched (JRCP-6-IL, 7-IL, 8-IL, 12-CAN, 13-NJ, 15-NY, 16-NY, and 18-IL). Patching on JRCP sections was at transverse cracks and transverse joints, and was usually necessitated by faulting or spalling of the cracks or joints. "D" cracking on three projects (JRCP-6-IL, 7-IL, and 8-IL) was corrected by patching. Small patches were also used on both JCP and JRCP sections where corner and diagonal slab cracking had occurred. Blowups or shattering in three projects (JRCP-12-CAN, 15-NY, and 18-IL) were corrected by patching.

Slab mud-jacking was performed on JRCP-4-TX, and mud-jacking using a lime slurry was used on JRCP-15-NY. One section (JRCP-13-MI) had localized joint and slab replacement using precast and cast-in-place concrete slabs.

In addition to these maintenance activities, extensive maintenance was performed on the shoulders of several JCP and JRCP sections. This activity is discussed under a separate heading. Other maintenance activities, such as surface treatment, grooving concrete to provide better skid resistance surface, grinding faulted joints (in JCP) and cracks to provide smoother ride, and installation of under drains as preventive maintenance are also used on JCP and JRCP projects.

Table 3.11. Maintenance Activities on JCP Sections Included in Field Study

Environment	Project No.	Project State	Distresses Requiring Maintenance	Types of Maintenance Received
WET + FREEZE	1	IL	CF, CS, TC, D, JF	P
	2	IL		
	3	IL	D	
	4	IL	D	
	11	NJ	JF	
WET	5	WA		
	12	GA	JF, JS	
DRY + FREEZE	9	UT		
DRY	6	CA	JF, TC, DC	P
	7	CA	JF	
	8	TX		
	10	AZ		

Distresses

JF: Joint Faulting
JS: Joint Spalling
CF: Crack Faulting
CS: Crack Spalling
TC: Transverse Cracking
B: Blowups
CC: Corner Cracking
LC: Long Cracking
D: "D" Cracking
PU: Pumping
DC: Diagonal Cracking

Maintenance

CF: Crack Filling
P: Patching with PCC or AC
JR: Joint Replacement
SR: Slab Replacement
M: Mudjacking
L: Lime Jacking

Table 3.12. Maintenance Activities on JRCP Sections Included in the Field Study

Environment	Project No.	State	Distresses Requiring Maintenance*	Types of Maintenance Received
WET + FREEZE	5	NJ		
	6	IL	TC, CF, CS, CC, D	P
	7	IL	TC, CF, JS	P
	8	IL	TC, CF, CC, D	P
	9	IL		
	10	IL		
	11	CAN		
	12	CAN	TC, CF, JS, B	P
	13	MI	CF, JS, TC, CC	P, JR, SR
	14	MI		
	15	NY	TC, CF, JS, LC	P, L, SR
	16	NY	JS, JF, TC, B	CF, P, SR
	17	NY	JS	
	18	IL	TC, B, CC, DC, CF, JS	P, CF
WET + NO FREEZE	1	TX		
	2	TX		
DRY	3	TX	PU, CC, LC	P
	4	TX	PU, TC, LC, DC	M, CF

* See Table 3.11 for distress and maintenance codes.

3.3 CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

3.3.1 General

A total of 12 CRCP projects are included in the field survey. A summary of the projects and their location is shown in Table 3.13. Traffic volumes for these pavements ranged from a high of approximately 35.7 x 10^6 18-kip ESAL for the Dan Ryan in Chicago, to a low of approximately 2.0 x 10^6 18-kip ESAL for several of the Texas pavements. The range in age for all 12 CRCP included is 5 to 27 years. Six of the 12 pavements were maintenance-free, 4 had localized maintenance, one had been overlaid, and one (CRCP-5-TX) was in need of extensive maintenance at the time of the survey. The pavements with both the highest and lowest traffic volumes were approximately the same age (9 and 13 years respectively), and both required maintenance.

In addition to the specific projects included in the field survey, the performance and problems associated with CRCP were discussed in detail with all engineers visited during the field surveys. More than 20 agencies and sub-agencies, such as highway departments and district offices, were visited.

3.3.2 Types and Causes of Distress

Types and causes of distress in CRCP are summarized and analyzed in this section. Specific types and causes of distress were developed from the 12 CRCP sections inspected during the field survey and from discussions with engineers from all of the agencies visited. Experience of the project staff

Table 3.13. Summary of 12 CRCP Projects.

Project No.-State	Traffic[1] $\times 10^6$	Age (Years)	Thickness (inches)	Subbase Type	Subbase Thickness (inches)	Steel (%)	Steel Type	Maintenance Required	Maintenance Type
1 IL	35.7	9	8	Granular	6	.61	Smooth Mesh	yes	Major Reconstr.
12 NJ	32.0	27	10	Granular	12	.72	Smooth Mesh	yes	Patching
2 IL	10.3	10	10	Cr. Stone	6	.60	DEF BAR[2]	no	
4 MI	7.7	9	9	A. C Granular	4 10	.70	DEF BAR	no	
5 TX	6.2	8	8	CTB	6	.55	DEF BAR	yes	Overlay Needed
11 TX	6.0	19	9	Granular	8	.55	DEF BAR	no	
8 TX	4.0	14	8	Granular	6	.55	DEF BAR	yes	AC & PCC Patches
9 TX	3.2	14	8	Lime Stab.	6	.55	DEF BAR	no	
7 TX	2.6	13	8	Granular	6	.70	DEF BAR	no	
10 TX	2.4	9	8	Lime Stab.	6	.55	DEF BAR	yes	Full Depth Patch
6 TX	2.0	13	8	CTB	6	.62	DEF BAR	yes	Patching w/AC
3 MN	2.0	5	9	Granular	6	.60	DEF BAR	no	

1. 18-kip SAL equivalent in heaviest traveled lane.
2. Deformed bar.

and published literature (36, 42, 79, 80, 81, 82, 127, 128, 136, 137, 142) were additional sources of information.

Typical types and causes of distress generally found in CRCPs are summarized in Figure 3.11. The major distress types can be broken into three main categories of cracking, surface distortion, and disintegration. These can be further broken down as indicated in Figure 3.11. The primary causative and contributing factors of distress in CRCP are shown in the figure.

Nearly all of the types of distress indicated in Figure 3.11, except "D" cracking, were observed during field visits to the 12 CRCP projects included as a part of this study. Some of the distress types were more common than others, and some pavements obviously had a greater concentration of a given type of distress than others. An analysis of the observed distresses and their probable cause is presented below. First, however, it is necessary to define the criteria used to classify the major distress types.

Distress Types. Distress on CRCP was classified according to sub-type and severity during the field survey. Most severity ratings were subjective, but some guidelines were developed to provide a uniform basis for rating by the project staff. The initial ratings were given by the senior staff member in the field at the time of the field survey. The ratings were then reviewed by the entire staff after viewing the slides and movies made of the pavement section. A discussion of the most common types of distresses and the basis for the severity rating is presented below. Severity was rated on a scale of zero to five with subjective ratings of none, minor,

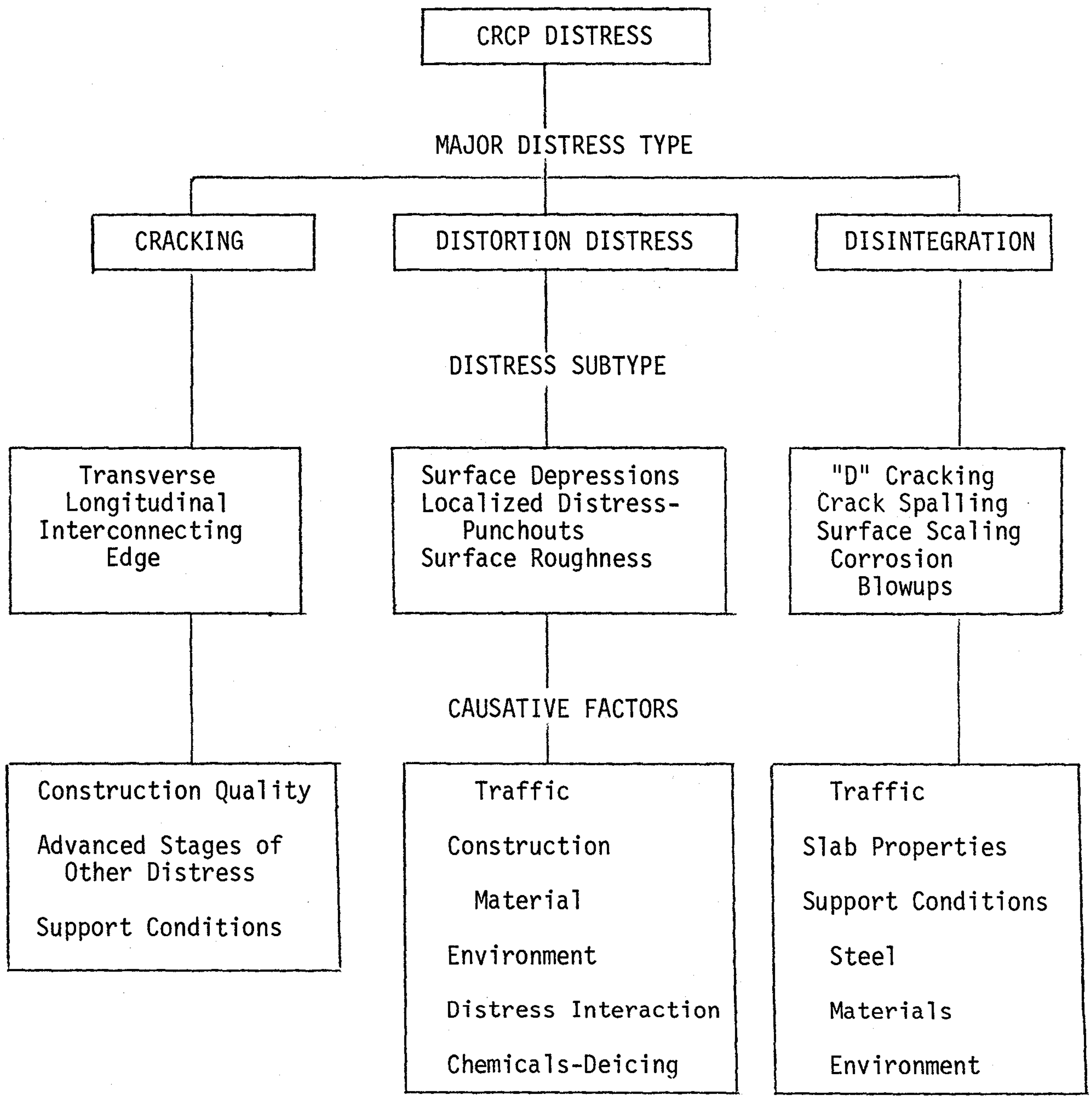


Figure 3.11. Distress and Causative Factors for Distress in CRCP.

moderate, major, and severe; 5 being the most severe. A summary of the guidelines for the severity rating of cracking distress is shown in Table 3.14.

Transverse cracking: This is one of the more difficult types of distress to classify as CRCP are expected to crack transversely, and all pavements surveyed had transverse cracks. The severity of transverse cracks was rated as "none" to "minor" if the spacing of the cracks was from 5 to 8 feet, and all cracks were tightly closed. "Severe" rating was given to those pavements with a close crack spacing and wide crack openings. Figure 3.12 shows an example of a CRCP with severe transverse cracking and patching.

Longitudinal cracking: Since CRCP do not normally have longitudinal cracks, this provides a logical basis for the zero end of the severity scale. The severity rating for longitudinal cracking was assigned as shown in Table 3.14.

Intersecting cracking: Transverse cracks usually do not cross the pavement in a straight line, but tend to meander. If the meander patterns are large, of if the transverse crack spacing is close, the transverse cracks may intersect. This produces cracks with "Y" formations and results in pavements significantly weakened against bending in the transverse direction. Guidelines for the severity rating of intersecting cracking distress are shown in Table 3.14.

Crack spalling: The phenomenon of crack spalling is well known to pavement engineers, and needs no further description here. The severity rating system used for this survey is given in Table 3.14.

Table 3.14 SUMMARY OF GUIDELINES FOR SEVERITY OF CRACKING DISTRESS ON CRCP

Distress Type	0 None 1 Minor	2 Moderate 3 Major	4 Severe 5
Cracking			
Transverse	cracks tightly closed little no spalling	cracks somewhat open some spalline	most cracks open and spalling
Longitudinal	less than 5% of pavement length	5 to 25% of pavement length	greater than 25% of pavement length
Intersecting	less than 5% of transverse cracks	5 to 10% of transverse cracks	greater than 10% of transverse cracks
Crack Spalling	less than 10% of cracks have no secondary crack which is a pre-curer to crack spalling (0-1). 0-10 percent of cracks have secondary cracks and some loose spalls are missing (1-2).	10-20 percent of cracks show secondary cracking with 25-50 percent of the concrete spalls missing (2-3). 20-50 percent of cracks show secondary cracking with missing spalls (3-4).	More than 50 percent of cracks spalled with spalls missing. Many spalls large enough to be seen and felt at 30 mph. (4-5).

Localized distress - punchout: McCullough (79, 80) separates localized distress and punchouts. It was found, however, that it was often very difficult to distinguish between a punchout and a localized distress, especially since the localized distress may lead directly to a punchout, and vice versa. Figure 3.13, 3.14, and 3.15 show some localized failures on CRCP. The failure shown in Figure 3.14 could also have been classified as a "punchout". Severity rating used for localized failures and punchouts was a linear scale varying from no localized distress for a zero rating to 2 or 3 per mile for a rating of "severe," depending upon the seriousness and size of the distressed areas.

Pumping: This type of distress was rated by evidence of fines being forced out along the edge of the pavement (edge pumping) and stains along the cracks (crack pumping). Severity rating was on a linear scale of from 0 for "none" to 5 for "severe" if pumping was observed at 4 or 5 locations along the test section. Severity ratings were also tempered by the apparent size of area involved and severity of the pumping action.

Steel rupture: Rupture of steel was evidenced by the width of opening of the transverse cracks. Severity was rated on a scale of 0 for "none" to 5 for "severe" if rupture occurred at a rate of one or more per mile of pavement.

Surface depression: Surface depression on CRCP is normally associated with pumping or consolidation of the subgrade. Severity was based on a scale of 0 for "none" to 5 for "severe" if they significantly affected the ride-ability of the pavement.

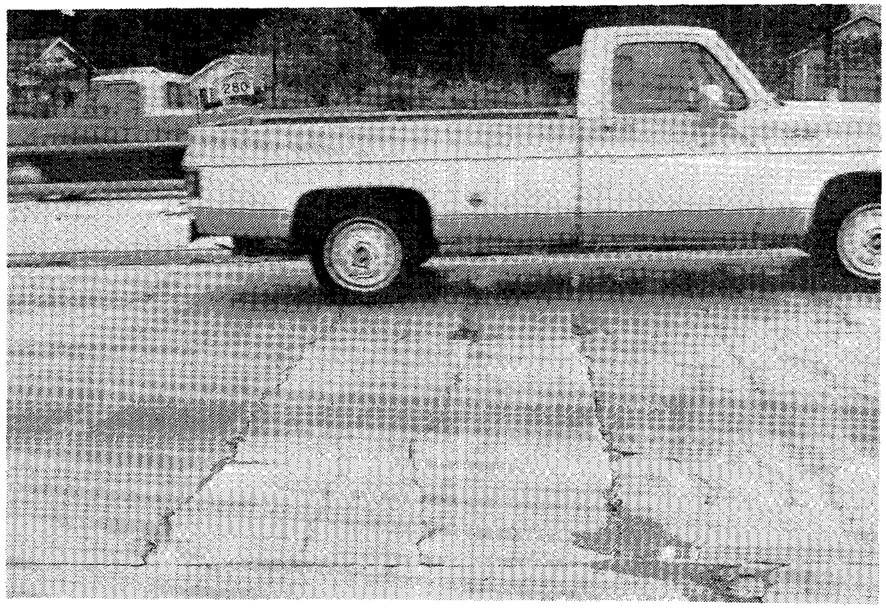


Figure 3.12. Illustration of severe transverse cracking on a continuously reinforced concrete pavement (CRCP-6, Houston, Texas).

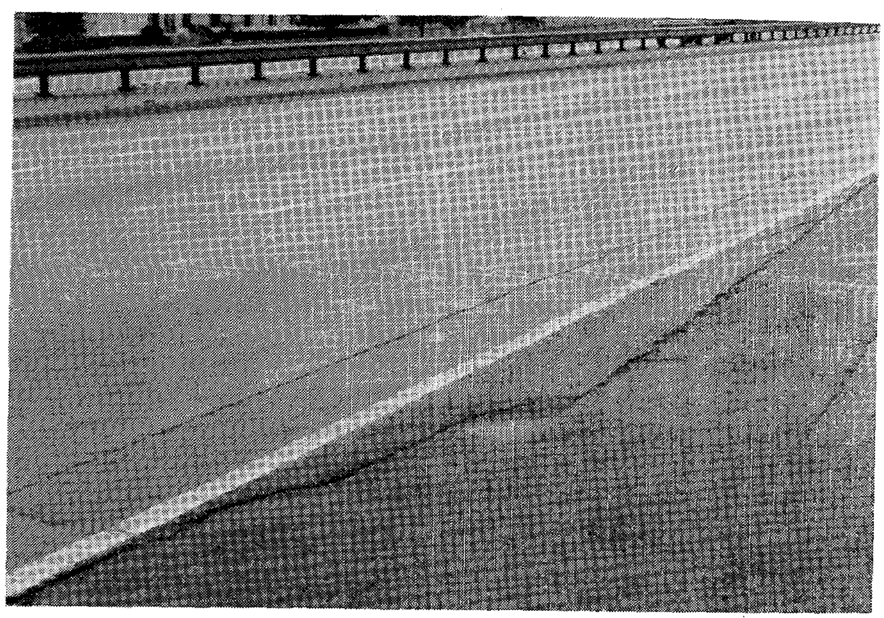


Figure 3.13. Illustration of edge distress on a continuously reinforced concrete pavement (CRCP-8, Dallas, Texas).

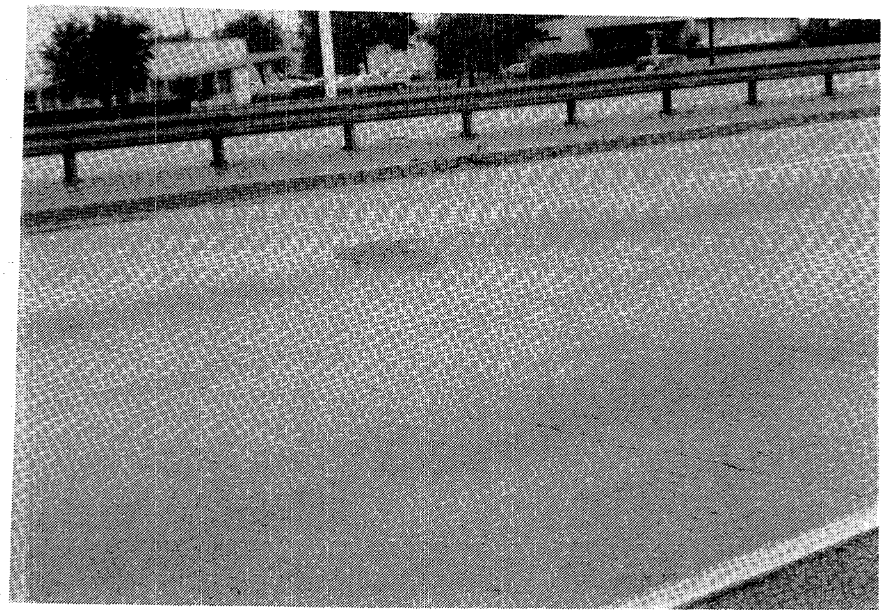


Figure 3.14. Illustration of a localized failure on a continuously reinforced concrete pavement (CRCP-8, Dallas, Texas).

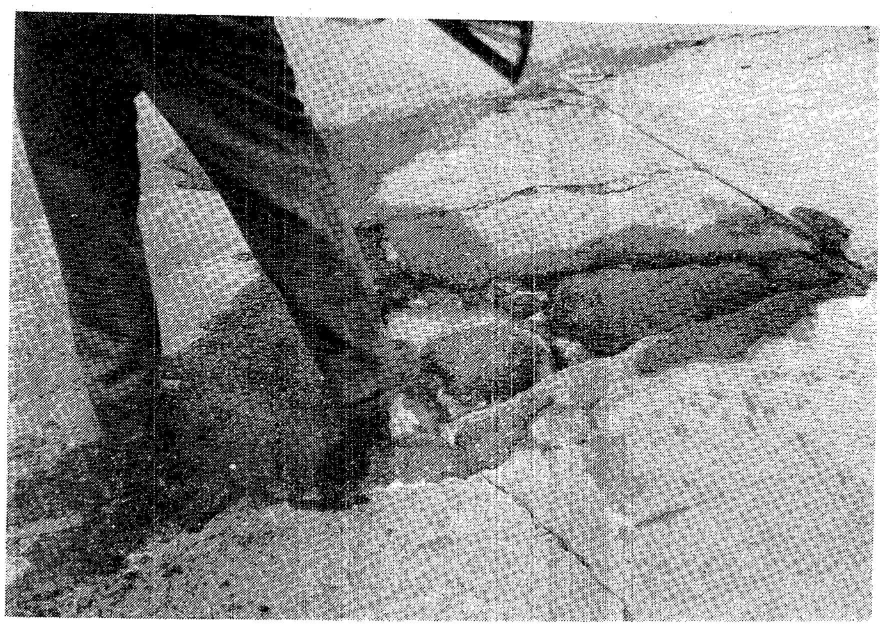


Figure 3.15. Illustration of a localized failure on a continuously reinforced concrete pavement (CRCP-6, Houston, Texas).

A summary of distress by type on the CRCP projects included in the field survey is shown in Table 3.15. The ratio of surveyed pavements with "moderate" to "severe" distress by type are shown in Table 3.16, along with a ratio of maintained to distressed pavements. A review of the values in these tables shows that (excluding shoulder distress) crack spalling, surface depression, interconnecting cracking, and punchouts are the most common types of distress. In addition, 5 of the 12 pavements surveyed also had repair patches, but these cannot truly be called a distress, but rather the effects of distress. It is reasonable to assume, however, that many of the repair patches were due to localized failures. Also, shoulder distress was not included as a part of this discussion as shoulder distress and its causes are discussed later in the report.

A rating of CRCP by distress type and severity alone is not sufficient as some types of distress are more critical than others. That is, when certain types of distress occur, maintenance is needed almost immediately if the pavements are to remain functional. The more critical types of distress in CRCP include 1) rupture of the steel, including steel splice failures and rupture due to corrosion; 2) punchouts; and 3) major subsidence. Major maintenance or repair must be undertaken as soon as possible when any of these distresses occur. The other types of distress such as edge pumping, surface scaling, and crack spalling are usually more long term in nature, and maintenance and repair activities can sometimes be forestalled without serious loss in serviceability. Obviously, the quicker that maintenance

Table 3.15. Summary of Distress by Type for the 12 CRCP Evaluated in the Field Survey.

	None	Minor	Moderate	Major	Severe		Total
	0	1	2	3	4	5	Surveyed
Longitudinal Cracking	9	1	1	1			12
Transverse Cracking	7	4	1	1			12
Interconnecting Cracking		8	4				12
"D" Cracking	12						12
Crack Spalling	2	4	3	2	1		12
Surface Scaling	10	2					12
Pumping (edge)	7	2	2	1			12
Pumping (cracks)	7	4		1			12
Repair Patches	5	2	4	1			12
Punch Out	6	2	4				12
Surface Depression	1	4	7				12
Shoulder Distress		6	4	2			12

TABLE 3.16

Summary of the Most Predominant Distress Types
Found on CRCP Rated Moderate to Severe

Distress Type	PROJECTS Distressed/Total	PROJECTS Maintained/Distressed*
1. Surface Depression	7/12	0/7
2. Crack Spalling	6/12	2/6
3. Punchouts	4/12	4/4
4. Interconnecting Cracks	4/12	2/4
5. Longitudinal Cracking	2/12	0/2
6. Steel Rupture	2/12	2/2

*Maintenance applied to only specific distress indicated.

is performed on any type of distress, the easier is the repair, and the greater the pavement salvage value. Pumping, in particular, will lead to much more serious distress if not corrected soon after it starts.

The major distress types were correlated with the maintenance activities and requirements for the 12 CRCP included in the field survey. Table 3.16 gives a summary of these correlations. Some interesting patterns can be seen by a study of the information presented in the table. Consider the distress labeled as "surface depression." According to Table 3.15, 7 of 12 CRCPs surveyed had "moderate" distress of this type. When this type of distress was correlated with pavement maintenance, it was found that of the 7 pavements with surface depressions, none had received pavement maintenance to correct this distress. Projects having interconnecting cracks to a moderate or more degree show 2 of 4 receiving maintenance.

Those pavements which had either localized distress (or punchouts) or steel rupture showed a strong correlation between this type of distress and maintenance activities.

In this type of analysis, care must be taken in the interpretation of the data. For example, while every pavement which had localized distress and punchouts also required maintenance, the converse of this is obviously not true. That is, not every pavement which required maintenance had localized distress. Obviously, this is due to the fact that other types of distress besides localized distress and punchouts caused maintenance to be performed.

Based on this analysis, the types of distress most often correlated with maintenance activities are: localized distress including punchouts, steel rupture, interconnecting cracks, and crack spalling. Attempts to combine types of distress such as the crack spalling and surface depression, and correlate them with maintenance activities, were not particularly successful.

Causes of Distress.

An analysis is presented on the causes of the four major types of distress.

The two projects included in the field survey which had ruptured steel were CRCP-1, the Dan Ryan, and CRCP-12, Route 130 in New Jersey. CRCP-1 consisted of 8 inches of CRCP on a granular subbase over a poor subgrade. The reinforcement consisted of 0.6 percent longitudinal steel in the form of a smooth welded wire mesh. The pavement, while only 9 years old when replaced in 1971, had nevertheless carried nearly 36 million 18-kip ESALs in each of the critical lanes. At the time of replacement, the distress consisted not only of ruptured steel, but of blowups, intersecting cracks, crack spalling, and other types of distress. CRCP-12, on the other hand, had 27 years of service at the time of the field survey, and had carried approximately 32 million 18-kip ESALs. Longitudinal steel for this pavement was also smooth wire, welded mesh with 0.7 percent steel. This pavement consisted of 10 inches of a CRCP slab on 12 inches of non-pumping granular subbase. Other forms of distress in this pavement consisted of some crack spalling and some localized distress. Maintenance of this section consisted primarily of patching of the localized distress areas with asphalt concrete. It is believed the higher than normal concrete quality flexural strength $>$750 psi) is partially responsible for the generally excellent performance of this pavement (141, 142). These two pavements were the only ones containing smooth wire mesh, and were the only ones to have steel rupture.

The problem of localized distress is believed to be related to loss of subgrade support for the slab and repeated traffic loadings. McCullough (80) in his analysis of this type of distress on CRCP in Ohio, stated:

> ". . .it was noted that these failure areas were generally depressed relative to the longitudinal profile. The extent of the depression can be detected by eye and is roughly equal to the distressed area of the pavement. Generally, stains of fine materials from the subbase layer can be detected on the shoulder surface."

This conclusion is supported by results from the field survey. Table 3.17, for example, shows the four CRCP projects in the survey with localized distress severity of moderate and higher. Also shown is the edge pumping severity rating for these pavements. With the exception of CRCP-12 (which is believed distressed mainly by steel rupture), the data show that where punchouts exist, edge pumping also occurs.

Additional surveys and examination of localized failures occurring on Interstate highways in Illinois indicates the following: (1) there is approximately the same amount of cracking in the truck lane as there is in the passing lane; however, nearly all of the localized failures occur in the truck lane, (2) localized failures generally occur between two closely spaced transverse cracks (the cracks seem to have widened to the extent that aggregate interlock was partially lost), (3) the cause of the crack widening may be corrosion of the reinforcement (which was observed at many failure areas) and/or debonding of the concrete near the cracks due to high stresses, (4) a vertical shearing of the reinforcement at the cracks was noted at some failure areas which may occur after the crack widens excessively and aggregat- interlock is lost, and (5) the concrete between the wide transverse cracks soon becomes cracked longitudinally and then the entire rectangular block generally punches downward with additional traffic to as much as an inch or more. Hence, the localized failure mode of CRCP is quite different from jointed concrete pavements.

Longitudinal cracking has been linked with loss of subgrade support at levels in the subgrade (141). Data from the CRCP in the field study do not invalidate this conclusion, but also add nothing to support it. Two pavements for which the longitudinal cracking was rated moderate or higher were CRCP-5 and 6, both in Houston, Texas. In both cases, the traffic was relatively light (2 and 6 million 18-kip ESALS, respectively), and only CRCP-6-TX had any significant edge or crack pumping. No evaluation was made on the possible deep soil movement for these pavements.

Of the remaining types of distress, surface depressions and crack spalling were the most frequent, being observed on 7 and 6, respectively, of the 12 CRCP projects surveyed. These two types of distress, however, did not correlate strongly with the maintenance activities on the pavements. This is also true for pavements with interconnecting crack patterns.

While some types of distress did not correlate strong with maintenance activities and maintenance needs for CRCP, it is obvious that most of the indicated distress types will have to be substantially eliminated if zero-maintenance CRCP are to be realized. For example, while none of the pavements included in the field survey had any significant amount of "D" cracking, this type of distress has been observed on a number of CRCP with disastrous results. Similarly, of the pavements surveyed, none of them had ruptured steel which could be attributed directly to corrosion or the

Table 3.17. CORRELATION OF PUNCH-OUT DISTRESS IN CRCP WITH EDGE PUMPING AND TRAFFIC

Pavement Designation	State	Traffic (Eq. 18^k SAL) x 10^6	Distress Rating: Punch-Out	Distress Rating: Edge Pumping
10	TX	2.4	Moderate	Major
8	TX	4.0	Moderate	Moderate
6	TX	2.0	Moderate	Major
12	NJ	32.0	Moderate	None

effects of deicing agents. Review of the literature (36, 178), however, and personal observations of corrosion in pavements near Detroit, Michigan, and Chicago, Illinois (5-10 years old), show the potential seriousness of the problem. Also, while crack spalling did not correlate well with maintenance activities, it is clear that significant spalling must be eliminated if zero-maintenance pavements are to result.

Causes of distress in CRCP are many and varied, and time does not permit a detailed discussion of all types and causes in this report. While data from the 12 CRCP sections included in the field study do not necessarily reinforce previous concepts in all instances, none of these data indicate the previous concepts to be significantly in error.

Faiz and Yoder (42), McCullough (79, 80), McCullough and Treybig (81, 82), and others (91, 127) suggest the following factors to be the major causes of distress in CRCP:

1. Pavement support, particularly the subbase;
2. Construction, construction controls, and materials used;
3. Traffic, especially the heavy truck; and
4. Design factors such as slab thickness, type of steel, amount of steel, and location of steel.

Correlation of the maintenance activities and maintenance needs for the 12 CRCPs included inthe field study for factors such as edge pumping and punchouts indicates that one of the most significant factors in performance of CRCP is the subbase used. Pavements which showed evidence of significant edge pumping generally required maintenance.

In general, stabilized subbases are more reliable for eliminating pumping than the granular subbases. However, it should be noted that when particular care is taken to insure that the granular subbase materials are non-pumping, i.e., have few fines, as for CRCP-12 (New Jersey Route 130), there is no evidence of pumping after 27 years of heavy duty service (142). This pavement section has a 10-inch CRCP slab which may, of itself, reduce pumping compared to an 8-inch slab.

A number of investigators have suggested that construction controls and practices are a major factor in the distress on CRCP (42, 79, 80, 91, 127, 128). Time did not permit a detailed evaluation of this aspect of the distress in CRCP included in the field survey. It was obvious, however, from the patterns of distress observed, especially the localized distress, that much of it was related to variations in the pavement system. It was also noted that the high concrete strength in CRCP-12-NJ (core strength in excess of 4500 psi, flexural strength greater than 750 psi) (140, 141, 142) is probably responsible for the longevity and relative lack of crack spalling for a pavement 27 years old and having carried over 32 million 18-kip ESALs. Thus, while data from this study do not reinforce the concept that construction practices and controls are a major factor in the performance of CRCP, the data do lend credence to this concept, and support it indirectly.

A number of investigators have concluded that traffic volume is a major factor in performance of CRCP. CRCP project included in the field study for this project are listed in Table 3.13 in a decreasing order of traffic volumes expressed in terms of 18-kip ESAL. Traffic volumes for these 12 pavements range from nearly 36 million to only 2 million 18-kip ESAL, but both the pavements with the heaviest and lightest traffic volumes required maintenance after 9 to 13 years of service.. However, CRCP-1-IL required excessive maintenance, while CRCP-6-TX required only a small maintenance effort. The other 4 pavements requiring maintenance prior to the survey had traffic counts between the 2 and 36 million and were interspaced with the 6 pavements which carried from 2.0 to 10.2 million 18-kip ESALs without maintenance.

Thus, while these data do not substantiate the concept that heavier traffic is a major cause for distress in CRCP, they do not disprove this thesis, either. These data do indicate there are factors other than traffic which have a major impact on the maintenance needs of CRCP.

There appears to be a strong correlation between slab thickness and maintenance-free performance of CRCP. Two of the pavements included in the field survey had thicknesses of 10 inches, and 3 had thickness of 9 inches. The performance of the 9- and 10-inch pavements in terms of maintenance-free service is significantly better than that of the 8-inch pavements, as none of the 9-inch pavements required maintenance, and only one of the 10-inch did. Also, both 10-inch pavements had heavy traffic volumes, as CRCP-12-NJ carried 32 million 18-kip ESALs over a 27-year period with only minor patching and CRCP-2-IL carried 10.3 million 18-kip ESAL over a 10-year period with no maintenance on the pavement slab. Also, most, if not all, of the distress on CRCP-12-NJ was associated with rupture of the smooth wire fabric, a product which has proven unsatisfactory in other pavements. The steel was placed in two layers with the top steel in CRCP-12 placed nearer the pavement surface (2 inches) than is generally recommended in current practice. It is also significant that none of the 3 CRCPs having a 9-inch slab required maintenance. Thus, there is evidence that thicker CRCP will provide substantially longer and more reliable maintenance-free service than the thinner slabs. This is further substantiated by sections of the Dan Ryan not included in this study which are 10 inches thick and have carried over 16 million 18-kip ESAL for over 4 years with no distress or maintenance.

3.3.3 Maintenance Activities

Except for CRCP-1-IL which has undergone major reconstruction and CRCP-11-TX which was overlaid with a friction course, the only type of maintenance activity applied directly to the CRCP was patching, either in the form of filling distressed areas with asphalt concrete, or full depth concrete patches in areas of localized distress. Some patches extended the full width of a pavement lane whereas others were very localized covering a few square feet or less. There was also some maintenance of the shoulders and sealing of the shoulder joint adjacent to the CRCP slab.

Patching of CRCP with asphalt concrete is generally not a permanent installation, but serves merely to improve tempoerarily the serviceability of the pavement in areas of severe localized distress. The only permanent type of patching for CRCP is full depth concrete patches, the installation of which seriously disrupts traffic for some period of time. Also, full depth concrete patches, improperly installed, may give poor performance which results in continuing maintenance activities.

3.4 FLEXIBLE PAVEMENT

3.4.1 General

The nineteen flexible pavements included in the field survey are located in 9 states with widely ranging environments, and have moderately high traffic volumes with two-directional ADT generally ranging from 14,000 to 35,000, but with one project having an ADT of 138,500. Total 18-kip ESAL applications in the heaviest traveled lane ranges up to 12 million. The projects vary in age from 7 to 24 years with a mean of 14 years. Only 9 of the 19 projects showed maintenance-free performance. In addition to these projects, other pavements in the various regions visited were also observed, and the general performance characteristics of flexible pavements in the region discussed with resident pavement engineers.

3.4.2 Types and Causes of Distress

The major types of distress occurring on high traffic volume flexible pavements are summarized in this section. Specific causes of these distresses are also identified from interviews with engineers in the various regions visited, observations of the project staff during the field survey, literature surveys, and analytical analyses of pavement systems in the field survey.

Distress Types. Previous research studies have identified and defined numerous types of distress that occur in flexible pavements. Studies by Barenberg, Bartholomew, and Herrin (13), those reported in the Transportation Research Board Special Report 113 (59), and Shahin and Darter (110), identify 24 different types of distress. Most of the identified distresses were found in high traffic volume pavements included in the survey, but a few were not. Specifically, the distresses identified as abrasion, corrugation, block cracking (large blocks greater than 2 ft on a side), slippage, imprint indentation, shoving, streaking, significant waves, and serious upheaval were not observed on any of the pavements surveyed.

The distresses and their level of severity observed in the field study are summarized in Table 3.18. Each distress type occurring on a given project has been given a subjective rating by the project staff during the field visit. The ratings are subsequently reviewed by the entire staff using slides and 8 mm movies taken of each project during the field visit. The general definitions of the rating are given in

Table 3.18. Summary of Flexible Pavement Projects With Distress Type and Severity Rating

Distress Type	Distress Severity**					Total Surveyed
	0 none 1	minor 2	moderate 3	major 4	severe 5	
Alligator Cracking	6*	5	1	3	4	19
Edge Cracking	9	9		1		19
Longitudinal Cracking (Jt)	5	3	6	3	2	19
Transverse Cracking	3	6	2	5	3	19
Slippage Cracking	19					19
Depression	5	11	3			19
Rutting		13	3	2	1	19
Upheaval	19					19
Corrugation	19					19
Potholes	17	1	1			19
Bleeding	16	3				19
Polished Aggregate	1	10	7	1		19
Raveling	13	5		1		19
Weathering	2	13	2	2		19
Shoulder Distress	2	4	6			12***
Reflection Cracking	15		1	2	1	19

*Each number represents the number of projects in this distress category and severity rating.

**As defined in Table 3.19.

***Seven of the projects had gravel shoulders, so this distress type could not be evaluated.

Table 3.19 for the four major distress types. Other distress severity ratings are based on the subjective evaluation of the project staff with respect to the effect on pavement structural integrity and surface condition.

<u>Major Distress Types</u>. The major distress types (rated "moderate" to "severe") occurring on high traffic volume flexible pavements are summarized in Table 3.20 in terms of percentage of occurrence. Longitudinal cracking is the major distress type on more project than any other type.

Some types of distress are more critical than others in requiring maintenance. For example, alligator or fatigue cracking occurred on 9 of 19 projects, and 5 of the 9 received maintenance for that distress type. The major distress types occurring on the flexible pavement projects that normally received maintenance include longitudinal cracking, transverse cracking, alligator or fatigue cracking, rutting (if "severe"), and combinations of these distress types.

Further observation of the field data show that all projects requiring maintenance had a distress rating of "severe" for one or more of the four most critical distress types. Hence, it appears that a combination of these four types of distress in flexible pavements usually leads to maintenance activities being performed.

Based upon data from the field survey, and a review of the literature, the major distress types occurring in heavily trafficked flexible pavements that require or receive maintenance and their level of severity are as follows:

Table 3.19. Summary of Definitions Applied to Obtain Distress Ratings for Major Distresses for Flexible and Composite Pavements.

Distress Type	Minor	Moderate	Major	Severe
Alligator (fatigue) Cracking	Class 1 (minimal)	Class 1	Class 2	Class 3*
Longitudinal Cracking	<25% length, tight cracks	<25% length, significant spalling	>25% length, minor or no spalling	>25% length, significant spalling or patched cracks
Transverse Cracking (including reflective cracking)	>76 ft spacing	31-75 ft spacing	16-30 ft spacing	1-15 ft spacing
Composite Pavements Only	minor spalling	moderate spalling	major spalling	severe spalling
Rutting	<1/4 in.	1/4-1/2 in.	>1/2-3/4 in.	>3/4 in.

*AASHO Road Test Definitions (Ref. 57).

TABLE 3.20

Summary of Distress Types Found on
Flexible Pavements Rated as Moderate to Severe

Type of Distress	PROJECTS Distressed/Total	Maintained/Distressed
1. Longitudinal Cracking (lane joint in nearly all cases)	11/19	5/11**
2. Transverse Cracking (including reflective)	10/19	7/10
3. Alligator (fatigue) Cracking	9/19	5/9
4. Polished Aggregate	8/19	0/8
5. Rutting	6/19	1/6
6. Weathering Asphalt	4/19	0/4
7. Depressions	3/19	0/3
8. Alligator or Transverse Cracking*	14/19	9/14
9. Alligator or Transverse or Longitudinal Cracking or Rutting*	17/19	10/17

*Whichever of the distress types was rated highest for each pavement.
**Maintenance performed only for distress indicated.

Alligator or fatigue cracking: This type of distress occurs mainly in the wheel path with severity levels of Class 2 and 3. Typical fatigue cracking is shown in Figures 3.16 and 3.17 for flexible pavements with granular base layers, and in Figures 3.18 and 3.19 for pavements with cement stabilized base layers.

Transverse cracking: This type of distress occurs transverse to the longitudinal axis of the roadway, and includes low temperature as well as reflective cracking (resulting from a cement treated base, for example), with spacing of less than 30 ft and possible spalling.

Longitudinal cracking: This type of distress almost always occurs at the lane construction joint, often over more than 25% of the length of a project, and is spalled. Typical examples of transverse and longitudinal cracking in flexible pavements are shown in Figure 3.20.

Rutting: This type of distress occurs in the wheel paths, and normally a depth of 1/2 inch or more requires maintenance. A typical example of rutting is shown in Figure 3.21.

Besides four major distresses previously identified on the projects surveyed, other distress types that occur in varying amounts and degrees depending on construction, foundation soil, materials, environment, etc., are as follows:

Surface deterioration: Includes aggregate polishing, bleeding, weathering, and studded tire wear. Skid resistance and hydroplaning were not under consideration in this study, but can be major problems with flexible pavements.

Figure 3.16. Illustration of alligator (fatigue) cracking distress in wheelpaths of flexible pavements (FLEX-17, Arizona).

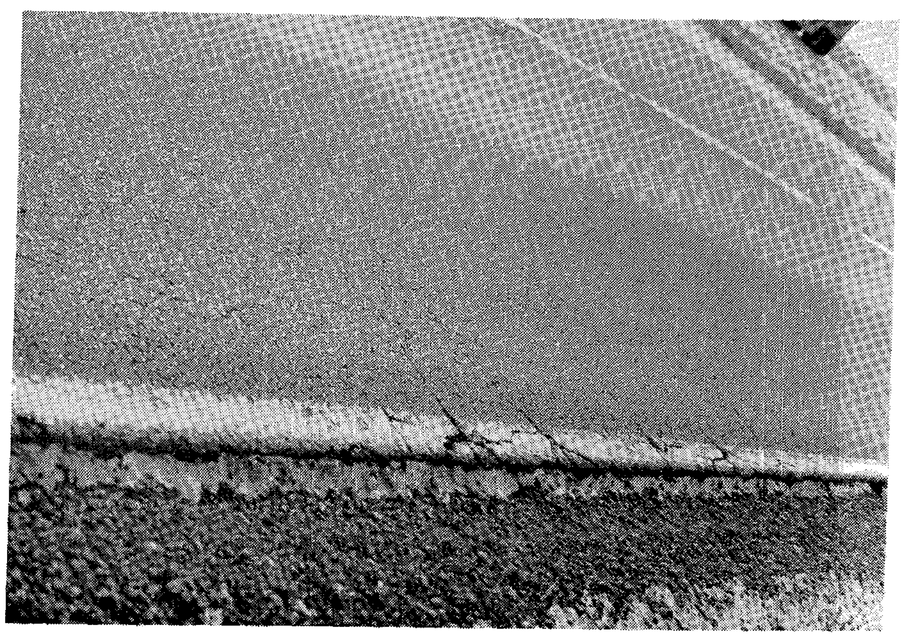


Figure 3.17. Illustration of alligator (fatigue) cracking distress in wheelpaths of flexible pavements (FLEX-7, Illinois).

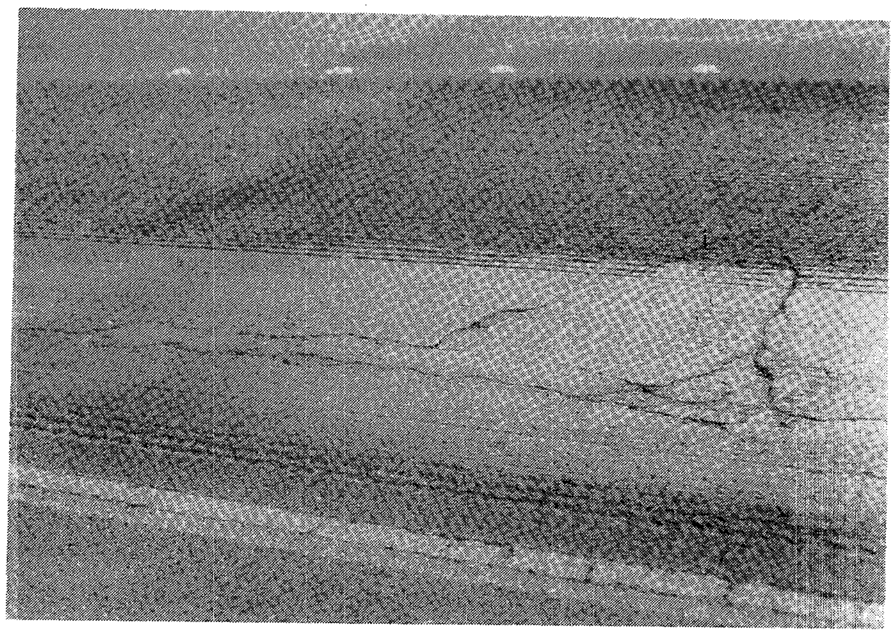


Figure 3.18. Illustration of fatigue cracking distress for flexible pavement with cement treated base (FLEX-13, California).

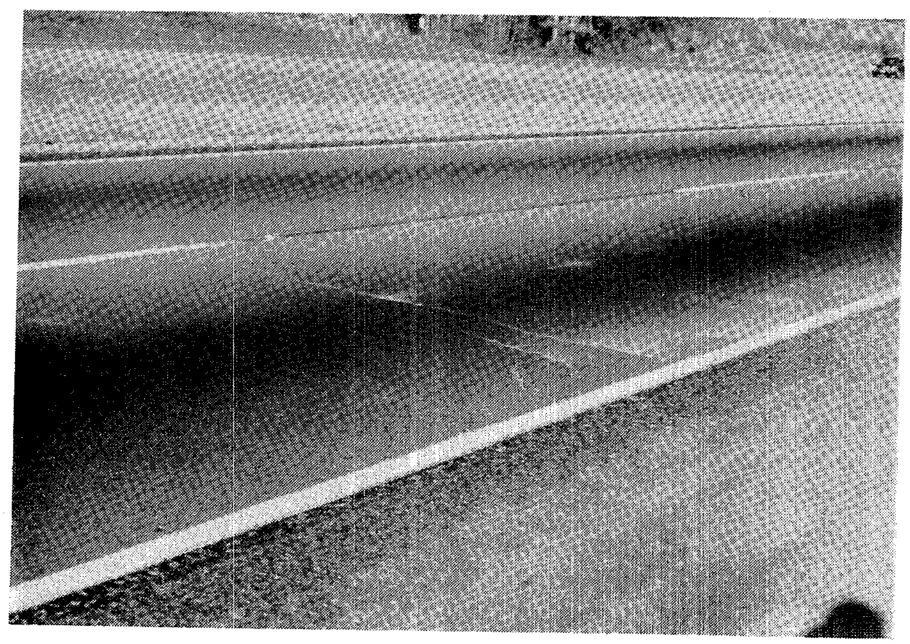


Figure 3.19. Illustration of fatigue cracking distress for flexible pavement with cement treated base (FLEX-19, Georgia).

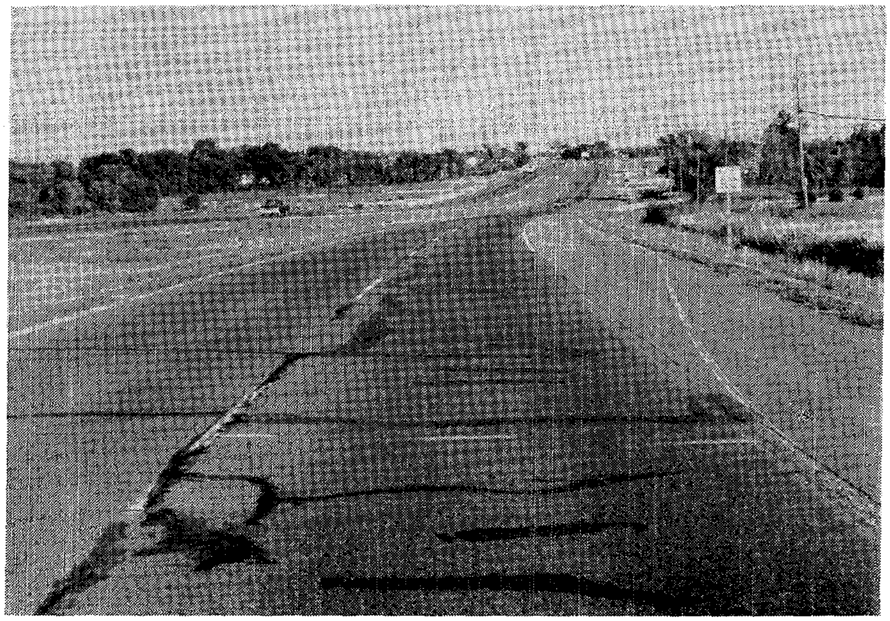


Figure 3.20 Illustration of transverse and longitudinal cracking of flexible pavement (FLEX-4, Minnesota).

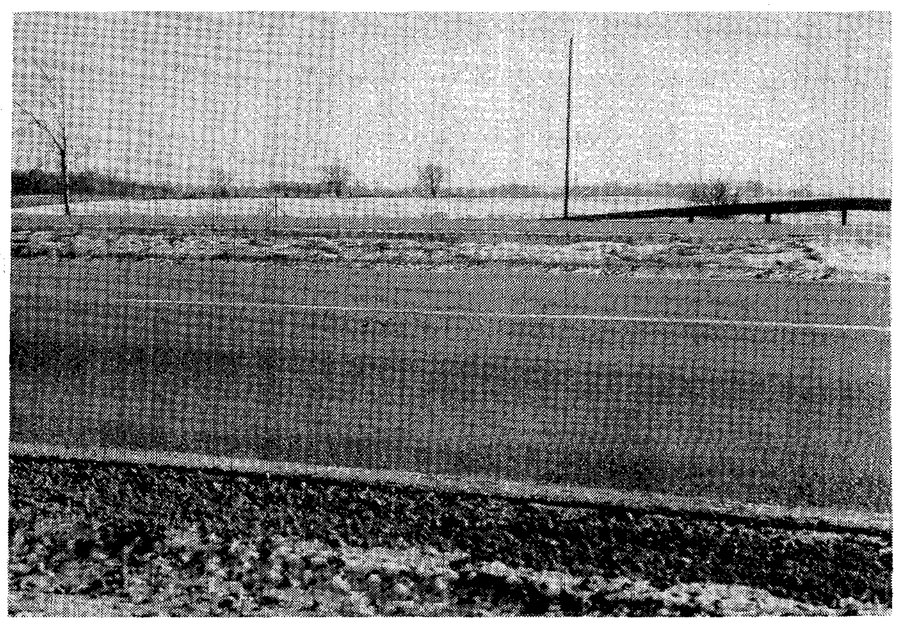


Figure 3.21. Illustration of rutting of flexible pavement (FLEX-8, Illinois).

Swelling or heaving foundations: Certain regions of the country have serious problems with swelling and heaving of the subgrade. This distress may cause serious loss in serviceability, and consequently, the pavement may require maintenance (129).

Shoulder distress: Shoulders on all types of pavements surveyed showed distress. The entire approach to design and maintenance of shoulders needs review, but this subject is beyond the scope of this paper.

Depressions: Settlement of noticeable degree was found on only 3 of the 19 projects. Most high type flexible pavements are constructed with a significant pavement thickness over the subgrade which tends to minimize this type of distress.

Asphalt stripping: Stripping has occurred in some areas, and is a serious problem, causing rapid deterioration of the asphalt concrete pavements. Basically, it is due to the breaking of the adhesive bond between the aggregate surface and the asphalt cement resulting in cracking and surface raveling (47).

<u>Causes of Distress</u>. Based upon field observations and analytical studies, the causes are identified for each of the major distress types previously listed.

<u>1. Alligator or Fatigue Cracking:</u> The major cause is repeated traffic loading which induces tensile stresses and strains in the bottom of the asphalt concrete surface layer if the pavement has a granular base, or at the bottom of the base layer if the base is stabilized with cement, asphalt,

or lime. If the tensile stresses and strains under the wheel loads are of sufficient magnitude, and the number of load applications is high, fracture can occur near the bottom of the critical layer, and progress upward with additional load applications until it eventually reaches the surface (157). Eight of the flexible pavements included in the field study showed "moderate" to "severe" alligator cracking. Of these eight, four had cement treated bases, two had asphalt treated bases, and two had granular bases.

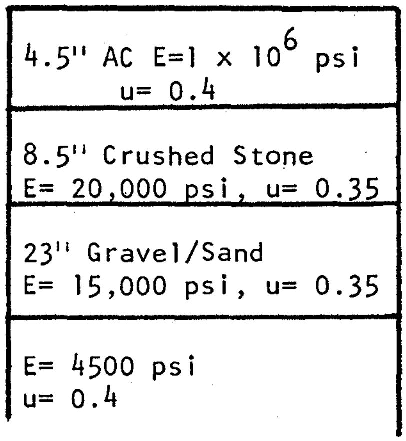
For a number of the flexible pavements surveyed, an analysis was conducted with the objective of determining the cause of alligator cracking by predicting fatigue damage distress using elastic layered theory and Miner's fatigue damage expression (Equation 4.6, presented later in this report). The analysis was based on several simplifying assumptions as the analysis is intended only to illustrate that fatigue is an important factor in alligator cracking of flexible pavements, and not to recommend a method of analysis. The assumptions made in conjunction with this analysis are as follows:

a. The resilient modulus and Poisson's ratio were determined for each pavement layer and for the subgrade from either laboratory tests (for some of the projects), or from the results of other studies (107, 119, 88). Since resilient modulus values for granular and fine-grained soils are stress dependent, values were selected which closely corresponded to the range of expected bulk and deviator stresses in the pavements. The stiffness of asphalt concrete was determined from the stiffness/temperature data given in the literature (119), and was selected as a constant value corresponding to the mean annual temperature for each region.

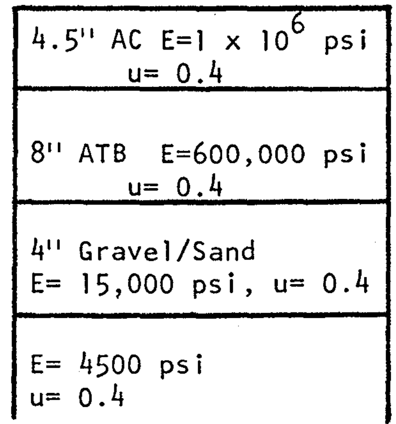
b. The traffic loading used in the fatigue analysis is the accumulated 18-kip ESAL applications estimated for the heaviest traveled lane of each project over its design life. The use of equivalent load applications instead of the actual load distribution is not exact, but is sufficiently reliable for this analysis, and greatly reduces the work required.

c. Fatigue curves of initial bending strain versus load applications developed by Shell researchers (40) using the controlled stress mode of testing were used for asphalt concrete. A fatigue curve of strain versus load applications for cement treated bases, published by Monismith and Finn (88), was used.

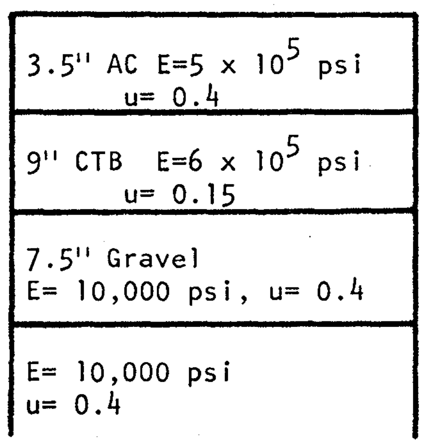
Calculations for three projects, FLEX-7, FLEX-8 from the AASHO Road Test Site in Illinois, and FLEX-19 from Atlanta, Georgia, are used to illustrate the analysis. The specific resilient modulus, Poisson's ratio, and pavement sections used are given in Figure 3.22. The critical radial tensile strains and stresses were computed under a 9-kip wheel load using the linear elastic layered system analysis program. These values are given in Table 3.21 for each project analyzed. Fatigue damage was computed using estimated 18-kip ESAL applications accumulated over the life of the pavement.



(a) FLEX-7



(b) FLEX-8



(c) FLEX-19

Figure 3.22. Pavement sections and materials properties used to compute critical stresses/strains in example flexible pavements.

TABLE 3.21 SUMMARY OF FATIGUE ANALYSIS OF 15 FLEXIBLE PAVEMENTS INCLUDED IN FIELD SURVEY

Project No.	State	Layer	Critical Stress psi	Critical Strain in./in. x 10^{-4}	n, Actual 18 kip ESAL x 10^6	N, Allowable* 18 kip SAL x 10^6	$D = \Sigma \frac{n}{N}$	Alligator Cracking Observed
2	NJ	APM	66	0.44	12.40	100.00	0.12	No
3	MN	AC	246	1.97	2.01	.31	6.48	Yes
4	MN	AC	144	0.52	1.91	60.00	0.03	No
7	IL	AC	249	1.54	3.87	0.80	5.88	Yes
8	IL	ATB	69	0.70	3.97	13.00	.37	No
9	IL	ATB	54	0.55	3.89	45.00	0.09	No
10	IL	CTB	48	0.36	4.60	10.27	.54	Q**
11	WA	AC	111	0.70	1.66	13.00	0.13	No
13	CA	CTB	83	0.50	8.81	1.66	5.31	Yes
14	CA	AC	125	1.60	2.27	1.00	2.27	Yes
15	CA	CTB	66	0.40	3.70	6.20	0.60	No
16	UT	CTB	83	1.20	2.75	0.00	>100.00	Yes
17	AZ	AC	231	3.00	4.27	0.06	71.20	Yes
18	GA	AC	70	0.88	4.00	8.00	0.50	No
19	GA	CTB	70	0.99	2.38	0.17	14.00	Yes

* According to fatigue considerations (Ref. 88,40,43,24)

** Severe durability disintegration occurred in this CTB and it is difficult to ascertain the cause of distress.

ATM: Asphalt penetration Macadam base; AC: Asphalt concrete

ATB: Asphalt treated base; CTB: Cement treated base

Analysis for FLEX-7 shows relatively large radial tensile stresses and strains (1.54×10^{-4} in/in) in the bottom of the asphalt concrete surface. Using the fatigue curve for asphalt concrete (constant stress mode), the number of load applications of a 9000 pound wheel load to failure is approximately 800,000. The number of 18-kip equivalent single axle load applications is 3,870,000. Hence, the analysis indicates a fatigue failure. As confirmation, FLEX-7 has extensive alligator (fatigue) cracking, and is currently being overlayed.

FLEX-8 is a pavement section adjacent to FLEX-7 on I-80 in Illinois, but has a deep strength asphalt treated base. The computed strain in this section is 7.04×10^{-5} in/in, which corresponds to an allowable number of 9000 pound wheel applications of approximately 13 million, compared to the 3.97 million applications to which the pavement was subjected. There is, as expected, no observable fatigue cracking on the surface of this project.

FLEX-19 has a cement treated base that showed serious fatigue failure in the field, and the computed results agree with this observation.

Similar calculations were made for 15 flexible pavements with the results given in Table 3.21. The analysis of the 8 projects which showed evidence of fatigue cracking showed excessive stresses or strains, and an accumulated fatigue damage of greater than 1.0. Theoretically, these projects should show fatigue distress, and they do. All of the projects (7 total) not showing fatigue damage had cumulative damage ratios of less than 1.0. Hence, the analysis appears to be reasonable (even under the assumptions made) since fatigue or alligator cracking occurred on all projects where analysis indicated this distress type to be likely, and where the analyses indicated there should be no fatigue distress, there was none.

Five of the projects in the field study had cement treated bases, and four of the five showed severe fatigue cracking. The computed stresses/ strains in the four pavements that failed indicate that fatigue damage is likely.

Based on the results of this analysis, it is concluded that the cause of fatigue distress is one or more of the following:

1. Failure of the structural design procedure to provide adequate thickness of the surface or other pavement layers to keep the tensile strains within allowable limits for the applied traffic loads.

2. Loss of support in various pavement layers, or subgrade, due to excessive moisture, shear failure, etc.

3. Gradual hardening or aging of the AC surface with time, resulting in decreased fatigue life.

Alligator (fatigue) cracking has been studied in numerous field studies by others (57, 60, 157), and identified as a major distress type which must be prevented if maintenance-free life is to be achieved. A general observation from results of this field survey is that fatigue distress seems to occur more often in warmer climates. Consider, for example, FLEX-4 in Minnesota and FLEX-14 in California. The pavement structures for these two pavements are roughly equal, both having a 7-inch asphalt bound upper layer over granular subbase. The applied 18-kip ESAL applications are roughly equal, as are their ages (FLEX-4 is 13 years old, and FLEX-14 is 9 years old). FLEX-4 in Minnesota shows no signs of load associated fatigue cracking (but does have temperature transverse cracking), whereas FLEX-14 in California shows "moderate" to "major" fatigue cracking. The computed cumulative fatigue damage shown in Table 3.21 indicates fatigue distress for FLEX-14 (i.e., D = 2.27), but no fatigue distress for FLEX-4 (i.e., D = 0.03).

The alligator type cracking has also been attributed, in a few instances, to hardening or shrinkage of the asphalt concrete (11, 41), which may be related to absorption of asphalt into highly absorptive aggregates (41). This phenomenon appears to be a rare occurrence, however, on high type flexible pavements, especially when compared to the extent of identifiable load associated fatigue distress.

2a. Transverse Cracking (non-reflective): This type distress which is usually manifest by crack spacings of from 10 to 150 ft, is usually associated with low temperatures found in the northern freeze areas of the U. S. The surveyed projects that exhibit this distress are in New Hampshire ("severe" cracking), Minnesota ("severe"), Illinois ("major"), Utah ("major"), Michigan ("severe"), New Jersey ("moderate"), and Ontario, Canada ("severe"). This distress was not observed (or observed only in minor amounts) in the areas visited in central and southern Texas, western Washington, California, Arizona, and Georgia. Specifically, projects located in areas designated as freeze areas (freezing index greater than 100) in Table 2.1 exhibited significant transverse cracking while those designated as non-freeze areas (freezing index less than 100) did not show transverse cracking. There obviously are exceptions to this conclusion, such as occurrence of transverse cracking in western Texas.

While most researchers have attributed the transverse cracking distress to low temperatures, Shahin and McCullough (109) concluded that transverse cracking can be attributed to both low temperature shrinkage and to thermal fatigue. Thermal fatigue has been shown to occur for asphalt concrete in the laboratory by thermally cycling restrained beams (by Tuckett, Darter, et al (65, 139)). Considerable research has been performed to determine the causes and cures for this distress type, as in northern regions of the U. S., it is probably the most predominant distress type for flexible pavements. As indicated in the field survey, 6 out of 10 projects located in freeze areas showed transverse cracking as the most severe distress occurrence.

2b. Transverse Reflective Cracking: Five projects containing cement treated bases are included in the field survey. All projects but one (FLEX-16-UT) showed extensive transverse reflective cracking. The cause was definitely attributed to reflective cracking since other types of flexible pavements in the same areas do not exhibit transverse cracking. Crack spacing varies as follows:

FLEX-10: 20 ft (Ottawa, Illinois)

FLEX-13: 15 ft (San Franscisco, California)

FLEX-15: 50 ft (Los Angeles, California)

FLEX-19: 15 ft (Atlanta, Georgia)

The specific cause of this distress is believed to be a crack initiating in the cement treated base course which reflects through the asphalt concrete to the surface. This is a relatively serious distress which has required maintenance in several of the projects surveyed. One project in particular (FLEX-10-IL), located in a wet-freeze thaw area, was subjected to heavy applications of deicing salts (about 10 tons/lane/mile/year) and approximately 12 freeze-thaw cycles per year. On this project, cement treated base disintegrated at each transverse crack resulting in a depression of the AC surface, and serious alligator cracking and spalling of the surface cracks as shown in Figure 3.23.

3. Longitudinal Cracking: This distress type which existed on 11 of the 19 surveyed projects, almost always occurred at the lane construction joint near the lane strip marking. The basic cause was usually attributed to poor joint construction practices. During the construction, the joints are allowed to cool, usually without compaction, until the paver passes in the adjacent lane. This results in asphalt concrete at the joint being less dense after compactions, and therefore, possessing less tensile strength. During periods of low temperatures, shrinkage tensile stresses develop in the pavement with the maximum stress at the centerline joint for pavements with 2 lanes or more. Since the material is relatively weak at this joint, the pavement fractures after one or two winters, and a longitudinal crack results. Also, the lower densities result in a more permeable surface, and hence, greater water infiltration of the joint. This infiltration also increases salt infiltration, and a greater tendency toward deterioration of the base under freeze-thaw action.

4. Rutting: This distress type occurred on 6 of the 19 projects surveyed to a "moderate" to "severe" level (1/4 to 3/4+ inches). Rutting was

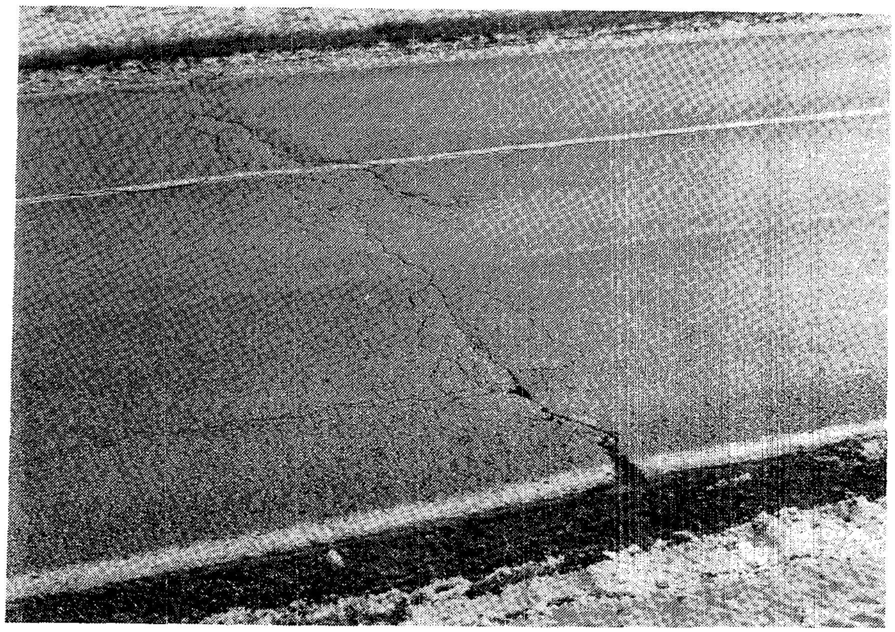


Figure 3.23. Illustration of transverse cracking of flexible pavement with cement treated base (CTB) with disintegration of the CTB and subsequent cracking of the asphalt concrete surfacing (FLEX-10, Illinois).

severe (> 3/4 inch) on only one project (FLEX-2-NJ) where some patching had been performed in the wheel path, and major (1/2 to 3/4 inch) in two projects. Rutting is traffic load associated, and occurs when any of the pavement layers permanently deforms under load. The extent of deformation of each layer on various projects included in the field study is unknown. However, data from the AASHO Road Test (where pavements consisted of an AC surface, crushed stone base, and gravel-sand subbase over a clay (A-6) subgrade), showed the following percentages of total rutting occurring in each layer and the subgrade:

Surface	32 percent
Base	14 percent
Subbase	45 percent
Subgrade	9 percent
Total	100 percent

Based on the results of the field surveys and review of the literature, it is concluded that rutting is caused by:

a. Consolidation by traffic of one or more layers of the pavement structures and/or the embankment material, and

b. Displacement outward from the center of the wheel path of the material in one or more layers of the pavement structures and/or the embankment material (57).

Most rutting in the Road Test pavements was attributed to changes of thickness of the pavement layer. Changes in thickness of the pavement layers are believed to be due primarily to lateral movement of the materials, but some due to the increased density. Five projects included in the field study were subjected to the identical traffic on I-80 in Illinois at the site of the AASHO Road Test. Rut depth data from these pavements are shown in Table 3.22. FLEX-7, with crushed stone base, has serious rutting while a pavement with a portland cement concrete base (COMP-5), which was included here for comparative purposes, has the least rutting. These projects are all 12 years old and have carried approximately 5 million ESAL applications. By implication, it is assumed that a considerable rutting in FLEX-7 is due to the crushed stone base and/or the gravel subbase. Comparing FLEX-8 and 9 with asphalt treated bases with COMP-5 and FLEX-10 (cemented bases), it can be theorized that some rutting has also occurred in the asphalt treated bases of these pavements.

3.4.3 Maintenance Activities

The types of maintenance performed on flexible high-type pavements were determined from the following sources:

1. Field survey of 19 pavements located in 9 states,
2. Discussions with maintenance personnel in most of these states, and
3. Previous research studies.

A summary of maintenance performed for various distress types on flexible pavements is shown in Table 3.23. The most common types of maintenance performed are crack filling (6 projects), skin and deep patching (6 projects), shoulder repair (approximately 6 projects), and grooving (1 project). Other types identified by maintenance personnel were overlay reprocessing and heater planer operations.

TABLE 3.22 RUT DEPTH DATA FROM FIVE PROJECTS LOCATED ON I-80, ILLINOIS SUBJECTED TO THE SAME TRAFFIC, ENVIRONMENT AND SUBGRADE

Project No.	Pavement Composition Surface -in.	Base -in.	Subbase -in.	Mean Rut Depth - in.
FLEX-7	4.5 AC	8.5 CS	23.0 G	0.66
FLEX-8	4.5 AC	8.0 ATB	4.0 G	0.51
FLEX-9	4.5 AC	10.0 ATB	4.0 G	0.40
FLEX-10	4.5 AC	12.0 CTB	4.0 G	0.38
COMP-5	3.0 AC	8.0 PCC	6.0 G	0.34

AC = asphalt concrete

CS = crushed stone

ATB = asphalt treated base

CTB = cement treated base

PCC = portland cement concrete

G = gravel

TABLE 3.23 SUMMARY OF MAINTENANCE PERFORMED FOR VARIOUS DISTRESS TYPES ON FLEXIBLE PAVEMENTS

Distress Type	Type Maintenance Performed
Alligator Cracking (fatigue)	Crack filling Overlay Skin patch Deep patch
Transverse and Longitudinal Cracking (including reflection)	Crack filling Patching w/ AC Overlay
Rutting	Patching with skin patch Overlay
Shoulder Distress	Seal coat Patching (skin and deep) Replace
Surface defect (polished aggregate, weathering of asphalt, raveling)	Overlay Grooving
Depressions	Skin patch level-up Overlay
Swells	Heater planning Skin patch level-up Overlay
Potholes	Patch w/ AC

3.5 COMPOSITE PAVEMENTS

3.5.1 General

Composite pavement is defined herein as asphalt concrete surface over a portland cement concrete base. There may or may not be one or more intermediate layers between the AC surface and the PCC base. Seven composite pavements located in four states, and in one Canadian province, were surveyed in the field study. Locations and statistical information on the pavement surveyed are shown in Table 3.24. The projects have high traffic volumes with two-directional ADT ranging from approximately 20,000 to 100,000. The total 18-kip ESAL applications in the heaviest traveled lane range up to nearly 28 million applications. The projects vary in age from 8 to 15 years, with an average of 12.5 years. Four of the seven projects provided zero-maintenance performance, while two required maintenance on the pavement proper, and one had maintenance on the shoulders. All of the projects were originally constructed as composite pavements, except COMP-1, which was an AC overlay over an existing PCC pavement.

3.5.2 Types and Causes of Distress

The major types of distress and their causes occurring on high traffic volume composite pavements are summarized in this section.

Distress Types: Many of the distress types associated with flexible pavements also occur on composite pavements. Of the 24 distress types identified in the literature for flexible pavements (13, 59, 110), the following were not found on any of the composite pavements surveyed: abrasion, char, corrugation, block cracking, alligator cracking, slippage, imprint indentation, shoving, streaking, significant waves, and serious upheaval.

Table 3.24 Summary of Location and Significant Properties of Composite Pavements Included in Field Survey

COMP Project No.	Location State	Age (Years)	Traffic (1) 18-kip ESAL x 10^6	PAVEMENT Surface	Base	Subbase 1	Subbase 2	Subbase 3	MAINTENANCE Performed	Type
1	MI	9	7.7	11 AC	3 AC	4"PCC 30'JT	-	-	Yes	Shoulder
2	CAN	15	12.4	3¼ AC	8PCC	6G	6G	-	Yes	Patching Crack Filling
3	CAN	15	12.4	3¼ AC	8PCC 100'JT	6G	6G	-	Yes	Patching Crack Filling
4	CAN	14	27.9	6 AC	8½PCC	6G	-	-	No	--
5	IL	12	4.8	3 AC	8PCC	6G	-	-	No	--
6	NJ	11	8.5	3½ AC	4STONE MAC	3G	8PCC 15'JT	6G	No	--
7	WA	8	1.2	4 AC	6PCC	2G	7G	-	No	--

(1) in heaviest traveled lane
AC = Asphalt Concrete
PCC = Portland Cement Concrete
G = Granular
Stone Mac = Stone Macadam

Distresses and their levels of severity observed in the field study for the 7 projects are summarized in Table 3.25. The general definitions of severity ratings for the major distress types are the same as for flexible pavements, and were presented earlier in Table 3.19 (Section 3.4.2). There is a minor difference between the severity rating on transverse cracking bewteen the composite and flexible pavements. The severity rating for transverse reflection cracking for composite pavements is based on crack spalling or disintegration, rather than crack spacing, since the former is more critical. Other distress severity ratings are based on the subjective evaluation of the project staff with respect to the effect on pavement structural integrity or functional surface condition.

Major Distress Types: The major distress types (rated "moderate" to "severe") occurring on high traffic volume composite pavements are summarized in Table 3.26. Transverse cracking is the most predominent distress, and it occurred on 6 out of 7 projects surveyed.

Some of these distresses are more critical than others in requiring maintenance. Table 3.26 shows the ratio of the number of pavements with "moderate" to "severe" distress to the number receiving maintenance by distress type. All composite pavement having "severe" transverse cracking (defined as severe spalling) received maintenance (2 of 7), whereas those projects with "moderate" to "major" transverse cracking had not received maintenance (4 of 7). Longitudinal cracking rated "moderate" to "major"

Table 3.25 SUMMARY OF SEVEN COMPOSITE PAVEMENT PROJECTS FIELD SURVEY FOR DISTRESS TYPE AND SEVERITY RATING

Distress Type	Distress Severity**					Total No. Surveyed
	0 None 1	Minor 2	Moderate 3	Major 4	Severe 5	
Alligator Cracking	7*					7
Edge Cracking	2	2	2	1		7
Longitudinal Cracking	3		3	1		7
Transverse Cracking	1		3	1	2	7
Slippage Cracking	7					7
Depression		5	2			7
Rutting		2	5			7
Upheaval	7					7
Corrugation	7					7
Potholes	4	3				7
Bleeding	7					7
Polished Aggregate	1	4	2			7
Raveling	3	3	1			7
Weathering	1	5	1			7
Shoulder Distress	4		2		1	7
Reflection Cracking	1		3	1	2	7
Disintegration Cracking	4	0	3			7

*Each number represents the number of projects in this distress type and severity rating

**As defined in Table 3.19

TABLE 3.26

Summary of Distress Types on Composite Pavements Where Distress is Rated as Moderate to Severe

Type of Distress	PROJECTS Distressed/Total	PROJECTS Maintained*/Distressed
1. Transverse Cracking	6/7	2/6
2. Rutting	5/7	0/5
3. Longitudinal Cracking	4/7	2/4
4. Edge Cracking	3/7	1/3
5. Random Cracking	3/7	1/3
6. Depression	2/7	0/2
7. Polished Aggregate	2/7	0/2
8. Raveling and Weathering	1/7	0/1

*Maintenance performed for only the distress indicated received maintenance in 2 out of 4 projects. Pavements having "moderate" rutting distress (1/4 to 1/2 inch) had not received maintenance as indicated in Table 3.27, and no projects had "major" or "severe" rutting (> 1/2 inch). Disintegration cracking in composite pavements is somewhat similar to the alligator or fatigue distress described for flexible pavements, but its causes are significantly different. Two pavements exhibiting this distress type to a "moderate" degree have received maintenance.

The observations indicated that transverse cracking is the predominant type of distress requiring maintenance. The two projects that received maintenance had "severe" transverse crack spalling. Other distress types, including longitudinal cracking, deterioration cracking, and edge cracking also received maintenance, but to a much lesser degree than the transverse cracking. Since there was no major or severe rutting (> 1/2 inch), no conclusion can be reached in this regard from the field survey. However, there is no reason that composite pavements will not show similar results as for flexible pavements where "major" to "severe" rutting usually received maintenance. This is primarily a safety problem, as severe rutting traps surface water which can cause hydroplaning.

Based upon data from the field survey which includes a limited number of projects, the major distress types that require or receive maintenance on high traffic volume composite pavements are as follows:

Transverse cracking: Occurs transverse to the longitudinal axis of the pavement at spacings from 50 to 100 ft. Cracking is usually rated

Table 3.27. Summary of Some Major Distresses Occurring on High Traffic Volume Composite Pavements

Project No.	State	Climate	18-kip ESAL x 10^6	AC Surface Thickness, inches	Transverse Crack Space and Severity, ft	Rutting, inches
1	MI	WF	7.7	11.0	90' Moderate	1/4
2	CAN	WF	12.4	3.3	85' Severe	<1/4
3	CAN	WF	12.4	3.3	80' Severe	<1/4
4	CAN	WF	27.9	6.0	70' Moderate	3/8
5	IL	WF	4.0	3.0	55' Major	3/8
6	NJ	WF	8.5	3.5*	None	3/8
7	WA	W	1.2	4.0	100' Moderate	3/8

* This project has 8 in. granular layer between the AC surface and PCC slab.

"major" or "severe", with evidence of deterioration to require maintenance. A transverse crack rated "moderate", and not requiring maintenance, is shown in Figure 3.24. A transverse crack rated "severe", and requiring maintenance, is shown in Figure 3.25.

Longitudinal cracking: Usually occurs at the lane construction joint for over more than 25 percent of the length of the project, and shows some evidence of deterioration to require maintenance.

Rutting: Occurs in the wheel paths and ruts greater than 1/2 inch in depth require maintenance.

Edge cracking: Occurs at the outer pavement edge in either the asphalt concrete surface and/or the PCC base. Must be "major" to "severe" to require maintenance.

Disintegration cracking: Occurs randomly over the pavement and mainly on pavements with thin asphalt concrete surface (i.e., 3 inch). Maintenance depends on the degree of severity.

Other Distresses: In addition to the five major distress types listed above, other distresses may occur in varying amounts depending on construction, foundation soil, materials, environment, etc. These distress types were previously discussed under flexible pavements, and include surface deterioration, swelling or heaving foundation soils, shoulder distress, depressions, and asphalt stripping. One other distress particularly related to composite pavements is disintegration of the PCC base from deicing salts in wet-freeze areas.

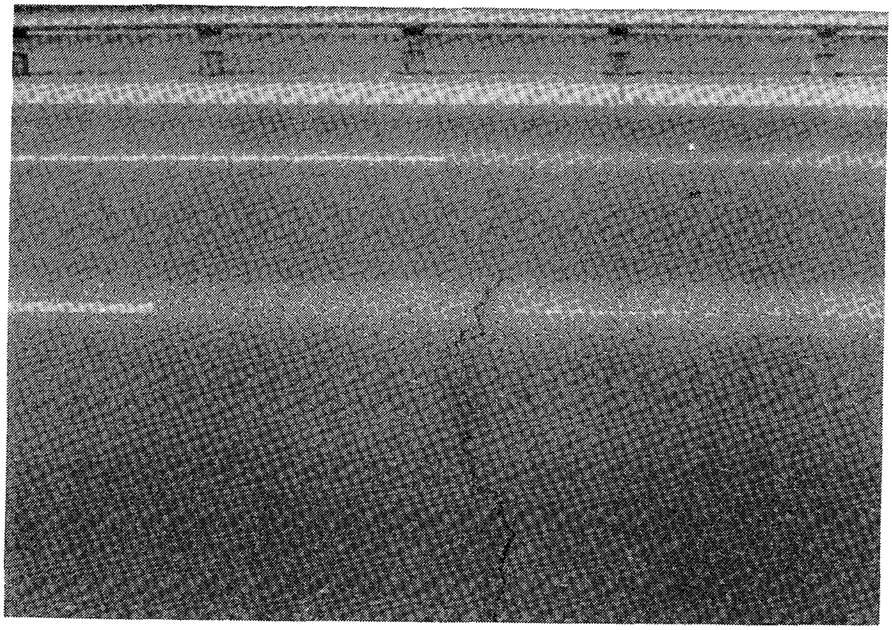


Figure 3.24 Illustration of a tight transverse reflection crack of a composite pavement (COMP-4, Toronto).

Figure 3.25 Illustration of a severely spalled transverse reflection crack of a composite pavement (COMP-3, Toronto).

Causes of Distress: The known causes are summarized for each major distress type identified in the field survey as follows:

1. Transverse Cracking: This distress occurs in composite pavements in both freeze and non-freeze regions. There are two basic causes: low temperature cracking of the asphalt concrete surface, as previously discussed, and reflective cracking emanating from the PCC slab. A summary of transverse crack spacing and severity for each composite pavement is shown in Table 3.27. All composite pavements, except COMP-6, have an AC surface directly over the PCC slab.

In transverse cracking, the crack spacing ranges from 50 to 100 ft, with an average of 80 ft. By comparison, plain PCC slab without joints initially cracks at approximately 40 to 150 feet spacing soon after its placement (101), and this initial cracking, which usually occurs before the AC surface is placed, eventually reflects through the AC surface. There was also some intermediate cracking on some of the pavement. These may be the result of secondary PCC shrinkage cracks or low temperature AC cracks. For example, COMP-7 is located near Seattle, Washington, in a non-freeze area. Flexible pavements in the area show none or very minor transverse cracking, and hence, the 100 ft crack spacing on this project can be attributed solely to the reflection cracking of the PCC slab. Similarly, COMP-5 in north central Illinois, has a crack spacing of 55 ft, and is situated adjacent to flexible pavements constructed at the same time with the same materials, and which have a transverse crack spacing of approximately 150 ft. Thus, the crack spacing in COMP-5 can therefore be attributed to reflection cracking of the PCC slab. COMP-1, 2, 3, and 4, which have relatively short crack spacing, are all located in severe freeze areas (Detroit and Toronto) where low temperature cracking occurs on almost all flexible pavements.

COMP-6, which has a different design than all the other composite pavements surveyed has not transverse cracking. This pavement is made up of eight inches of non-stabilized granular material between the AC surface and the PCC slab, which has a 15 ft joint spacing. This design has prevented reflective cracking entirely in the AC surface, even after 11 years of service and heavy traffic loading of 8.54 million 18-kip ESAL applications.

The basic mechanisms involved in reflective transverse cracking as discussed in the literature are as follows (67, 78):

a. Horizontal movement: The horizontal movement of the PCC slab due to temperature and/or moisture changes can cause excessive tension in the AC surface, eventually leading to fracture. This type of cracking can occur independent of traffic.

b. Load-temperature: During cold period, the cracks in the PCC have their greatest opening, and the load transfer across the crack is at a minimum. The vertical deflection at the crack, along with a relatively stiff AC surface material, produces high tensile stress in the AC surface, leading to fracture.

c. Differential vertical movement: Excessive vertical differential movement at a crack in the PCC slab caused by heavy loads can cause high shear stresses in the AC surface, resulting in a possible fracture.

The occurrence of transverse reflective cracks in composite pavements is not critical if the cracks remain "tight". Three composite pavements (COMP-1, 4, and 7) exhibited cracking, but did not receive any maintenance because the cracks remained tightly closed. However, two of the pavements (COMP-2 and 3) cracked and seriously spalled, resulting in maintenance. COMP-5 had major spalling, and is currently being overlayed.

COMP-1-MI, and particularly COMP-4-CAN, both have heavy traffic loadings, with AC surfaces of 11 and 6 inches, respectively. Despite the traffic, the transverse cracks in these pavements did not show serious spalling. Conversely, COMP-2, 3, in Canada, and 5 in Illinois, which also had heavy traffic loading, but with AC surfaces of 3.3, 3.3, and 3.0 inches, respectively, all showed serious crack deterioration. Thus, the AC surface thickness (for a given PCC slab) appears to be a very significant factor affecting crack spalling. A relatively thin AC surface under heavy traffic is not capable of withstanding heavy traffic applications expected on high traffic volume pavements.

2. Longitudinal Cracking: The cause of this distress is the same as that identified for flexible pavements with one additional causative factor. There is usually a weakened longitudinal joint in the PCC slab, placed for the purpose of construction, or to control slab cracking, which may result in reflection cracking of the AC surface. Longitudinal cracking occurred on 4 of the seven projects surveyed, to a "moderate" to "severe" degree, but was definitely not as serious as the transverse cracking with respect to requiring maintenance. In one project (COMP-7-WA), the longitudinal

crack occurred at the center of the traffic lane, which was also the edge of the paving lane, because of problems with construction equipment.

3. Rutting: The causes of rutting in composite pavements are the same as those identified for flexible pavements. Rutting for all composite pavements included in the study was rated only "minor" to "moderate" (i.e., < 1/2 inch) even under very heavy traffic such as on COMP-4-CAN. As shown earlier in Table 3.22, the PCC slab seems to have a definite effect on minimizing AC surface rutting.

4. Edge Cracking: The cracking of the PCC slab along its outer lane edge occurred only on one project, COMP-3-CAN, but represents a potential problem. Many of the composite pavements are designed with the edge of the PCC slab and the AC surface directly at the outer edge of the traffic lane. This practice can lead to edge cracking through loss of support due to pumping beneath the PCC slab, especially when excessive moisture and heavy loads are present.

5. Deterioration Cracking: This type of distress is somewhat similar to alligator cracking in flexible pavements. The AC surface tends to break into small rectangular or polygonal shapes or random cracks are developed. The mechanism involved, however, is probably not fatigue cracking as in flexible pavements because a stress/strain analysis of a COMP pavement shows the AC surface to be in compression throughout its entire depth. The two pavements (COMP-2 and 3 in Canada) that showed severe deterioration cracking also exhibited serious asphalt stripping, and had relatively thin AC surfaces. The stripping of the asphalt is believed by the engineers in

Ontario to be primarily responsible for the deterioration cracking. The large compressive and therefore, shear, stresses occurring in such a thin AC layer overlaying a very stiff PCC slab may also contribute.

3.5.3 Maintenance Activities

The maintenance performed on the two composite pavements included filling of transverse and longitudinal cracks, asphalt concrete patching of spalled areas, and patching the edge cracking. The general types of maintenance that may be performed on high traffic volume composite pavements will be similar to that summarized in Table 3.23 for flexible pavements. A summary of the maintenance performed on each composite pavement surveyed is given in Table 3.24. The results indicate that transverse cracking (which subsequently spalls) is by far the most serious distress in composite pavements which requires maintenance. Other potentially serious distresses include rutting, longitudinal cracking, edge cracking, and deterioration cracking.

3.6 SHOULDERS

The shoulder is considered as part of the pavement for purposes of zero-maintenance design, since structural distress of the shoulder that requires maintenance, usually requires the closing of the adjacent traffic lane, thereby restricting traffic and causing delays. Asphalt concrete surfaces were used on the shoulders of all pavements included in the field survey, excepting a few with gravel surfaces. PCC shoulders have also been used on several high traffic volume pavements in the U. S. Typical shoulder cross-sections for the pavements surveyed include 1-4 inches asphalt concrete over a granular base and subbase. The granular subbase is often assumed to be a "drainable" material, but field experience does not support this assumption.

This section briefly summarizes the major types and causes of distress on shoulders, and types of maintenance performed. Most of the distresses are related to asphalt concrete surfaces, since that is what was used on all sections included in the field study (excepting a few gravel shoulders). However, PCC shoulders have been constructed on several miles of high-volume pavements and some data are available on their long term performance (160, 161). A more detailed discussion of shoulder distress and its causes can be found in the final report for NCHRP study 14-3 on Improved Pavement Shoulder Design (180), and from a state of the art review of Portigo (181). Results determined in the field study of 68 pavements generally support the findings of Hicks and Barskdale (180).

Types and Causes of Distress

Several of the distresses that occur on flexible pavements also occur on asphalt concrete surfaced shoulders. The major types of distresses found in the field study for shoulders consisting of an asphalt concrete surface (1-4 inches) over a granular base and subbase include:

1. Alligator cracking (including edge cracking) near the pavement-shoulder joint.
2. Heave and settlement
3. Transverse and longitudinal cracking

The primary causes of these distress types are traffic loadings and environmental factors, with excessive moisture and frost heave being significant contributing factors. Shoulders receive some traffic when vehicles stop for rest or repairs. Also, a recent study in several states indicates considerable paved shoulder encroachment by transport trucks during normal operations (158). Sixty-five percent of all sample trucks were found to encroach on the outside shoulder sometime within a prescribed 10 mile length of rural freeway for both concrete and asphalt surfaced pavements. Considerable free moisture also collects in the base of the shoulder, particularly where a lane/shoulder joint exists (159). This moisture reduces the support of the granular base course and subgrade causing increased stresses in the AC surface under traffic loadings. A complete description of the causes of alligator cracking is given under flexible pavements in Section 3.4.2. Alligator cracking in shoulders normally occurs in a strip from 0 to 3 feet from the pavement edge which is the area of most of the truck encroachment.

If the outside pavement lane has been subjected to a large number of load application, a portion of which (i.e., 1-2 percent) would likely have encroached onto the shoulder. If the pavement is located in a wet-freeze region, the granular base probably contains excess free moisture during most of the year, especially if the pavement is not properly drained. This type of shoulder is clearly underdesigned structurally.

Another major cause of shoulder distress is frost heave and subsequent depression after the spring thaw period. Several typical examples of this distress occurred in the projects surveyed. This distress usually causes the lane joint to open and displace vertically creating a significant safety hazard. Heaving has also occurred in areas where swelling soils exist due to moisture entering the pavement shoulder joint and causing volume change in the subgrade soil.

Results from a survey by Hicks and Barksdale (180) show that use of full depth asphalt concrete shoulders greatly reduces the cracking and separation at the joint. Also, a shoulder study in Illinois indicated that a full depth asphalt stabilized aggregate shoulder performed better than either cement-aggregate or pozzolan-aggregate base shoulder (161). Field inspections in five states by Hicks and Barksdale showed that cement stabilized bases tend to pump resulting in faulting of the mainline slab, settlement, and subsequent cracking and deterioration of the shoulder in the vicinity of the transverse joint.

Portland cement concrete shoulders were first constructed in Illinois in 1965. Since then, several states have constructed PCC shoulders. Results from the Illinois studies (160, 161) show essentially maintenance-free performance over the 10 years since construction. The thickness of PCC shoulders varies from 6 inches to a thickness equal to the slab. A separation of the shoulders from the traffic lane occurs if tie bars are not used.

Based on results observed during the field surveys, discussions with practicing engineers and literature surveys, it is concluded that the conventional shoulder design of 1-4 inches of asphalt concrete over granular base is definitely inadequate fro zero-maintenance pavements. The performance of full depth asphalt concrete shoulders appears to be much improved over the conventional design. Performance of PCC shoulders over an 8-10 year period in Illinois has shown that maintenance-free performance on rural interstate highways is feasible.

Maintenance Performed

The maintenance that is performed on asphalt concrete surfaced shoulders includes patching, crack sealing, lane joint sealing, surface treatment, and resurfacing. On a few projects, underdrains were placed during the resurfacing operations to permit free drainage of moistures. More maintenance was required in wet freeze regions than in any other environmental regions probably due to the increased distress from temperature and moisture.

CHAPTER 4

ADEQUACY OF COMMONLY USED DESIGN METHODS TO OBTAIN MAINTENANCE-FREE PAVEMENTS

4.1 GENERAL

Highway pavement design procedures commonly used in the United States are based upon one or more of the following:

1. Theoretical considerations,
2. Results of road tests,
3. Laboratory studies, and
4. Engineering experience and judgment.

However, due to the extremely complex nature of the pavement/subgrade system and a variety of traffic and climatic "loads" imposed on the system, the development of most of the commonly used design procedures has relied more heavily upon engineering judgment and the empirical results of field tests than on the theoretical concepts. These design procedures have achieved varying degrees of success in providing "acceptable" pavements, but all of these methods are based upon the premise that routine maintenance is required to keep the pavement in service throughout its design life. Further, the definition of "design life" is nebulous. Most of the pavements designed using existing design procedures for a specified design life have required major resurfacing or rehabilitation during the specified design life, or have been kept serviceable for the duration of the design life through the use of heavy maintenance.

The purpose of this chapter is to make a critical and comprehensive evaluation of some of the commonly used design procedures to provide pavements that will not require structural maintenance over a design life ranging up to 40 years. The evaluations are based on determining whether or not the design procedures (and specifications) correctly consider all factors that may cause pavement distress which require maintenance. Results of this evaluation will be the basis for the development of zero-maintenance design procedures.

The evaluation of commonly used design procedures involves the following two approaches:

1. Conceptual Evaluation: This involves an analysis of the fundamental basis for development of the procedure, an assessment of theory and data, and a critical review of the assumptions used in development of the method. In addition, the probable limitations of the procedure to design against important distress types existing on high type pavements (as documented in Chapter 3) are also discussed.

2. Analytical Evaluation: This involves a comparison of the actual performance of the 68 projects officially included in the field survey with a "redesign" of each project using the specified design method. "Redesigning" is accomplished by using actual input values obtained during the field visits (including materials, thicknesses, soils, traffic, and other data) and a design life equal to the current age of the pavement. Total 18-kip equivalent single axle loads were computed as shown in Appendix A. The design procedure is then evaluated for each project as to whether it provides:

(a) Designs that have lasted x years in a maintenance-free condition (where x equals the current age of the project),

(b) Designs that do not provide maintenance-free performance over x years, and if not

(c) Identify the distress types which resulted in maintenance requirements.

Each design procedure is evaluated using the observed performance of pavements serving under a range of environmental conditions. This approach provides a meaningful evaluation which assists in verifying the conclusions reached in the conceptual evaluation in Item 1.

The two approaches described above provide a complete evaluation of the design procedure relative to its ability to provide maintenance-free pavements over a specified design life. This dual approach provides a comprehensive analysis of a design procedure by combining the theoretical evaluation with actual field performance evaluation ranging over many regions of the United States. It will provide a valuable basis for the development of a zero-maintenance design procedure during the subsequent phases of this study.

The design procedures evaluated in this study are listed in Table 4.1. The procedures evaluated for rigid pavements basically involve two categories, those based on performance criteria or developed from results of the AASHO Road Test which is the basis for the AASHO Interim Guide, and those based upon a fatigue structural analysis such as the Portland Cement Association (PCA) method. The two procedures selected for evaluation of CRCP design are based on the AASHO Road Test results. The procedures selected for evaluation of flexible and composite pavements are based on the AASHO Road Test results with the exception of the California DOT procedure.

Table 4.1 Summary of Commonly Used Design Procedures Evaluated for Adequacy

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I. Jointed Concrete Pavements (JCP and JRCP)

   A. AASHO Interim Guide (1)
   B. AASHO Procedure as applied by a state DOT (63)
   C. Portland Cement Association (104)
   D. Basic PCA Procedure as applied by a state DOT (21)

II. Continuously Reinforced Concrete Pavement

   A. AASHO Interim Guide (1)
   B. American Concrete Institute (5)

III. Flexible Pavements

   A. AASHO Interim Guide (1)
   B. Asphalt Institute (9)
   C. Resilience procedure as applied by a state DOT

IV. Composite Pavement

   A. AASHO Interim Guide (1)

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

Each of these procedures is evaluated with jointed concrete pavements first, and followed by continuously reinforced pavements, flexible pavements, and composite pavements.

4.2 JOINTED CONCRETE PAVEMENT

The major components of design of jointed concrete pavements are (1) structural thickness of the pavement layers, (2) joint design, and (3) reinforcement (if used). Each of these components of design are considered with the design methods under study.

4.2.1 AASHO Interim Guide

Introduction

The "AASHO Interim Guide for Design of Pavement Structures" (3) was developed in 1962 by the AASHO Design Committee through its subcommittee on Pavement Design Practices. The Guide was evaluated under NCHRP Project 1-11, and subsequently published in 1972 (1). The fundamental procedure in the 1972 Interim Guide was not changed from that originally prepared in 1962.

Conceptual Evaluation

The Rigid Pavement Guide is based upon 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 (120) which was developed from theoretical considerations of slab behavior, field observation, and laboratory investigations at Iowa Engineering Experiment Station. The nature of the design equation is basically empirical. A complete description of the development of the structural design model is given in Appendix D of the 1972 Interim Guide and in the AASHO Road Test report (57). The following information summarizes the basis for derivation of the Guide, and its conceptual evaluation:

1. 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).

$$\log_{10} W = \log\rho + G/B \tag{4.1}$$

where

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

$\log\rho = 5.85 + 7.35 \log (D + 1) + 4.62 \log (L1 + L2) + 3.28 \log (L2)$

$$B = 1.00 + \frac{3.63\ (L1 + L2)^{5.20}}{(D + 1)^{8.46} L2^{3.52}}$$

$$G = \log \left(\frac{P1 - P2}{P1 - 1.5}\right)$$

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

2. Using the Spangler equation, the empirical model given by Equation 4.1 was modified and extended to include material properties including PCC flexural strength (F), 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 basic AASHO Road Test Equation 4.1, and

b. Any change in the ratio tensile stress/strength resulting from changes in the values of E, k, and F 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 4.1.

The resulting final structural design model is given as follows:

$$\log W_{18} = 7.35 \log (D + 1) - 0.06 + \frac{G}{1 + \dfrac{1.624 \times 10^{7}}{(D + 1)^{8.46}}}$$

$$+ (4.22 - 0.32P2) \quad \log \quad \left(\frac{F}{215.63J}\right)\left(\frac{D^{0.75} - 1.133}{D^{0.75} - \dfrac{18.42}{Z^{0.25}}}\right) \qquad (4.2)$$

where W_{18} = number of 18 kip single axle loads to reduce the serviceability index from P1 to P2

F = flexural strength of the concrete slab (28-day cure, 3rd point loading), psi

D = PCC slab thickness, inches

J = load transfer coefficient

$Z = E/k$

E = PCC slab modulus of elasticity, psi

k = modulus of foundation support, pci

3. Specific Conditions - The general conditions under which the AASHO Road Test was conducted and the basic structural design equation developed from the results are as follows:

a. 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.

b. Length of Pavements - The length of the test section was 120 ft for the jointed concrete slabs and 240 ft for jointed reinforced concrete slabs. The slab lengths are discussed under Item f.

c. 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.

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

e. Climate - Climate in northern Illinois has about 30 inches annual precipitation and +4 inches more annual precipitation than evaporation. The average depth of frost penetration is about 30 inches, and the number of freeze-thaw cycles is 12 per year at the subbase level in the pavement.

f. 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 slabs with 15 ft joint spacing.

g. Length of Test - The test was conducted over a two year period; too short for effective evaluation of corrosion of dowels and deterioration of concrete.

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

4. Accuracy of Structural Design Model - The empirical thickness design expressed by Equation 4.1 was derived from results from the Road Test data, and relates specifically to the conditions listed above. Within these conditions, the ability of Equation 4.1 to predict the exact number of load applications to any given level of serviceability index for a pavement was fairly poor as shown in Figure 4.1. 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 4.1, for example, if the slab thickness were 8 inches, the resulting number of 30-kip single axle load applications to a terminal serviceability index of 2.0 would range between 400,000 to 1,910,000 for AASHO Road Test conditions. If Equation 4.1 is used for conditions different than those for which it was developed, its range of accuracy or associated error of prediction will undoubtedly be greater. The modified expression, Equation 4.2, allows for changes in k, E, and F, but the accuracy of these adjustments is unknown. Equation 4.2 is the basic design expression used in the AASHO Interim Guides where a safety factor is applied by reducing the flexural strength, F, by 25 percent, so that instead of using the actual modulus to replace value F, a value of 0.75F is recommended for design.

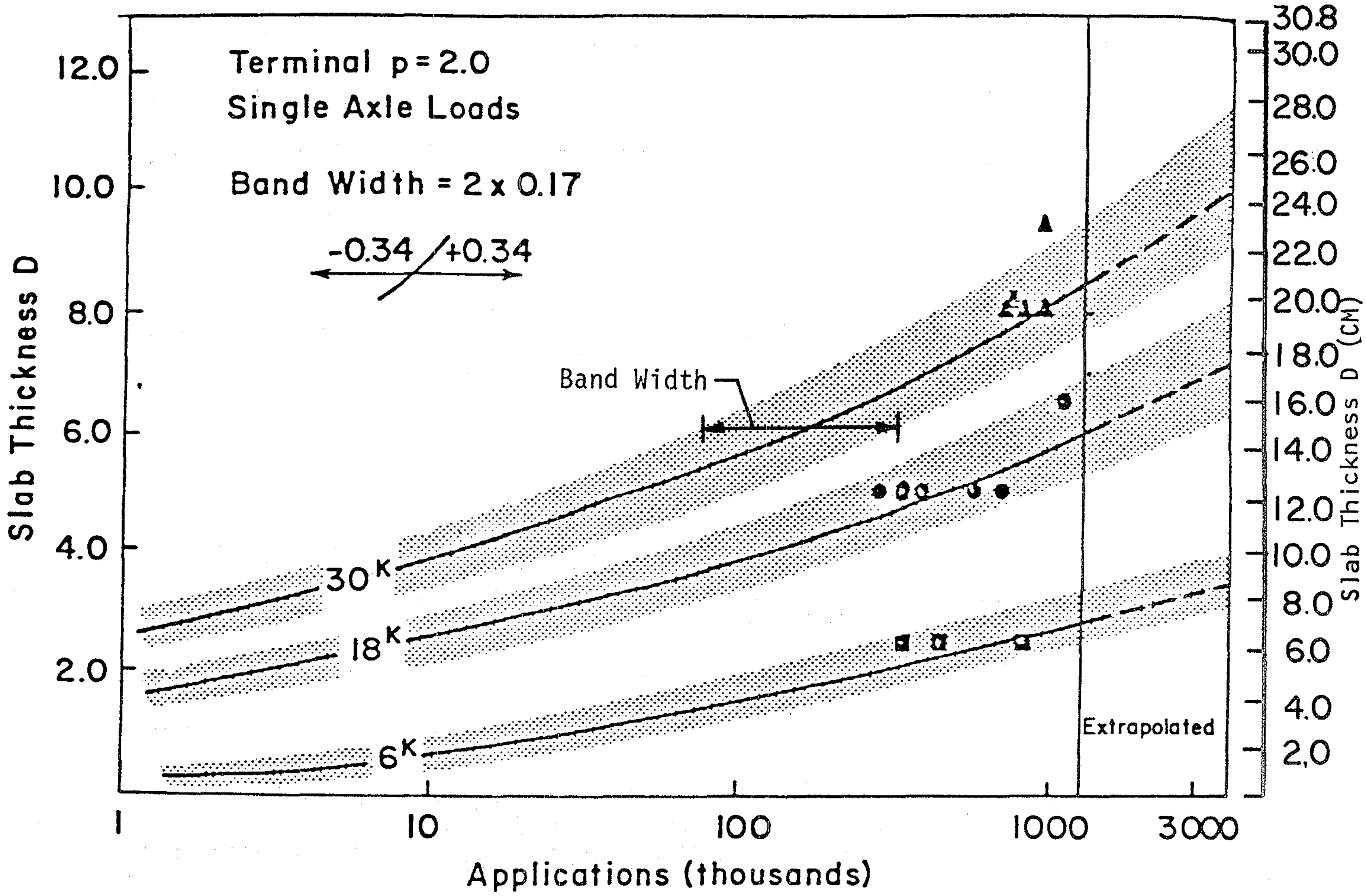


Figure 4.1. Illustration of Error of Prediction of Basic AASHO Design Model (from 56).

5. Joint Design - The AASHO Interim Guide recommends transverse contraction joints. Expansion type joints are recommended for use only adjacent to structures. No guidance is provided with regard to joint spacing, but only 15 foot plain and 40 foot reinforced slabs were evaluated in the Road Test. Recommendations on the depth of joint are approximately 1/4 of the thickness of the slab. Recommendations as to load transfer device include the diameter of dowels as a function of slab thickness. Normal dowel spacings is 12 inches.

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

$$A_s = \frac{fLW}{2f_s} \qquad (4.3)$$

where A_s = cross sectional area of steel per foot width of slab, square inches

f = average coefficient of resistance between slab and subgrade (1.5 recommended for design), but much higher values may be found with the stabilized subbase materials.

L = distance between free transverse joints or between free longitudinal ends, ft

w = weight of pavement slab, lb/sq ft

f_s = allowable working stress in the steel, psi

Based on these facts, some limitations of the design procedure to provide for a maintenance-free pavement are summarized as follows:

1. Variability - A serious limitation of the AASHO Design Procedure is that Equations 4.1 and 4.2 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 AASHO Interim Guide would therefore be expected to show significant localized failures before the <u>average</u> project serviceability index drops to P2. (This conclusion is valid despite the fact that a safety factor of 1.3 is used in design.) There are also variations in other design variables for which no factors of safety have been allowed. Routine maintenance is usually required well before the project reaches the average serviceability index of 2.5 used in design as is described in Chapter 5.

2. Loss of Foundation Support - Many of the Road Test sections showed significant pumping of the subbase. Therefore, the Equations 4.1 and 4.2 are biased towards this condition. If a particular pavement does not pump (due to the use of stabilized subbase, or as a result of being in a dry region) the pavement would last longer than predicted by Equation 4.2, provided all other factors were similar to those in the AASHO Road Test. No recommendations are provided for drainage to allow removal of moisture to prevent pumping.

3. Design Period - Design periods under consideration in this project range from 20 to 40 years. The 1,114,000 applications, upon which Equation 4.1 and 4.2 are based, represent only a fraction of the load applications that would be expected on a high volume pavement over the design

period (10 to 50 million kips 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 to cause severe deterioration of the pavements even without the load applications (i.e., corrosion of steel, joint freeze-up, D-cracking, etc.). Therefore, in severe climates, the pavements would be expected to endure fewer load applications (or fewer years) than predicted by Equation 4.2.

4. Joint Design - Only one type of joint design was used at the Road Test. If other types of joints are used, such as joints without dowels (as evidenced by the performance of the transverse cracks), or with some unusual type of load transfer devices, the pavement life would be significantly changed as documented in Chapter 3. Basic deficiencies in the joint design recommendations are little or no guidance for 1) joint spacing; 2) shape of joint and its relationship to joint sealer, 3) corrosive resistant dowels; 4) when mechanical LTDs are required; 5) joint sealer type; and 6) load transfer system other than dowels.

5. Reinforcement - 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 point 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 the crack.

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. 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.

7. Load Equivalency Factors - The load equivalency factors relate specifically to the Road Test materials, pavement compositions, climate, and subgrade soils. The accuracy of extrapolating them to other regions and materials, etc., is not known, but is questionable, especially if the pumping were eliminated.

8. Limiting Criteria - The design procedure is based on a terminal serviceability of 2.0 and 2.5. As shown in Chapter 5, there is a high probability jointed concrete pavement will require maintenance by the time it reaches an average serviceability index of 2.5. Thus, higher terminal serviceabilities should be considered. This cannot be done without a complete analysis of the design procedure.

In summary, the conceptual limitations discussed above show that the AASHO Interim Guide will not provide maintenance-free pavements, especially over all ranges of climatic conditions. The major theoretical deficiencies are: 1) the lack of consideration of variations in construction and materials; 2) design periods and traffic being much greater than the data on which the empirical design method is based; 3) questionable extrapolation of load equivalency factors; 4) joint designs different from those at the AASHO Road Test not considered and very little guidance provided on joint design; 5) variable climatic conditions not considered in the design procedure; and 6) reinforcement design to be inadequate if joint lockup occurs.

However, there are some factors which would lead to a longer life than predicted by the design equation. These factors are: the prevention of subbase pumping and the reduction (by 25 percent) of design flexural strength of the PCC slab. Since most of the Road Test pavements exhibited pumping, and the average flexural strength was used to derive the design equation, these factors, if present, will lead to longer pavement life than predicted by the basic design equation.

Analytical Evaluation

A comparison of the actual performance of each JCP and JRCP project included in the field survey was made with a "redesign" of each project to provide analytical data by which to evaluate the design procedures. "Redesigning" is performed with input values obtained during the field surveys and a design life equal to the current age of the pavement. An example "redesign" is provided in Appendix A. The evaluation for the design method is conducted with respect to its thickness, joints, and reinforcement design capabilities for JCP and JRCP.

Thickness Design. The existing foundation, including subbase conditions at each project site, is held constant, and the required slab thickness is determined using the AASHO Interim Guide (1). Detailed traffic data were obtained from each highway agency, and the total accumulated 18-kip equivalent single axle loads (ESAL) were computed over the existing pavement as described in Appendix A.

The design procedure is evaluated for each maintenance-free project as to whether it provides 1) a similar or more substantial pavement structure than that which has lasted x years in a maintenance-free condition (where x equals the current age of a project), or 2) a pavement structure less than the existing maintenance-free pavement. The redesign is also conducted for several projects that did not provide maintenance-free performance, to determine if it would indicate a more substantial structure is needed. The procedure is also evaluated by considering the specific distress type(s) which caused maintenance to be performed, and whether the particular distress(es) are considered in the design process. A summary of all the actual and redesign thicknesses for all JCP and JRCP pavements is given in Table 4.2. The significance and interpretation of the results are discussed in the following sections.

1. JCP Maintenance-Free Projects - Ten out of twelve non-reinforced jointed concrete pavements (JCP) exhibited maintenance-free performance. Maintenance-free performance is defined in this section to include all structural maintenance except joint sealing and shoulder repair. A plot of existing versus redesign slab thicknesses for these projects is given in Figure 4.2.

Table 4.2 Summary of "Redesigns" Using the AASHO Interim Guide for Jointed Concrete Pavements

Project Number	State	Existing in.	Existing (cm)	Zero Maint. Performance	Redesign in	Redesign (cm)	Redesign As % Of Existing	Does Redesign Give "0" Maint. Perf. Considering Thickness Only
JCP-1	IL	8.0	20.32	No	10.2	25.91	123	Yes
2	IL	9.5	24.13	Yes	10.9	27.69	115	Yes
3	IL	11.0	27.94	Yes	11.0	27.94	98	Yes
4	IL	12.5	31.75	Yes	11.1	28.19	87	Yes
5	WA	9.0	22.85	Yes	8.3	21.08	92	Q
6	CA	8.0	20.32	No	9.8	24.89	123	Yes
7	CA	8.0	20.32	Yes	11.5	29.21	148	Yes
8	TX	10.0	25.40	Yes	7.8	19.81	78	Q
9	UT	9.0	22.85	Yes	8.0	20.32	89	Q
10	AZ	9.0	22.85	Yes	11.5	29.21	128	Yes
11	NJ	10.0	25.40	Yes	11.1	28.19	111	Yes
12	GA	8.0	20.32	Yes	9.7	24.64	121	Yes
JRCP- 1	TX	9.0	22.85	Yes	8.6	21.84	96	Q
2	TX	9.0	22.85	Yes	7.3	18.54	81	No
3	TX	8-6-8	---	No	9.6	24.38	---	Q
4	TX	10.0	25.40	No	8.8	22.35	88	No
5	NJ	10.0	25.40	Yes	11.0	27.94	110	Yes
6	IL	8.0	20.32	No	10.2	25.91	123	No
7	IL	9.5	24.13	No	10.9	27.69	115	No
8	IL	11.0	27.94	No	11.0	27.94	98	No
9	IL	12.5	31.75	Yes	11.1	28.19	87	No
10	IL	10.0	25.40	Yes	9.7	24.64	95	Q
13	MI	10.0	25.40	No	9.0	22.85	90	No
14	MI	10.0	25.40	Yes	10.0	25.40	100	Yes
15	NY	9.0	22.85	No	9.0	22.85	100	No
16	NY	9.0	22.85	No	9.5	24.13	106	No
17	NY	9.0	22.85	Yes	8.5	21.59	94	Q
18	IL	10.0	25.40	No	10.3	26.16	103	No

Q = Questionable

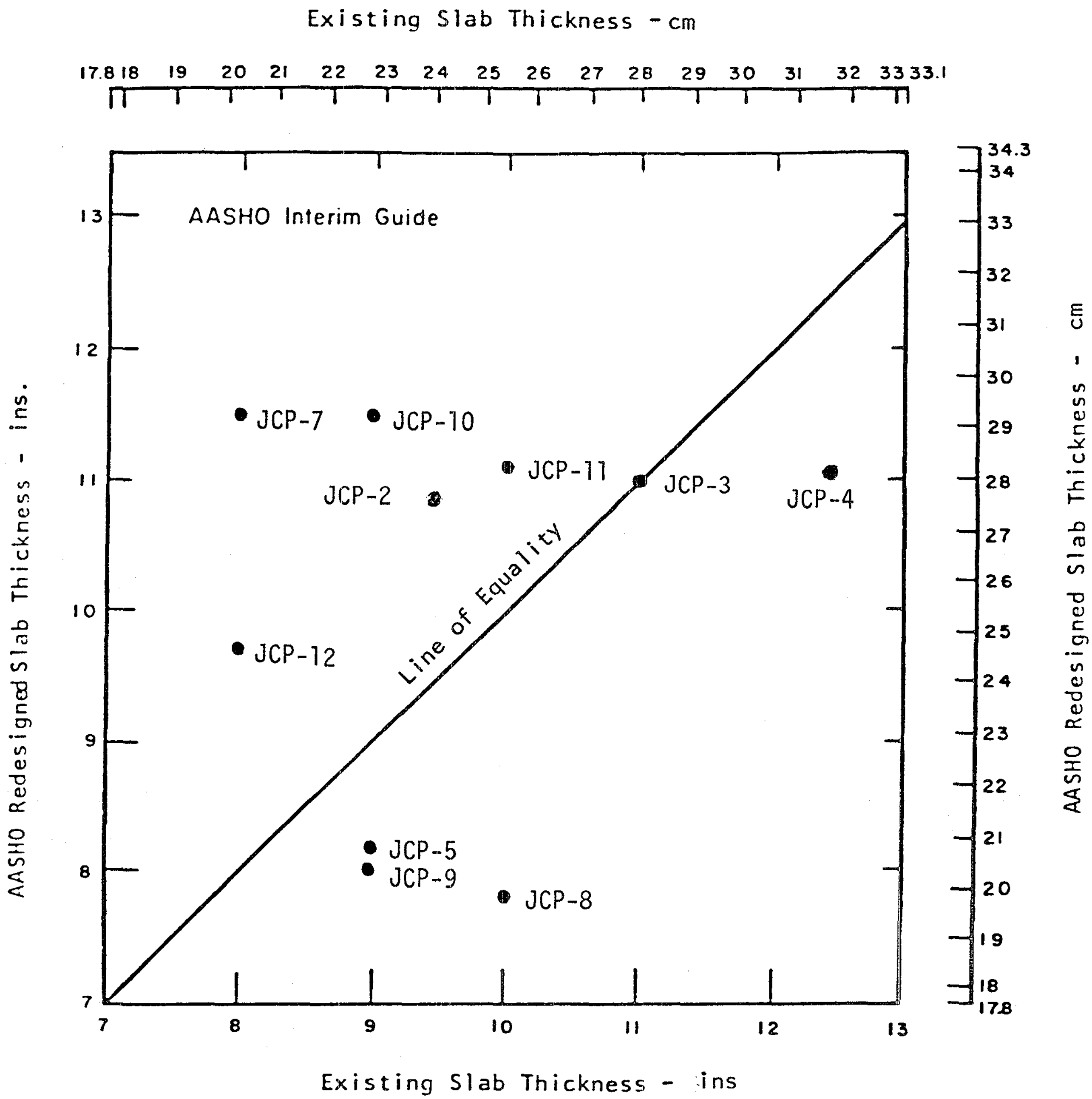


Figure 4.2. Jointed Concrete Pavement Redesign Slab Thickness Versus Existing Slab Thickness (all projects shown provided maintenance-free performance).

a. Seven of the ten maintenance-free pavements redesigned using the AASHO Interim Guide Procedure gave structures that provide maintenance-free performance. Of these, two projects (JCP-3 and 4) would likely provide maintenance-free performance even though the redesigned thicknesses were less than the actual pavement. These projects had 11 and 12.5 inch slabs and were part of the original AASHO Road Test (Loop 6) experiment. The existing conditions show that even a 9.5 inch slab provided maintenance-free performance for the current life of the pavement (17 years).

b. The remaining three project redesigns (JCP-5, 8, 9) were from 0.7 to 2.2 inches less than the actual slab thicknesses. These projects, however, had relatively low traffic loadings. It is doubtful whether these projects would have provided maintenance-free performance if thinner slabs had been used.

2. JCP Project Requiring Maintenance - The two projects (JCP-1, 6) that did not give maintenance-free performance had redesign thicknesses significantly greater (approximately 2 inches) than the actual pavement structures. It can be hypothesized that these pavements would have provided maintenance-free pavements had the redesign thicknesses been used, since the cracking distress which required maintenance on these projects seems to have been caused by traffic fatigue damage, a distress caused primarily by high stresses due to inadequate pavement thicknesses.

3. JRCP Maintenance-Free Projects - Of the 16 JRCP pavements surveyed, 7 provided maintenance-free performance. A plot of actual versus redesign slab thickness is given in Figure 4.3. The redesigns of these 7 JRCP sections indicate the following:

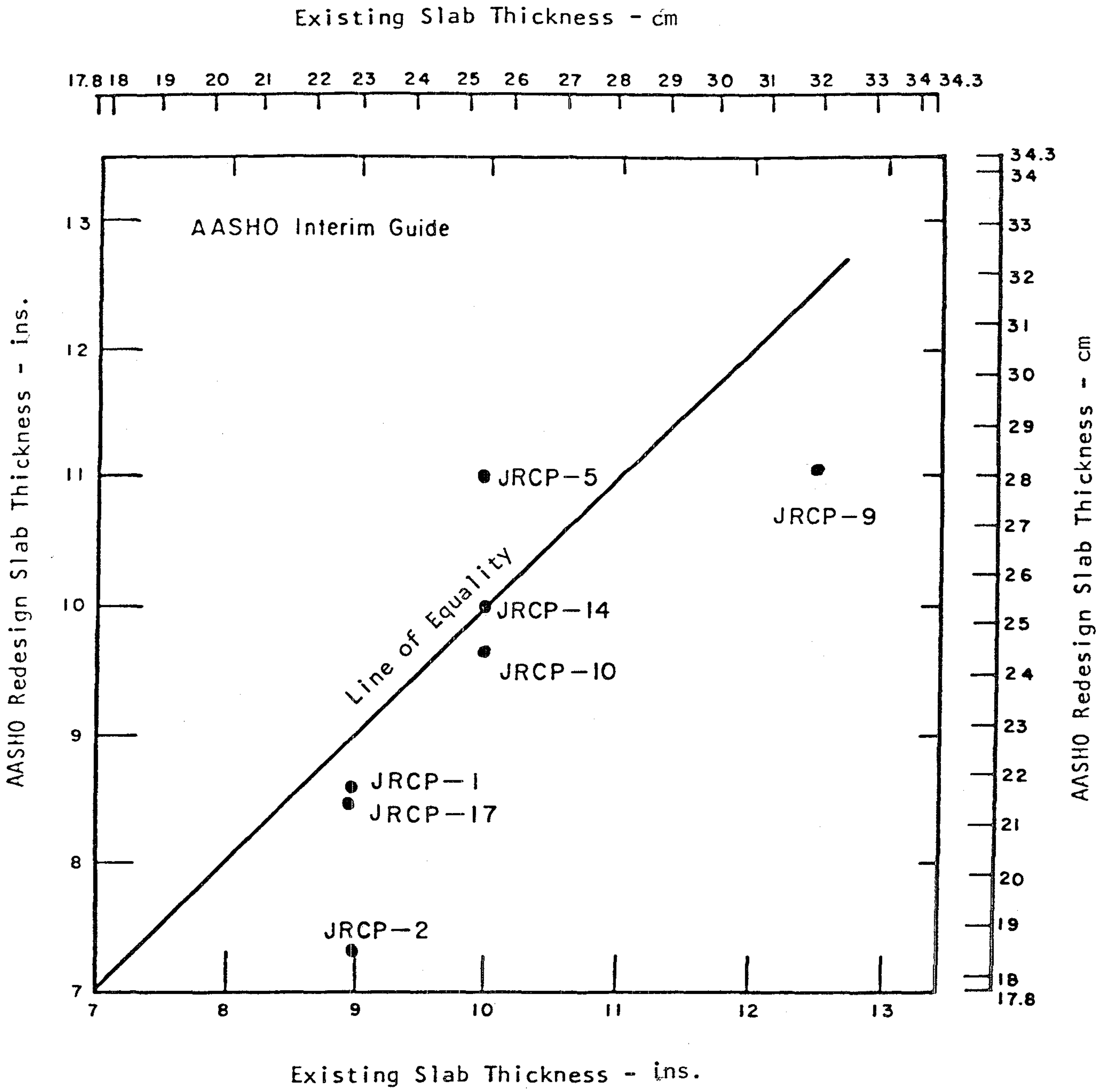


Figure 4.3. Jointed Reinforced Concrete Pavement Redesign Slab Thickness Versus Existing Slab Thickness (all projects shown provided maintenance-free performance).

a. Only two of the redesigns gave slab thicknesses greater than or equal to the existing maintenance-free structure.

b. The remaining five redesigns indicated thicknesses which ranged between 1/2 and 2 inches less than the actual thicknesses. It is noted that at least 2 of these redesigns would give thicknesses which would probably not be adequate to provide a maintenance-free performance. The redesigns of the other three may provide maintenance-free pavements. JRCP-9 has a 12.5" slab, redesigned by the AASHO Interim Guides as a 11.1" pavement. This design would not provide a maintenance-free performance because a pavement subjected to the same loading conditions with an 11" slab, JRCP-8, required maintenance due to serious cracking caused in part from traffic loadings.

4. JRCP Projects Requiring Maintenance - Of the 9 projects which did not provide maintenance-free performance, 6 of the redesigns provided thicknesses greater than or equal to the actual pavement thicknesses. It is noted that the redesigns for 2 of these projects, JRCP-6 and 7, would probably still not provide a zero-maintenance pavement even with the added thickness. This is concluded from the failure of JRCP-8, a 11-inch thick slab which failed to provide a maintenance-free pavement over 17 years. For the remaining 4 projects, it is doubtful that the indicated increase in thickness would provide maintenance-free performance, since most of the distress which occurred on these projects was caused by joint and reinforcement failures or swelling subgrade.

Joint Design. Several aspects of joint design which should be evaluated are: 1) joint spacing; 2) joint type; 3) joint shape; 4) sealant type; and 5) type of load transfer device. The AASHO Interim Guide, however, gives design recommendations for only a few of these critical parameters, and hence, is severely inadequate in this respect. The only recommendations that can be evaluated are type of joint, size of load transfer device, and depth of contraction joint.

1. Type of Joint - The AASHO Guide states that contraction joints provide adequate joints, and that expansion type joints are unneccessary except at structures. Three of the 18 JRCP projects surveyed had expansion joints exclusively, with doweled LTDs (JRCP-1, 5, 17), and none of these pavements showed joint distress. Three projects had a combination of expansion and contraction joints (JRCP-2, 3, and 14), and none showed significant joint deterioration. Seven of the 10 projects with contraction type joints had joint spacings ranging from 30 to 56 ft, and none of these projects had visible joint deterioration (several of these projects did have joint lockup, however, which is discussed later). The three projects with contraction joint spacings of 99 to 100 ft all showed serious joint deterioration (blowups and spalling). Of the 12 JCP projects with short joint spacing of 12 to 18 ft (and one with 30 ft), only two showed serious faulting, and none exhibited spalling or blowups. Thus, it is concluded that joint type interacts strongly with joint spacing to give a wide range of joint performance for which no specific recommendations have been provided by the AASHO Interim Guide.

2. Load Transfer Device - Specific recommendations are given for dowel bar sizes. A summary of the sizes of dowels in existing projects and AASHO recommended sizes is given in Table 4.3 for JRCPs. Of the 14 projects that used dowel bars as load tranfer devices, 5 projects contained dowels with a smaller diameter or larger spacing than the AASHO recommendations. The difference amounts to approximately 1/4 inch in diameter smaller for four projects, and a dowel spacing two inches larger for 1 project. Joint faulting was not significantly different on the projects with the lighter LTDs than for the other projects surveyed. Two projects had thicknesses greater than 10 inches for which the AASHO Interim Guide does not provide recommendations other than a dowel diameter as a function of slab thickness. The remaining 3 projects showed some joint deterioration, but did not exhibit serious faulting. Hence, it appears that the AASHO recommendations for dowel dimensions and spacing provide adequate load transfer to prevent faulting as long as joint spacing is adequate. Three of the projects contained proprietary LTDs other than dowels, and serious joint deterioration occurred on two of these projects (JRCP-15, 16). The proprietary LTD device used in JRCP-1 provided good joint performance, but this project had only 30 ft joint spacing, and is located in a wet/ non-freeze-thaw climate.

3. Depth of Contraction Joint - AASHO recommends that the depth of joint be 25 percent of the slab thickness. The 25 projects containing contraction joints showed a range of joint depth from 20 to 28 percent of the slab thickness. No particular distress could be attributed to depth of joint on any of these projects. So AASHO recommendations for depths of contraction joints appear to be adequate.

Table 4.3. Summary of Dimensions of Dowel Bars and AASHO Recommendations for JRCP Projects

Project No.	State	Slab Thickness	Existing Dowels Dia. (in.)	Existing Dowels Length (in.)	Existing Dowels Space (in.)	AASHO Design Dia. (in.)	AASHO Design Length (in.)	AASHO Design Space (in.)	Do Dowels Meet AASHO Requirements?
2	TX	9	1-1/4	20	9-12 edge 14 interior	1-1/4	18	12	Yes
3	TX	8-6-8	3/4	24	12	1	18	12	No
4	TX	10	1	20	12	1-1/4	18	12	No
5	NJ	10	1-1/4	16-3/4	12	1-1/4	18	12	No
6	IL	8	1	18	12	1	18	12	Yes
7	IL	9.5	1-1/4	18	12	1-1/4	18	12	Yes
8	IL	11	1-3/8	18	12	No Recommendations			---
9	IL	12.5	1-5/8	18	12	No Recommendations			---
10	IL	10	1	18	12	1-1/4	18	12	No
11	CAN	9	1-1/4	18	12	1-1/4	18	12	Yes
12	CAN	9	1-1/4	18	12	1-1/4	18	12	Yes
13	MI	10	1-1/4	18	12	1-1/4	18	12	Yes
14	MI	10	1-1/4	18	12	1-1/4	18	12	Yes
17	MI	9	1-1/4	18	12	1-1/4	18	12	Yes
18	IL	10	1	18	13.5	1-1/4	18	12	No

Reinforcement Design. Either deformed reinforcing bars or smooth welded wirefabric was used in all JRCP projects. A summary of the steel provided and the amount required as per AASHO recommendations (Eq. 4.3) is given in Table 4.4. The lockup of joints was suspected on several projects where considerable deicing salts were used during the winter. The lockup of a joint results in a significant increase of stress in the reinforcement (i.e., the pavement starts behaving similar to a CRCP)(176).

The following summarizes some results of the adequacy of the AASHO procedure for reinforcement design in JRCP pavements;

1. Nine out of 18 JRCP projects had an amount of reinforcement less than was provided for by the AASHO procedure. Four of these nine projects exhibited rupture of the reinforcement. Of the other 9 project, all of which had amounts of reinforcement larger than indicated by Equation 4.3, three had ruptured reinforcement.

2. All 7 projects that showed ruptured reinforcement were located in a wet/freeze-thaw climate where deicing salts had been used to an extensive degree as described in Chapter 3.

3. Ten of the 18 JRCPs surveyed appeared to have significant joint lockup. All 7 projects that exhibited ruptured reinforcement were in this category.

4. The rupture of the reinforcement does not appear to be related to joint spacing alone, since rupture of the reinforcement occurred with joint spacings of 40, 99, and 100 ft, while joint spacing of pavements not exhibiting ruptured reinforcing steel had joint spacings of 15, 20, 30, 40, 56, 60, 78, and 100 ft.

Table 4.4 Summary of JRCP Reinforcement and AASHO Redesign Reinforcement

Project No.-State		Actual Reinf. Type	Existing Reinf. Area (in^2/ft)	AASHO Design Area (in^2/ft)	AASHO Reinforcement As % Of Existing Reinforcement	Climate Region	Reinforcing Steel Rupture	Joint Lock-up Suspected
1	TX	DB	.131	.070	53	W	No	No
2	TX	DB	.025	.034	136	W	No	No
3	TX	DB	.393	.030	8	D	No	No
4	TX	WWF	.108	.062	57	D	No	No
5	NJ	DB	.176	.198	113	WF	No	No
6	IL	WWF	.093	.067	72	WF	Yes	Yes
7	IL	WWF	.108	.079	73	WF	Yes	Yes
8	IL	WWF	.126	.092	73	WF	Yes	Yes
9	IL	WWF	.148	.104	70	WF	No	Yes
10	IL	WWF	.172	.083	48	WF	No	Yes
11	CAN	WWF	.172	.105	61	WF	No	No
12	CAN	WWF	.172	.186	108	WF	Yes	Yes
13	MI	WWF	.172	.208	121	WF	Yes	Yes
14	MI	WWF	.172	.208	121	WF	No	No
15	NY	DB	.110	.228	207	WF	Yes	Yes
16	NY	DB	.110	.137	125	WF	No	Yes
17	NY	DB	.110	.228	207	WF	No	No
18	NY	WWF	.172	.208	121	WF	Yes	Yes

DB = deformed reinforcing bars
WWF = welded wire fabric
W = wet climate, no freeze-thaw
D = dry climate, no freeze-thaw
WF = wet climate, freeze-thaw

5. None of the 7 projects that exhibited maintenance-free performance showed ruptured reinforcement.

6. Significantly increased slab thickness and the corresponding increase in reinforcement apparently prevented rupture of the reinforcement and faulting of the resulting cracks. JRCP-9, having a slab thickness of 12.5 inches, did not rupture, whereas JRCP-6, 7, and 8 with 8, 9.5, and 11 inches of slab thickness, respectively, did have ruptured reinforcement. Each of these projects had more reinforcement than computed using Equation 4.3, but joint lockup was also apparent for these pavements.

7. The reinforcement recommendations provided by the AASHO for jointed reinforced concrete pavements provided adequate reinforcement for projects with granular subbases and where no joint lockup occurred. Projects where joint lockup was apparent generally exhibited ruptured reinforcement followed by spalling and faulting of the crack and significant pavement roughness, which often required maintenance.

Summary of Structural Design

Several deficiencies were identified in the AASHO Interim Guide with respect to its ability to provide maintenance-free mointed concrete pavements for long design periods. These conclusions are supported in general by the analytical evaluation. The ability of the AASHO Interim Guide to provide an adequate structure for JCP was rated as fairly adequate (8 out of 12 redesigned projects), but inadequate for the thickness design of JRCP (2 out of 16 redesigned projects). The major JCP distress requiring maintenance was traffic load associated cracking. The major JRCP distress was due to a combination of climatic effects (mainly joint lockup and warping stresses) and traffic loads. Hence, the Interim Guide <u>may</u> provide adequate structural sections for JCPs whereas it <u>does not</u> provide adequate structural sections for JRCPs since climatic effects are generally not considered in the Interim Guide structural design model.

<u>Joint Design Summary</u>

Recommendations are provided for only joint type, doweled load transfer, and depth of contraction joint. Recommendations for joint type are inadequate, whereas recommendations for dowel dimensions, dowel spacing, and depth of contraction joint appear to be adequate. Recommendations for joint spacing, joint shape and sealant material, corrosion resistant load transfer devices, and mechanical LTDS are non-existent.

<u>Reinforcement Design Summary</u>

The AASHO Interim Guide reinforcement design equation appears to provide adequate steel in non-freeze/thaw regions or in areas where joint lockup does not occur (where little deicing salt is used). Both theory and field results show that the recommended area of steel (Equation 4.3) is inadequate if joint lockup occurs. Therefore, if Equation 4.3 is used to compute the required reinforcement, provision must be made to prevent corrosion of the joint LTD and the concomitant lockup of the joint. Furthermore, theoretical studies (68) show that a fixed value of 1.5 recommended for the average coefficient of resistance between the slab and subbase may not be valid especially when a subbase with high slip resistance is used (such as a stabilized subbase). Hence, more detailed recommendations are needed for the effective coefficient of resistance for stabilized subbases. Significantly increasing slab thicknes (i.e., JRCP-9) may also reduce rupture of the reinforcement and subsequent crack faulting.

4.2.2 AASHO Interim Guide Applied by States

A number of states use a design procedure essentially the same as described in the AASHO Interim Guide. Many of the states have introduced minor variations to these procedures, but these modifications are usually intended to simplify the input data rather than to change the design approach and final result.

One state (63) has developed a time-traffic exposure factor to be applied with their version of the AASHO recommended procedure. The time-traffic exposure factor is defined as

$$TF = \frac{D}{D_2} \tag{4.5}$$

in which TF = time-traffic exposure factor;

D = Illinois slab thickness in inches; and

D_2 = Road Test slab thickness in inches.

The appropriate value for the traffic factor was established by comparing the performance of pavements in service with results from the AASHO Road Test. A mean value for the TF was found to be 1.34 (25). A factor of 1.3 has been adopted in developing the design nomograph, and this factor is incorporated in the slab thickness determination.

The major theoretical deficiencies in the AASHO design approach, when applied by the individual states, are essentially the same as those given for the AASHO Interim Guide with the exception of time-traffic exposure factor added to increase the slab thickness by 1.3. This is somewhat compensated for by dropping the factor of safety of .75 applied to the flexural strength as recommended in the AASHO Interim Guides. Another exception is that an overall averaging technique is used to obtain the total 18-kip ESAL applications for design. This technique in general produces traffic equivalencies that are smaller than those determined by a rigorous application of the AASHO Interim Guide procedure. For example, the total 18-kip ESAL applications given for project JRCP-10 are 5.85 for the state method compared to 7.73×10^6 for the AASHO Interim Guide method. In both design procedures, the traffic parameter has only a small influence on pavement thickness.

In summary, the state procedures are not expected to provide more reliable structural design than the AASHO Interim Guide, and will not provide maintenance-free pavements under a wide variety of design situations.

4.2.3 Portland Cement Association Design Procedure

Introduction

The Portland Cement Association has developed design procedures for concrete thickness design (104), subgrades and subbase (103), joint design (102), and distributed steel (100). The procedures developed are based upon theoretical studies, field tests, laboratory tests, and observations of performance of pavements in service. Several state agencies use some version of the PCA method for jointed concrete pavement design (21).

Conceptual Evaluation

The fundamental basis of the PCA procedure for structural design of concrete pavements is based on a linear fatigue damage concept. Stresses in the pavement due to traffic loadings are calculated (46) using the influence charts of Pickett and Ray (98) which were developed from Westergaard's model for maximum edge load flexural stress. A ratio of induced concrete flexural stress to the estimated flexural strength of the concrete is calculated to estimate the fatigue consumption. The modulus of subgrade reaction, k, for the slab support is calculated at the top of the subbase layer, and is a function of subbase thickness, the subbase material, and the "k" of the subgrade.

The linear fatigue damage model used in the PCA procedure is known as Miner's hypothesis (87). According to this hypothesis, fatigue failure occurs whenever

$$\sum_{i=1}^{m} \frac{n_i}{N_i} \geq 1 \cdot \qquad (4.6)$$

where n_i = number of stress repetitions at the i^{th} stress level

N_i = total number of stress repetitions at the i^{th} stress level which will product fatigue failure

m = number of load or stress increments used in this analysis

For this analysis, N_i is determined from the fatigue characteristics of the concrete. The fatigue characteristics are usually given in the form of an S-N curve, in which the ratio of the induced stress to the concrete strength is a linear function of the log of the number of load repetitions to failure. Thus, for a given ratio of induced stress to concrete strength, the number of load repetitions to failure can be determined. A table giving the recommended number of load repetitions vs stress/strength ratio is provided in the PCA design manual (104).

The essential features of the PCA procedure can be summarized as follows:

(1) Number of loads expected during the design life is based on current traffic data plus projections of future traffic growth, either by the use of yearly rates of growth or by use of lane capacity for design considerations.

(2) Traffic loads thus obtained are increased by an appropriate "load safety factor", usually around 1.2.

(3) k-value at the top of subbase is estimated from information on the thickness and type of the subbase and the properties of the subgrade soil.

(4) For each load category, the stresses are computed from charts developed for both single and tandem axle load configurations.

(5) Concrete flexural strength at 28 days is determined from beam tests loaded at third-points. The 90 day concrete strengths used for design are assumed to be 110 percent of the 28 day strengths.

(6) For each load category, stress/strength ratio is calculated and used to estimate the maximum number of allowable stress repetitions at that stress level based on fatigue of concrete.

(7) For each load category, the ratio of actual load repetitions to the maximum allowable load repetitions gives the "fatigue consumption" for that load category.

(8) Pavement thickness is selected so that the "total fatigue consumption", i.e., damage, by all load categories is approximately equal to 1.0, but not exceeding 1.25.

The analysis of possible limitations of the design procedure to provide zero-maintenance pavements can be summarized as follows:

(1) Loading - The PCA design procedure is based exclusively on the structural analysis of a pavement slab in which only those stresses produced by traffic loads are considered. The PCA procedure does not take into account the increase in load stresses due to curl in the slab due to temperature and moisture gradients through the slab.

(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 non-uniform support resulting from non-uniform subgrade and subbase materials, erosion of support due to pumping, and partial loss of support due to slab curl, and similar factors are not considered in the analysis of stresses for calculation of the fatigue damage in the PCA procedure.

(4) Variability - No provision is made in the design procedure for non-homogenity and variability in engineering properties of the concrete, subbase and subgrade materials, except through the factor of safety applied to the concrete fatigue curve, hence, some localized failures are expected.

(5) 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 performance and fatigue of pavement slabs.

(6) Stress Analysis - There is a lack of consideration of the many variables which affect pavement stress. Of particular importance in this respect is the curling in the slabs, and the effect of curling on the load induced stresses. It was shown, for example, at the AASHO Road Test that edge strain is one of the most reliable predictors of performance of concrete pavements. It was also shown that when the slabs are warped up along the edges, the strains (or stresses) induced by the applied loads are significantly greater than when the slabs are flat or warped down along the edges. Thus, any analysis based on the assumed flat slab condition with uniform subgrade support will inevitably result in an inaccurate stress state.

(7) Distress Mode - A significant factor to consider in evaluating the PCA design procedure is that the mode of distress observed in concrete pavements is not consistent with critical stress criteria used in the design procedure. The PCA design is based on a fatigue failure mode near a transverse joint. Failure from this loading would supposedly result in a longitudinal crack initiating at the transverse joint in both wheel paths with equal frequency. A study of the distress patterns from the AASHO Road Test shows that only the thinnest of the concrete pavements failed in this mode. With the thicker pavements (6 inches of greater), the major initial distress modes were transverse cracks well away from the transverse

joints. Also, in the field trips for this study, many examples of transverse crack distress were observed well away from the transverse joints, but only very isolated cases of longitudinal cracking were observed. Thus, the PCA procedure designs for a failure mode which occurs only rarely.

(8) Limiting Criteria - The limiting criteria applied in the PCA procedure relates only to the initial crack criteria, and is not necessarily related to a pavement performance failure criterion. PCA procedure recommends designing for a maximum of 125 percent fatigue consumption. Assuming that the PCA stress analysis provides correct estimates of stress, and that the Miner's damage law is valid (neither of which is probably true), this criteria (125 percent fatigue consumption) would indicate over 50 percent possible fracture of the PCC slabs along a typical project before 125 percent fatigue consumption occurred. This would occur because of the large material variations and fatigue life of pavements. While this amount of fracture may be tolerable on normal highways where maintenance can be performed, it would not be acceptable for a required zero-maintenance pavement.

(9) Joint Design - PCA provides several joint design recommendations in the revised 1975 procedure. 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 or less for JRCP and up to 20 ft for JCP. Specific dimensions for the joint sealant reservoir for various joint spacings are given. Dimen-

sions 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 JCP to prevent faulting. LTDs are recommended for all JRCP projects. These recommendations appear well founded theoretically. Recommendations are not provided for joint sealant by types, although they are discussed, and use of expansion joint systems is not recommended except under unusual conditions of construction, maintenance, and material usage.

(10) Reinforcement Design - Reinforcement design recommendations are similar to those provided by AASHO, hence, similar limitations would exist.

(11) Safety Factors - Factors of safety against premature failure are brought into the design processes in two ways. First, the typical PCC fatigue characteristic curve has been shifted so that the apparent number of loads to failure for a given stress/strength ratio is lower than the actual concrete fatigue data would indicate (69). Second, a factor of safety or impact factor of 1.2 is used to increase the magnitudes of the applied loads for the calculation of induced stresses for major highways with high volume of truck traffic.

In summary, based upon many conceptual limitations discussed above, the PCA procedure is not expected to provide maintenance-free pavements in all locations in the U. S. The applied safety factors are not large enough to allow for these limitations.

Analytical Evaluation

Thickness Design. A summary of all JCP and JRCP existing and redesigned slab thickness is given in Table 4.5.

1. JCP Maintenance-Free Projects - There are 10 out of 12 non-reinforced JCPs that exhibited maintenance-free performance. A plot of existing versus redesigned slab thicknesses is given in Figure 4.4.

a. Four of the 10 maintenance-free pavements redesigned by the PCA method give structures that provide maintenance-free performance. The redesign for JCP-4 of 11" was less than the existing slab thickness of 12.5". It is noted from the field survey, however, that a slab thickness of 9.5" is adequate to provide zero-maintenance performance under these conditions, and therefore, this design is considered to provide zero-maintenance performance.

b. Six of the 10 zero-maintenance JCP projects showed thinner structural sections than actually existed at the site. The PCA design provided slabs of 1 to 3 inches less than the existing thickness, and it is doubtful whether any of these redesigns would give zero-maintenance performance.

2. JCP Projects Requiring Maintenance - One of the projects (JCP-1) which did not give maintenance-free performance was given a redesign of 1/2" thicker than the slab in existence, which would probably not provide a zero-maintenance pavement. The remaining project, JCP-6, was given a redesign with slab thickness less than the existing thickness, which already had distress and has received maintenance.

Table 4.5 Summary of Redesigns Using the PCA Procedure for Jointed Concrete Pavements

PROJECT No.-State		Existing Slab (in.)	Existing Slab (cm)	Zero Maintenance	Redesign (in.)	Redesign (cm)	Does PCA Give "0" Maint. Performance?	Redesign As % of Existing
1	IL	8.0	20.32	No	8.5	21.59	No	106
2	IL	9.5	24.13	Yes	11.0	27.94	Yes	116
3	IL	11.0	27.94	Yes	11.0	27.94	Yes	100
4	IL	12.5	31.75	Yes	11.0	27.94	Yes	88
5	WA	9.0	22.85	Yes	6.5	16.51	No	72
6	CA	8.0	20.32	No	7.0	17.78	No	88
7	CA	8.0	20.32	Yes	7.0	17.78	No	88
8	TX	10.0	25.40	Yes	7.0	17.78	No	70
9	UT	9.0	22.85	Yes	6.5	16.51	No	72
10	AZ	9.0	22.85	Yes	10.5	26.67	Yes	117
11	NJ	10.0	25.40	Yes	8.0	20.32	No	80
1	TX	9.0	22.85	Yes	8.0	20.32	Q	89
2	TX	9.0	22.85	Yes	7.0	17.78	No	78
3	TX	7.0	17.78	No	9.0	22.85	Q	129
4	TX	10.0	25.40	No	7.5	19.05	No	75
5	NJ	10.0	25.40	Yes	7.5	19.05	No	75
6	IL	8.0	20.32	No	8.5	21.59	No	106
7	IL	9.5	24.13	No	11.0	27.94	No	116
8	IL	11.0	27.94	No	11.0	27.94	No	100
9	IL	12.5	31.75	Yes	11.0	27.94	No	88
10	IL	10.0	25.40	Yes	9.0	22.85	Q	90
13	MI	10.0	25.40	No	8.0	20.32	No	80
14	MI	10.0	25.40	Yes	8.5	21.59	No	85
15	NY	9.0	22.85	No	7.5	19.05	No	83
16	NY	9.0	22.85	No	7.5	19.05	No	83
17	NY	9.0	22.85	No	7.5	19.05	No	83
18	NY	10.0	25.40	No	9.0	22.85	No	90

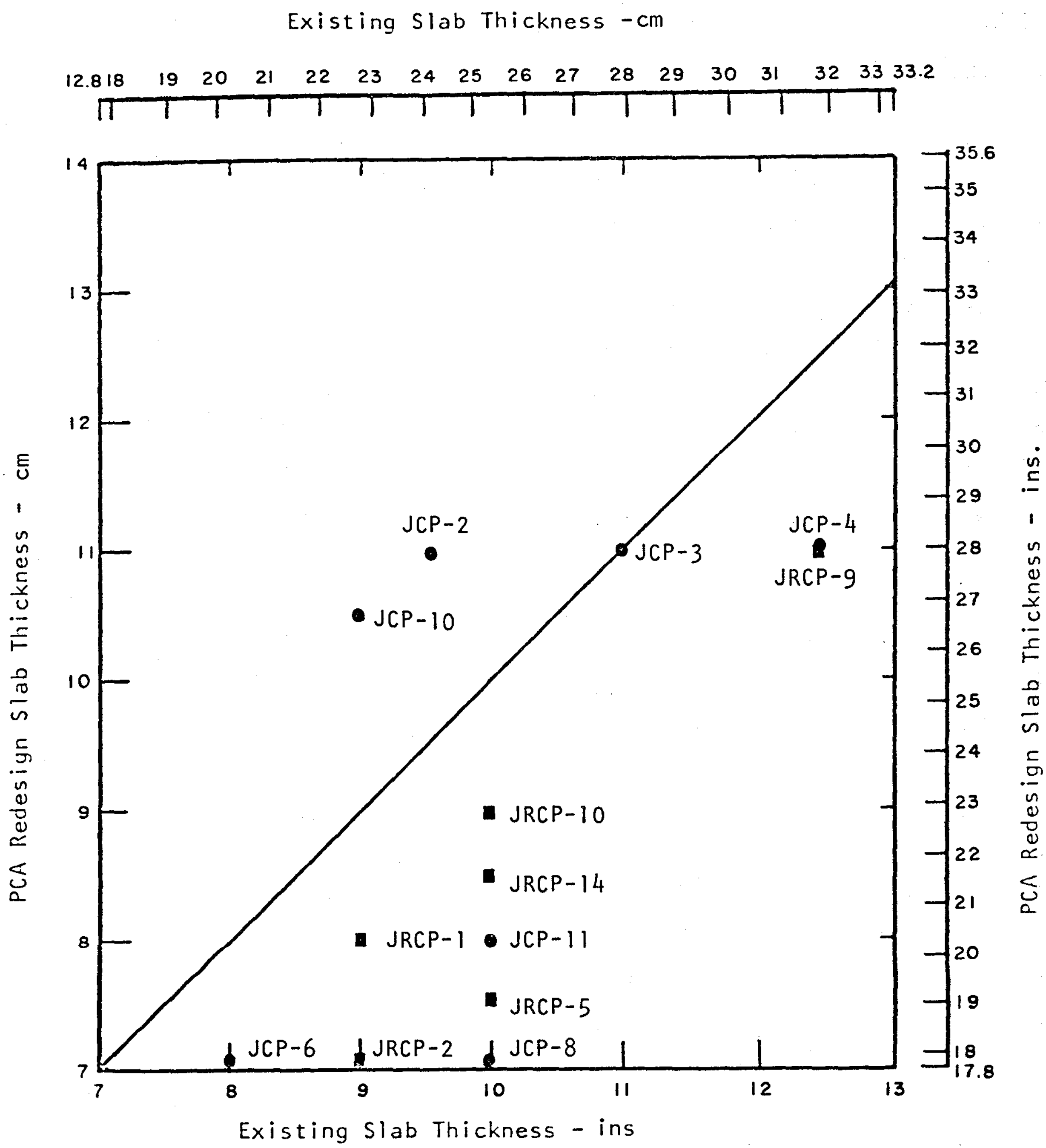


Figure 4.4. JCP and JRCP Redesign Slab Thickness Versus Existing Slab Thickness (all projects shown provided maintenance-free performance.)

3. JRCP Maintenance-Free Projects - Of the 16 JRCP sections surveyed, 7 provided maintenance-free performance. A plot of existing versus redesigned slab thicknesses is given in Figure 4.4. Observations of the redesigns indicate that all the redesigns by the PCA method gave less thicknesses than the existing sections. Therefore, pavements designed by the PCA method would not be expected to provide zero-maintenance pavements.

4. JRCP Projects Requiring Maintenance - Three projects were redesigned at greater than actual pavement thicknesses. It is noted that 2 of these projects, JRCP-6 and 7, still would not provide zero-maintenance performance with the thicker redesigns of 8.5" and 11", respectively, because a pavement subjected to the same loading conditions and having an 11" slab, JRCP-8, required maintenance as a result of serious transverse cracking apparently caused by traffic loadings and slab curling. The increase in thickness of the remaining project, JRCP-3, from a 7" to a 9" pavement may provide an adequate design. Some distress on this section is probably caused by loading, but some also from swelling soils, and an incremental increase in thickness may not be the complete solution for this project. All other redesigns were less than the existing structures, thus PCA fails to provide zero-maintenance performance for any of these cases.

Joint Design. Several of the PCA recommendations can be evaluated based upon the field performance of the projects surveyed.

1. Type and Spacing of Joints - The PCA manual indicates that contraction joints are adequate if certain conditions are met, and that expansion type joints are not necessary. One of these requirements, and probably the most important one, requires a contraction joint spacing of 40 feet or less.

Ten of the 18 JRCP projects (JRCP-4, 6, 7, 8, 9, 10, 11, 12, 13, and 18) surveyed have contraction joints only. Seven of these (JRCP-4, 6, 7, 8, 9, 10, and 11) have joint spacings from 30 to 56 ft, and none of them has visible joint deterioration. Some of them have evidence of joint lockup which will be subsequently discussed. Three other projects (JRCP-12, 13, and 18), with 100 ft joint spacing, showed serious joint deterioration (blowups and spalling). Thus, in general, the project evaluations support the PCA recommendation of approximately 40 ft maximum contraction joint spacing in providing maintenance-free joints. Three of the JRCP projects had expansion joints exclusively with doweled LTDS spaced from 30 to 100 ft. None of these projects showed significant joint deterioration. Also several other pavements with only expansion joints were examined in New Jersey that were 20-30 years old with maintenance-free performance. The 3 projects with a combination of contraction and expansion joints also showed no significant joint deterioration, and none evidenced joint lockup. Thus, projects with expansion type joints have provided excellent performance for high volume pavements for which no recommendations are provided by PCA. Discussions with many of the engineers interviewed and results from the joint spacing and type experiments conducted in the 1940s and 1950s (177) indicate that a mixture of the two types of joints is not desirable, however.

Two of the 12 JCP projects with short joint spacing of 12 to 18 ft (and one with 30 ft) showed serious faulting, and none had serious spalling or blowups. Therefore, it appears that the PCA recommendations for joint type are adequate, provided joint spacing is moderate.

2. Load Transfer System - Eight JCP projects out of 12 with short joint spacing did not contain mechanical load transfer devices. Two of these projects under heavy traffic had significant faulting. The other

projects exhibited only minor to moderate faulting, even though some were subjected to heavy traffic. All these latter projects had either a stabilized subbase (JCP-6), or skewed joints (JCP-10), and were in warm-dry climates. One project (JCP-12) with a 30 ft joint spacing and a granuar subbase low on fines, and located in a wet-non/freeze area with heavy traffic, did not exhibit significant faulting. Based upon the projects observed, it appears that skewed joints and stabilized subbases, or at least non-pumping subbases, will minimize joint faulting, but will not completely eliminate it. Climatic effects also influence the degree of faulting.

The PCA recommendations for dowels or other load transfer devices are essentially the same as for the AASHO procedure. Thus, the discussion relative to the projects surveyed is the same as for the AASHO method.

Three of the JRCP projects surveyed contained mechanical load transfer devices other than dowels. There was severe joint deterioration on two of these projects (JRCP-15 and 16) having joint spacings of 100 and 60 ft, respectively. The proprietary LTD used in JRCP-1 provided good joint performance. This project had a short joint spacing (30 ft), and is located in a wet/non-freeze/thaw area.

3. Depth of Contraction Joints - PCA recommends that the depth of the joint be 25 percent of the slab thickness. The 25 projects containing contraction joints had a joint depth of 20 to 28 percent of the slab thickness, and none had any distress to be attributed to depth of joint on any of these projects.

Reinforcement Design. PCA recommends the same equation (Equation 4.3) as AASHO Interim Guide for design of reinforcing bars or welded fabric. The adequacy of the PCA reinforcement for JRCP is therefore similar to that for the AASHO Interim Guide.

Summary of Structural Design

The theoretical deficiencies inherent in the PCA thickness design procedure limits its ability to design adequate pavement thicknesses required for zero-maintenance performance in high traffic volume situations. The conclusion is supported by the analytical evaluation of the in-service projects where only four projects out of 23, all plain jointed concrete pavements, were found, after redesign, to provide adequate pavement structures. Traffic load associated cracking distress was the major JCP distress found on the projects requiring maintenance. The PCA method fails to provide adequate thickness to design against this type of distress. The major JRCP distress occurred due to a combination of long term climatic effects and traffic loads. Climatic effects are not considered in the PCA thickness design procedure; therefore the PCA method will not design pavements adequately against distresses due to climatic effects.

Joint Design Summary

Recommendations are provided by PCA for joint type, spacing of contraction joints, shape, doweled load transfer devices including corrosion proofing, and aggregate interlock load transfer system. Most of these recommendations are, in general, supported by the field performance study. Other items not covered in the PCA, but shown to provide excellent field performance are expansion joints and sealant types. Also, specific recommendations are not included for determining when aggregate interlock is sufficient for load transfer and when mechanical devices are needed to prevent serious faulting for JCP.

4.2.4 PCA Design Procedure Applied by States

When the basic PCA approach to the thickness design of concrete pavements was applied by the individual state agencies, there were only minor modifications in the procedures. Differences were found in the manner of estimating traffic, subgrade support, and in the factor of safety applied to the concrete strength. At least one state (21) does not use load transfer devices in the dry/non-freeze areas, but does in the wet and mountainous areas.

A review of the PCA approach as applied by the states lead the project staff to conclude there are no significant differences in the final results when the method used is the PCA method, or that recommended by the states. Thus, all conclusions reached with regard to the PCA procedure earlier are valid.

4.3 CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

The design of CRCP has mostly been based on engineering judgment. All existing formal procedures of design are based upon the results from the AASHO Road Test. Two procedures are evaluated here.

4.3.1 AASHO Interim Guide

The AASHO Interim Guide for concrete pavement design was originally developed for jointed concrete pavements. However, the most recent version of the Interim Guide (1972) contains some recommendations and nomographs for the design of continuously reinforced concrete pavements (CRCP).

Conceptual Evaluation

The structural design of CRCP is based on Equation 4.2 which was developed from 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 JCP or JRCP design is the use of the joint continuity coefficient, J, which is 2.2 for continuous instead of 3.2 for jointed pavements. The direct effect of reducing J is a reduction in the calculated maximum corner stress in the concrete slab by 0.6875 (= 2.3/3.2), which is a significant reduction in stress. This usually results in approximately 1-2 inches reduction in slab thickness for CRCP as compared to JRCP or JCP.

The longitudinal reinforcement design requirements are determined by the following equation:

$$p_s = (1.3 - 0.2f)\left(\frac{f_t}{f_s}\right) 100 \qquad (4.7)$$

where

p_s = percentage of required steel

f = average coefficient of resistance between the slab and the subbase

f_t = tensile strength of PCC, psi

f_s = allowable working stress in steel, psi (0.75 of yield strength is recommended)

The transverse steel required, if provided, is determined similar to JRCP using Equation 4.3. Only deformed wire or bar reinforcement is recommended for CRCP.

Recommendations for bar size and spacing are provided for by the criteria that the ratio of bond area of longitudinal bars to the concrete slab volume must meet the following requirement:

$$Q = \frac{4P}{D_b} = \frac{\pi D_b}{SD} \geq 0.03 \tag{4.8}$$

where

Q = ratio of bond area to concrete volume, in.^{2}/in.^{3}

D = slab thickness, inches

P = ratio of area of longitudinal steel to the area of concrete

S = steel spacing, inches

D_b = diameter of reinforcing bar, inches

Specific limitations of the AASHO procedure are summarized as follows:

1. General - Nearly all limitations previously presented for the AASHO procedure for design of jointed concrete pavements 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 to CRCP of the equations developed for and using the performance data of jointed pavements at the AASHO Road Test has never been verified.

2. Joint Continuity Factor - Justification of the value for J = 2.2 is nebulous. The value was apparently first suggested by Hudson and McCullough (61) based on "comparisons of previous design procedures and performance studies." A value of 2.33 was later suggested by Treybig (138) based on edge deflection comparisons between JRCP and CRCP in Texas. This parameter

has a very significant effect on the resulting slab thickness. While this value may be appropriate in certain regions of the country under relatively dry climatic conditions, it may not be adequate in areas where there is loss of support due to moisture and edge pumping from heavy loads.

3. Reinforcement - The expression given by Equation 4.7 is a greatly simplified estimation of required steel percentage. Combinations of many factors affecting CRCP generally produce situations where steel stresses may exceed the yield stress resulting in rupture and/or undesirable crack opening or crack width. A comprehensive discussion of the factors and their effect on crack spacing and width is given by Abou-Ayyash (6).

4. Joints - Construction joints and terminal systems are critical factors in CRCP, and many failures have occurred at these locations. The AASHO Interim Guide does not provide any recommendations for these critical factors.

Based upon these limitations, it is doubtful that the AASHO Interim Guide will provide an adequate guide for structural thickness, reinforcement, and necessary joints to provide maintenance-free pavements in many design situations. Many important recommendations are missing altogether, and other recommendations are based on insufficient data.

Analytical Evaluation

A summary of the existing and redesigned thicknesses for each of the 12 CRCP projects is given in Table 4.6. The results are discussed in the following section.

Table 4.6 Summary of Existing and Redesign CRCP Slab Thickness using the AASHO Interim Guides

CRCP Project No.	State	Existing Slab (in.)	Existing Slab (cm)	Zero Maintenance Performance	AASHO Redesign Slab (in.)	AASHO Redesign Slab (cm)	Redesign as % of Existing	Does Redesign Give Zero-Maintenance Performance?
1	IL	8.0	20.32	No	10.0	25.40	125	Yes
2	IL	10.0	25.40	Yes	8.0	20.32	80	No
3	MN	9.0	22.85	Yes	6.0	15.24	67	No
4	MI	9.0	22.85	Yes	7.5	19.05	83	No
5	TX	8.0	20.32	Yes	6.6	16.76	83	No
6	TX	8.0	20.32	No	6.2	15.75	78	No
7	TX	8.0	20.32	Yes	7.0	17.78	86	No
8	TX	8.0	20.32	No	7.0	17.78	86	No
9	TX	8.0	20.32	Yes	6.4	16.26	80	No
10	TX	8.0	20.32	No	6.7	17.02	84	No
11	TX	9.0	22.85	Yes	7.4	18.80	82	No
12	NJ	10.0	25.40	No	9.5	24.13	95	No

1. Maintenance-free Projects - Seven out of the 12 CRCP projects exhibited maintenance-free performance. The redesigns for these seven pavements gave slab thicknesses ranging from 1 to 3 inches thinner than the existing slabs. It is doubtful that any of these 7 redesign thicknesses would provide maintenance-free performance, and that considerable load associated cracking would occur if they were constructed with the redesign thickness. A plot of existing versus redesign CRCP slab thicknesses is shown in Figure 4.5.

2. Projects Requiring Maintenance - The remaining 5 projects required various types of maintenance, but mainly from traffic load associated cracking and breakup. Only one of these pavements has redesign thickness greater than the existing thickness. This section (CRCP-1), the Dan Ryan Expressway in Chicago (truck lanes), is completely deteriorated after only 9 years of service and consists of an 8 inch CRCP slab over a granular subbase. The 18-kip ESAL applications over 9 years are estimated to be 35 million in the heaviest traveled lane. It is interesting to note that the redesign of the pavement in 1971 consisted of a 10-inch CRCP slab, a thickness equal to the redesign thickness shown here. It is believed that this structure will be adequate to prevent load associated distress, since CRCP-2, located in close proximity to CRCP-1 in Chicago, has a 10-inch CRCP slab, and has given zero-maintenance performance under heavy traffic for over 10 years.

Reinforcement Design. The required steel percentage was redesigned for each CRCP project using the AASHO Equation 4.7. A summary of the

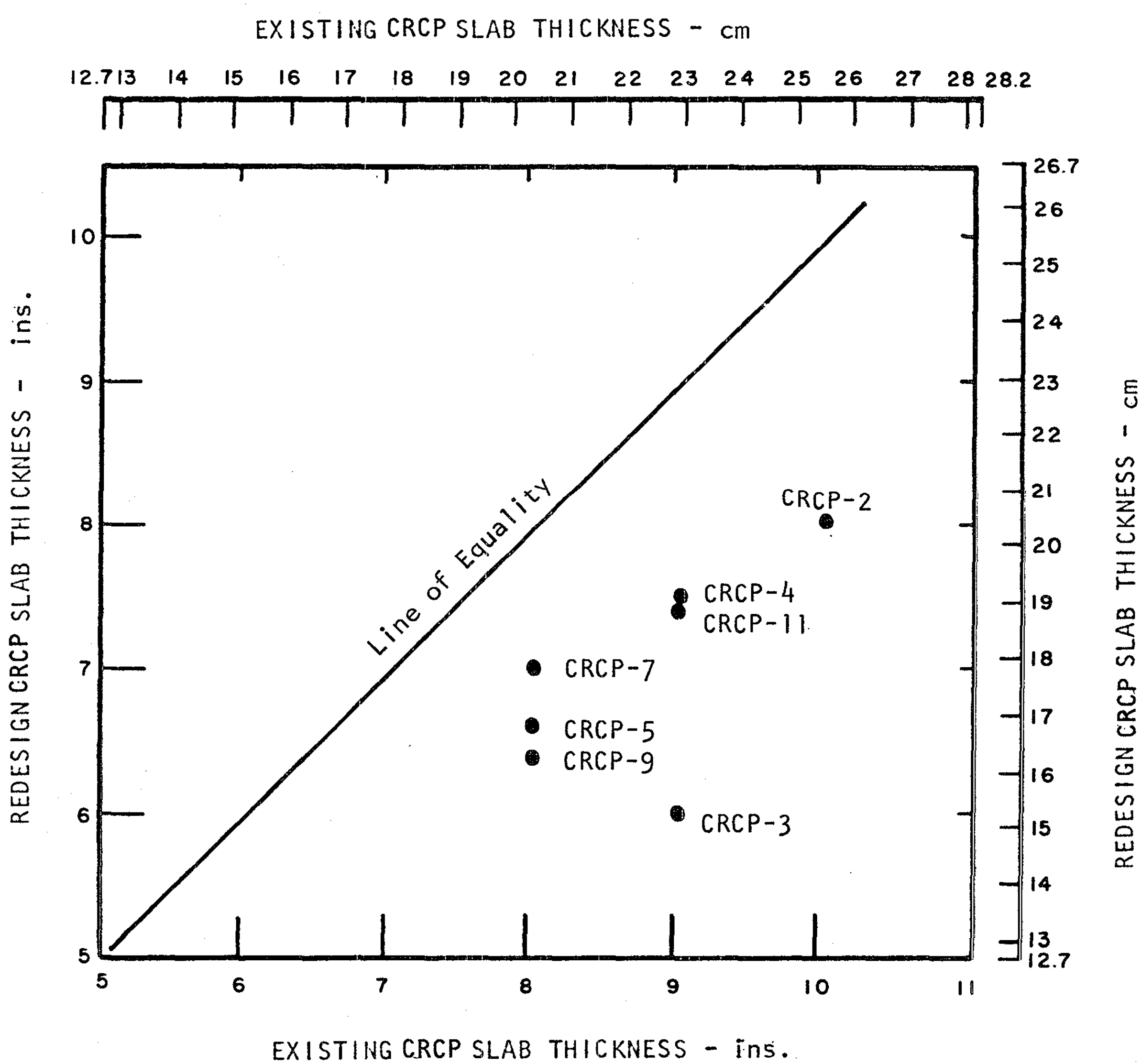


FIG. 4.5 Existing CRCP slab thickness versus redesign thickness by AASHO Interim Guide (each project shown provided maintenance-free performance).

existing versus the redesign percent steel is given in Table 4.7. Nine out of 12 projects had existing reinforcement less than that by the redesign using the AASHO equation, with an average difference of approximately 13 percent. Only two projects (CRCP-1 and 12) had distress which indicated a rupture of the reinforcement. These two projects were also the only projects with smooth wire fabric as the reinforcing steel. Since crack spacings and widths on the other projects were within normal ranges, it appears from this limited data that Equation 4.7 provides adequate longitudinal reinforcement.

The corrosion of the reinforcement was of concern to several highway agencies in wet-freeze/thaw regions, particularly in urban areas where large amounts of deicing salt are used each winter. None of the projects surveyed in wet-freeze/thaw regions were over 10 years old, hence, no conclusions can be reached in this regard. However, in Chicago and Detroit, some reinforcement was lost after 10 years of service, as discussed in Chapter 3, but this loss is not considered in the reinforcement design.

Summary

Results from both conceptual and analytical evaluations show that the AASHO Interim Guide does not provide adequate structural capacity for maintenance-free pavements. A major type of distress in CRCP was observed to be edge cracking and breakup of the CRCP slab. A more adequate structural section must be provided to prevent this type of distress. The reinforcement design appears to provide adequate reinforcement, but the method is greatly simplified and contains very few parameters, hence situations could arise where it would not provide adequate reinforcement.

Table 4.7 Summary of Existing CRCP Longitudinal Reinforcement and Redesign AASHO Reinforcement

CRCP Project No.-State		Reinforcement Type	Existing Reinforcement Percent	AASHO Redesign Reinforcement Percent	Redesign As % Of Existing Reinforcement	Reinforcement Steel Rupture	Climatic Region
1	IL	WF*	0.613	0.667	109	Yes	WF
2	IL	DB**	0.613	0.667	109	No	WF
3	MN	DB	0.619	0.653	105	No	WF
4	MI	DB	0.681	0.644	95	No	WF
5	TX	DB	0.511	0.636	124	No	W
6	TX	DB	0.590	0.650	110	No	W
7	TX	DB	0.511	0.557	109	No	D
8	TX	DB	0.511	0.616	121	No	D
9	TX	DB	0.511	0.658	129	No	D
10	TX	DB	0.511	0.513	100	No	D
11	TX	DB	0.614	0.604	98	No	D
12	NJ	WF	0.736	0.666	90	Yes	WF

*WF = smooth wire fabric
**DB = deformed reinforcing bar
W = wet climate, no freeze-thaw
D = dry climate, no freeze-thaw
WF = wet climate, freeze-thaw

4.3.2 American Concrete Institute (ACI) Design Procedure

The ACI procedure for design of continuously reinforced concrete pavements was developed by ACI Committee 325, Subcommittee VII, in 1972 (5). The method is very similar to the AASHO Interim Guide, but contains a few differences and some important additions. The structural design equation is, however, based directly upon the results of the AASHO Road Test plus the inclusion of Spangler's corner equation.

Conceptual Evaluation

A complete derivation of the design equation is provided in the procedure (5). The derivation, basically the same as for the AASHO Interim Guide Equation 4.2, is accomplished by the following two correlations:

1. Stresses calculated from corner strains measured under an 18-kip single-axle vibrating load on loop 1 of the AASHO Road Test correlated to the stresses predicted by the Spangler's equation due to a 9000 lb wheel load for the same pavements of the AASHO Road Test.

2. The term (D + 1) correlated to the stresses (strains) measured at the AASHO Road Test loop 1.

In this derivation, a "pavement life term" was applied to the final equation as derived by multiplying log w by .9155. This affected an increase in the design slab thickness similar to that achieved by the use of a working strength for PCC equal to ".75 x flexural strength" in the AASHO Interim Guide.

The final design equation is as follows:

$$\log W_{18} = -8.682 - 3.512 \log \left\{\frac{J}{FD^2}\left(1 - \frac{2.61a}{Z^{0.25}D^{0.75}}\right)\right\} - \frac{0.1612}{B} \qquad (4.9)$$

where all terms are defined previously for Equations 4.1 and 4.2. The initial and final serviceability index values incorporated in this equation are 4.5 and 2.5, respectively.

Specific limitations include all of those listed for the AASHO Interim Guide for structural design of CRCPs since Equation 4.9 provides approximately the same slab thickness for the same inputs. Based upon comparisons on several projects, the ACI Equation 4.9 averages slab thickness about 0.4 inches greater than the AASHO Interim Guide Equation 4.2, using J = 2.2 in both cases. No additional recommendations are provided for the selection of J except that "the committee considers a value of J of 2.2 to be reasonable" (5).

The reinforcement design recommendations are similar to AASHO, and the expression used for pavements with subbases of either granular or stabilized materials is given as follows:

$$p_s = \left(\frac{f_t}{f_y - nf_t}\right)(1.3 - 0.2f)(100)(1.3) \qquad (4.10)$$

where

p_s = percentage of required steel

f_y = yield of strength, psi

f_t = tensile strength of concrete, psi

$n = E_s/E_c$

E_s = modulus of elasticity of steel

E_c = modulus of elasticity of concrete

f = average coefficient of resistance between slab and subbase

This expression provides approximately the same steel requirements as Equation 4.7. Several recommendations are provided for steel placement, type, and lap lengths.

Considerable information is provided by ACI on terminal anchorage. Details are given for anchor lugs and for expansion systems. No recommendations on construction joint design are provided.

Based upon these limitations, it is doubtful that the ACI procedure will provide adequate structural CRCP sections for zero-maintenance pavements. The recommendations for reinforcement and joints are more adequate than with AASHO Interim Guide, but still lack important details.

Analytical Evaluation

A summary of the existing and redesigns for each of the 12 CRCP projects is given in Table 4.8. A plot of existing slab thicknesses versus redesign thicknesses is shown in Figure 4.6. The results are similar to those obtained using the AASHO procedure. The ACI thickness averages about 0.4 inches greater than the AASHO, and does not significantly affect the general conclusions reached for the AASHO method.

Reinforcement design also gave results approximately the same as the AASHO procedure. A summary of results obtained using the ACI Equation 4.7 is given in Table 4.9. Results show that the ACI procedure provides slightly more steel than the AASHO method, hence, the same general conclusions reached for the AASHO method are applicable to the ACI method.

Table 4. 8. Summary of Existing and Redesign CRCP Slab Thickness Using the ACI Procedure

CRCP Project No.-State		Existing Slab (in.)	Existing Slab (cm)	Zero Maintenance Performance		ACI Redesign Slab (in.)	ACI Redesign Slab (cm)	Redesign As % of Existing	Does Redesign Give Zero-Maintenance Performance?	
1	IL	8.0	20.32	No		10.7	27.18	134		Yes
2	IL	10.0	25.40		Yes	8.7	22.10	87	No	
3	MN	9.0	22.85		Yes	6.3	16.00	70	No	
4	MI	9.0	22.85		Yes	8.2	20.83	91		Q
5	TX	8.0	20.32		Yes	7.0	17.78	88	No	
6	TX	8.0	20.32	No		6.3	16.00	79	No	
7	TX	8.0	20.32		Yes	7.3	18.54	91	No	
8	TX	8.0	20.32	No		7.3	18.54	91	No	
9	TX	8.0	20.32		Yes	6.8	17.27	85	No	
10	TX	8.0	20.32	No		7.0	17.78	88	No	
11	TX	9.0	22.85		Yes	8.0	22.32	89	No	
12	NJ	10.0	25.40	No		10.3	26.13	103		Yes

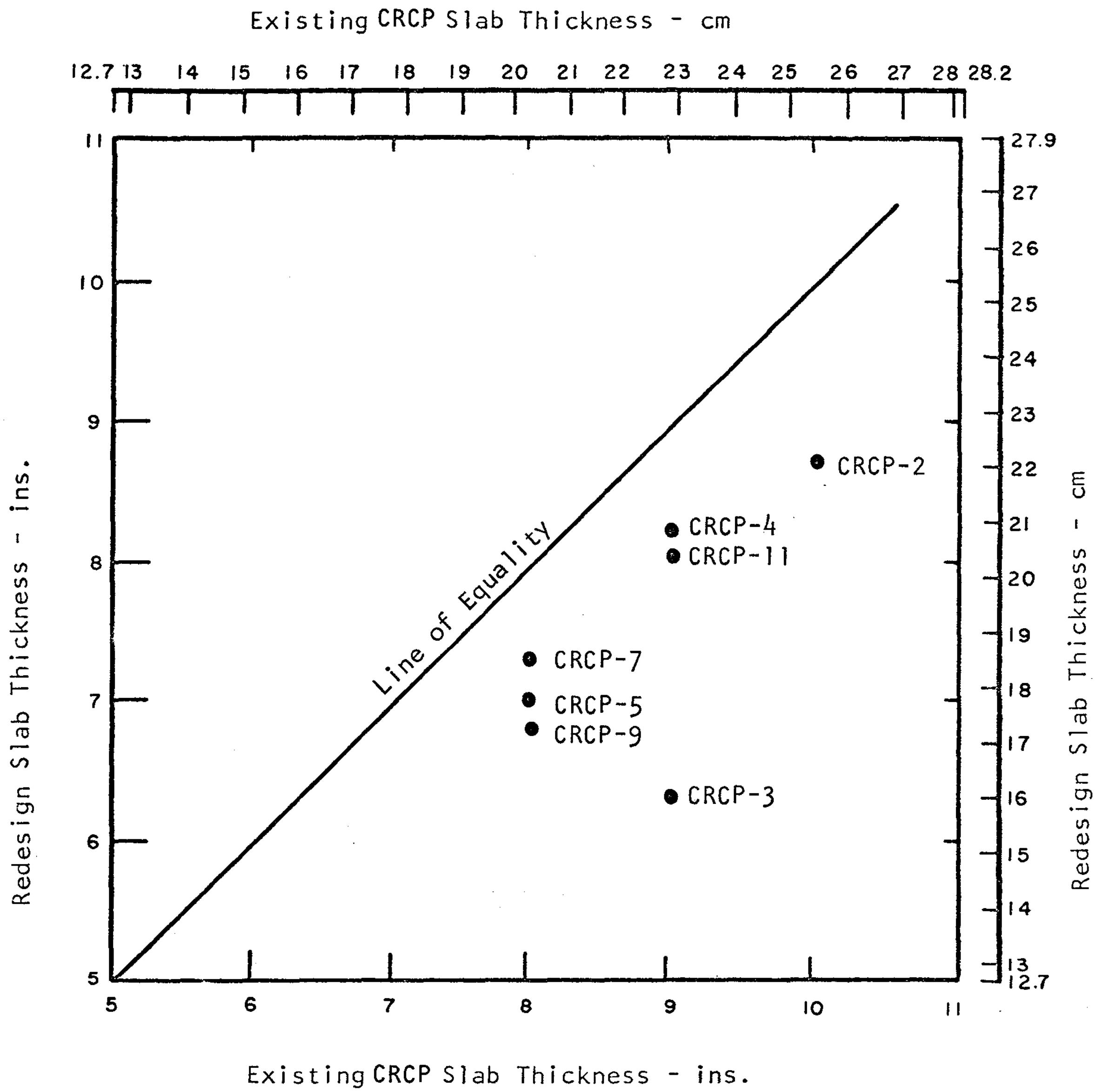


Figure 4.6 Existing CRCP slab thickness versus redesign thickness by ACI method (each project shown provided maintenance-free performance).

Table 4.9 Summary of Existing CRCP Longitudinal Reinforcement and ACI Redesign Reinforcement

CRCP Project No.-State		Reinforcement Type	Existing Reinforcement Percent	ACI Redesign Reinforcement Percent	Redesign As % Of Existing
1	IL	WF *	0.613	0.671	109
2	IL	DB **	0.613	0.671	109
3	MN	DB	0.619	0.658	106
4	MI	DB	0.681	0.650	95
5	TX	DB	0.511	0.641	125
6	TX	DB	0.590	0.655	111
7	TX	DB	0.511	0.561	110
8	TX	DB	0.511	0.620	121
9	TX	DB	0.511	0.662	130
10	TX	DB	0.511	0.590	115
11	TX	DB	0.614	0.609	99
12	NJ	WF	0.737	0.669	91

*WF = deformed wire
**DB = deformed reinforcing bar

Refer to Table 4.8 for Reinforcement Steel Rupture and Climatic Region information.

Summary

Results from both conceptual and analytical evaluations of the ACI design procedure for CRCPs show many limitations and deficiencies. As for AASHO, it is doubtful that the procedure will provide an adequate CRCP section with maintenance-free performance for long time periods. The design procedures evaluated indicate slab thicknesses from 1 to 3 inches less than in existing pavements that have performed maintenance-free over fairly long time periods. Reinforcement steel provisions to the ACI procedure are similar to AASHO procedure, but considerably more guidance is provided in the ACI procedure for reinforcement size, spacing, and laps, all of which appear to be adequate. Recommendations are also provided by ACI for CRCP terminal systems.

4.4 FLEXIBLE PAVEMENTS

4.4.1 AASHO Interim Guide

Introduction

The "AASHO Interim Guide for Design of Pavement Structures" (2) was originally published in 1962 by the AASHO Design Committee through its subcommittee on pavement design practices. The Guide was evaluated under NCHRP Project 1-11 (58), and subsequently republished in 1972 (1). The basic procedure in the 1972 Interim Guide was not changed from that originally adopted in 1962.

Conceptual Evaluation

The flexible pavement Interim Guide is based upon the results of the AASHO Road Test with the inclusion of a regional climatic factor and a soil support index. The design equation is completely empirical. The following section summarizes its derivation.

1. The Road Test data provided an empirical relationship between the structural number (representing various layers), axle load magnitude, number of axle load applications, axle type, and initial and terminal serviceability indices for the Road Test conditions (i.e., specific environment and materials) as follows:

$$\log_{10} W = \log \rho + G/\beta \tag{4.11}$$

where

W = number of applications of axle load L_1 and axle type L_2 to serviceability index p_2

$$\log \rho = 5.93 + 9.36 \log (SN + 1) - 4.79 \log (L_1 + L_2) \times 4.33 \log L_2$$

$$\beta = \frac{0.40 + 0.081\ (L_1 + L_2)^{3.23}}{(SN + 1)^{5.19} L_2^{3.23}}$$

$$G = \log \frac{P_1 - P_2}{P_1 - 1.5}$$

L_1 = load on a single or a tandem axle, kips

L_2 = axle code (1 for single axles and 2 for tandem axles)

SN = structural number = $a_1D_1 + a_2D_2 + a_3D_3$

D_1, D_2, D_3 = thickness of surface, base, or subbase, respectively (inches)

a_1, a_2, a_3 = corresponding statistically determined layer coefficients

P_1 = initial serviceability index

P_2 = terminal serviceability index

2. The empirical model given by Equation 4.11 was modified and extended to allow for different subgrades and environment. The basic assumptions made for this extension are as follows:

a. The total load applications are an inverse function of the regional factor such that

$$W_{18} = \frac{N_{18}}{R} \tag{4.12}$$

or

$$\log W_{18} = \log N_{18} + \log \frac{1}{R}$$

where

N_{18} = total unweighted load applications

R = regional factor ranging from 1 to 5

b. The soil support can be defined by a scale ranging from 3.0 to 10.0, three representing the soil at the Road Test, and ten representing the crushed rock base at the Road Test thick enough to minimize the effect of the roadbed soil. A linear scale between point 3.0 and 10.0 was assumed such that

$$\log W_{18} = \log N'_{18} + K(S_i - 3.0) \tag{4.13a}$$

or

$$10^{K(S_i - 3)} = \frac{W_{18}}{N'_{18}}$$

where

S_i = soil supporting value for any soil condition "i"

N'_{18} = total load applications for Road Test condition

W_{18} = total load applications for condition "i"

K = constant

Therefore, for the following AASHO Road Test condition:

SN	S	W_{18}(daily)
1.98	10	1000
1.98	3	2.5

a solution of Equation 4.13a yields the following value of constant K

$$K = 0.372$$

hence

$$\log W_{18} = 0.372\,(S_i - 3) + \log N'_{18} \qquad (4.13b)$$

The following model results when Equations 4.12 and 4.13b are incorporated in Equation 4.11:

$$\log W_{18} = 9.36 \log (SN + 1) - 0.20 + \frac{G}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + \log \frac{1}{R} + 0.372\,(S_i - 3.0) \qquad (4.14)$$

3. The specific conditions under which the Road Test was conducted and the empirical design model are as follows:

a. Construction Control - Extremely high quality control was exercised during the construction. Variations in asphalt concrete, aggregates, moisture, density, subgrade soil properties, etc., were much lower than those which will be obtained in a normal highway construction.

b. Length of Test Pavements - The test sections were 100 ft in length.

c. Base - The base course material was a well-graded high quality crushed limestone with high stability and with CBR greater than 80.

d. Subbase - A uniformly graded sand/gravel with considerable fine content.

e. Subgrade - A fine grained A-6 soil with a CBR of 2 to 4.

f. Climate - The northern Illinois climate has an annual precipitation of about 30 inches or +4 inches more annual precipitation than evaporation. The average depth of frost penetration is 30 inches, and the number of freeze-thaw cycles is 12 per year at the subbase level in the pavement.

g. Length of Test - The test was conducted over a two-year period.

h. Number of Load Applications - The maximum load repetitions applied to any pavement section were 1,114,000, but many sections failed before this number was reached.

The resulting empirical Equation 4.11 derived from the Road Test data specifically relates to these conditions. The measured error of Equation 4.11 in predicting pavement performance is fairly high. The mean residual error in predicting log W to the terminal serviceability level of 2.0 is illustrated by shaded bands in Figure 4.7. These bands indicate the ranges in load applications to include 90 percent of all performance data. For example, if SN were 4.0, the resulting number of 30-kip single axle load applications to serviceability index of 2.0 may range from 80,000 to 600,000. If Equation 4.11 is used for conditions different than those for which it was developed, it range of accuracy or associated error of prediction is unknown, but probably will be much larger. Equation 4.14 has parameters for different subgrade soil support and climatic conditions, but the accuracy of adjustments with these parameters is unknown. It should be noted that there is no safety factor applied in this method for determining the required SN.

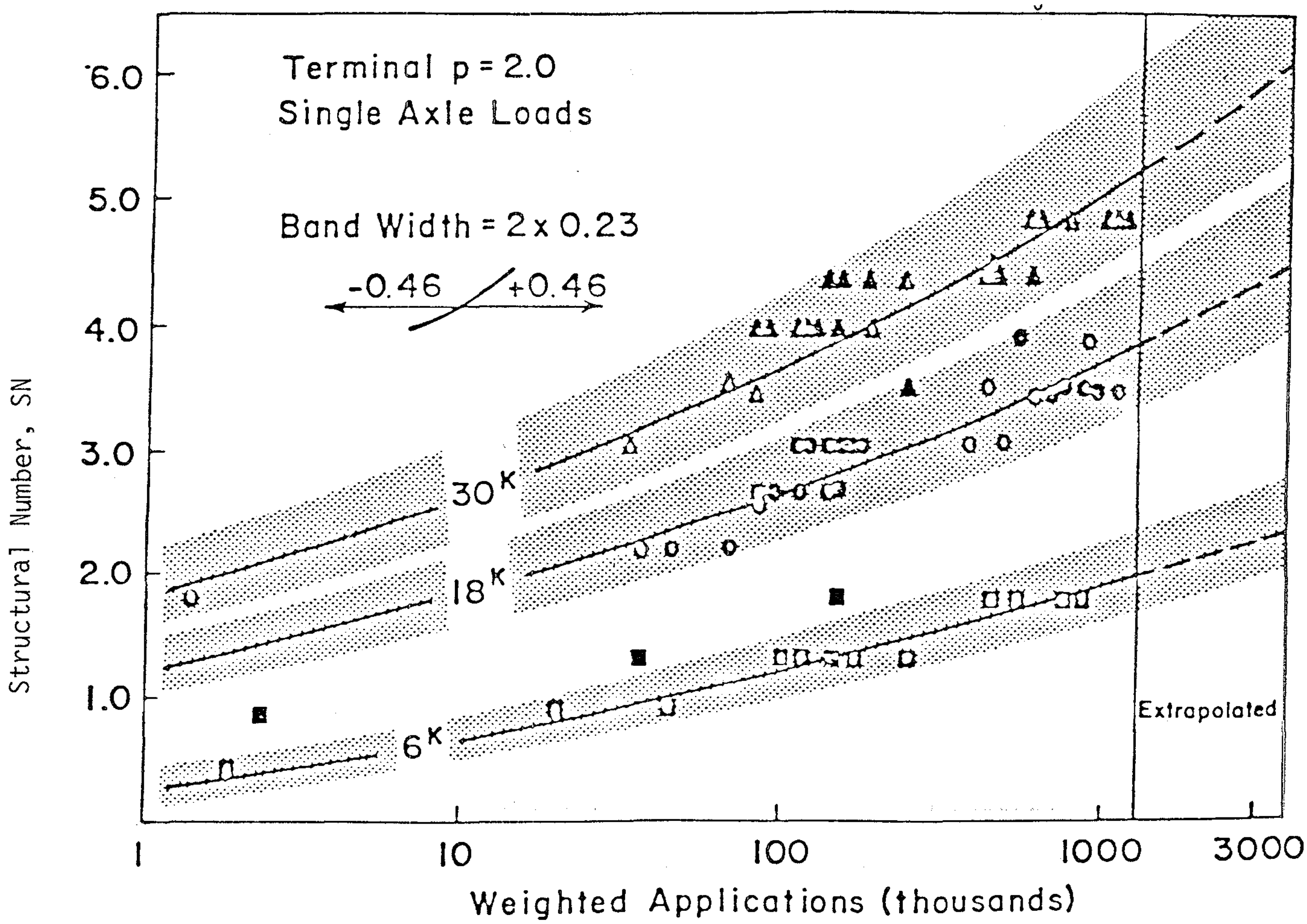


Figure 4.7. Illustration of Error of Prediction of Basic AASHO Design Model (From Ref. 56).

Based upon the above facts, the possible limitations of the Interim Guide are summarized as follows:

1. Variability - The greatest limitation is that Equations 4.11 and 4.14 are based upon very short pavement sections where construction and material quality were high. Normal projects are several miles in length, contain much greater variability, and usually exhibit localized failures due to variations in materials and construction. Therefore, projects designed with the Interim Guide would show significant localized failures before the average project terminal serviceability index drops to P_2.

2. Loss of Foundation Support - During the two consecutive spring periods, the base, subbase, and/or subgrade at the Road Test showed a significant decrease in strength which was indicated by higher surface deflections. A large proportion of the flexible pavements reached terminal serviceability of 1.5 during this period. The amount of cumulative serviceability loss occurring during the spring period due to loss of subgrade support would be increased if the design life of the pavement passes through 20 to 40 such periods. Therefore, Equations 4.11 and 4.14 are critically biased, and only consider two years of climatic damage for a given amount of traffic.

3. Design Period - The pavements under consideration in this project are to be designed for time periods ranging from 20 to 40 years. The 1,114,000 applications upon which Equation 4.11 and 4.14 are based represent only a fraction of the load applications (10 to 50 million 18-kip ESALs) that would be applied over this time period on high volume pavement.

Even if the equations can reasonably be extrapolated for this large number of load applications, there are time dependent climatic effects that cause severe deterioration of the pavement (i.e., freeze-thaw damage, low temperature asphalt concrete cracking). For example, the detrimental effect of low temperature cracking is not included in Equation 4.14 since this cracking was not significant at the Road Test which lasted two years. In certain regions of the U. S., this distress mode is predominant in flexible pavements, and causes a great loss in serviceability. The hardening effects of asphalt cement over long time periods are also not included in Equation 4.14. This effect can result in increased pavement fatigue cracking and loss in serviceability. Therefore, the pavements would last less than the design analysis period predicted by Equation 4.14.

4. Climate - Flexible pavement performance is highly dependent on climatic conditions. If the climate is more severe than northern Illinois climate, the pavement will not last the predicted design life. If the climate is less severe, the pavement may last longer than that predicted by Equation 4.14. The regional factor, R, attempts to adjust for this effect, but no proven guidelines are available to guide in the selection of this factor.

5. Pavement Configuration - For a given structural number, there are numerous combinations of materials and thicknesses that are possible. Different pavements, each with the same SN, may have different behavior and performance characteristics. Equations 4.11 and 4.14 are only applicable to configurations similar to those provided at the AASHO Road Test, i.e., moderately thick AC surface over base, with or without subbase course.

6. Terminal Criteria - The terminal serviceability recommended by the Interim Guide for the design of primary highways is 2.5. Results provided in Chapter 5 show that this criteria is inadequate, and the probability of a flexible pavement requiring maintenance before the average project serviceability reaches 2.5 is very high.

In summary, based upon the above conceptual limitations, the AASHO Interim Guide is not expected to provide maintenance-free pavements in most locations in the U. S. The major theoretical deficiencies are 1) the lack of consideration of variabilities in materials and construction, 2) design periods and traffic being much greater than the data on which the design method is based, 3) the questionable extrapolation of load equivalency factors to other pavement materials and subgrades, 4) inadequate consideration of different climates, 5) different compositions of the pavement section for a given SN considered to have the same performance, which is not always valid, and 6) terminal design criteria being inadequate.

However, longer life than predicted by the design equations may result if the loss in serviceability during the spring thaw can be prevented, since most of the loss in serviceability in the Road Test pavements occurred during the spring period. No safety factor has been provided in the design procedure.

Analytical Evaluation

1. Maintenance-Free Projects - Of the nineteen flexible pavement projects analyzed, nine were maintenance-free as shown in Table 4.10. A plot showing the existing and redesigned structural numbers for main-

Table 4.1C Summary of Redesigns using the AASHO Interim Guide for Flexible Pavement

FLEX Project No.-State		Existing SN	Maintenance Free Performance?	Redesign SN	Does Redesign Provide Maintenance Free Performance?	Redesign As % of Existing SN
1	NH	5.06	No	5.90	Q	117
2	NJ	5.24	Yes	3.50	No	67
3	MN	7.88	No	4.70	No	163
4	MN	5.20	No	4.60	No	88
5	MN	5.24	No	3.30	No	63
6	MN	2.70	No	3.00	No	111
7	IL	5.70	No	5.30	No	93
8	IL	5.14	Yes	5.30	Yes	103
9	IL	5.82	Yes	5.30	Yes	91
10	IL	5.06	No	5.00	No	99
11	WA	5.50	Yes	3.10	No	56
12	WA	5.45	Yes	1.60	No	29
13	CA	4.85	No	3.90	No	80
14	CA	5.85	Yes	4.60	No	79
15	CA	5.95	Yes	3.30	No	55
16	UT	3.52	No	4.70	Q	134
17	AZ	3.00	No	5.00	Yes	167
18	GA	5.94	Yes	4.25	No	82
19	GA	4.30	Yes	2.50	No	58

Q = Questionable

tenance-free projects is shown in Figure 4.8. The "redesigns" of these projects indicate the following:

a. Only one of the nine zero-maintenance flexible projects redesigned by the AASHO Interim Guide method provided a SN greater than or equal to the existing structural systems.

b. Eight of the nine redesigned zero-maintenance flexible projects showed less structural sections than the existing. The AASHO design method provided structural numbers (SN) of 1 to 4 less than the existing SNs, and it is doubtful if these redesigns would give maintenance-free performance.

2. Projects Requiring Maintenance - Out of the ten projects that did not give zero-maintenance performance, five provided a SN greater than the existing. It is believed that only one of these redesigns is likely to provide maintenance-free performance.

Pavements constructed with approximately the same SN do not necessarily give the same performance. For example, FLEX-7 and FLEX-9 have approximately the same structural numbers, and are located adjacent to each other on I-80 in Illinois. FLEX-7 has developed severe problems of fatigue cracking and rutting, while FLEX-9 has only a few tight transverse cracks and moderate rutting which, until now, has not required any maintenance.

Summary

The conceptual evaluation of the flexible AASHO Interim Guide shows many deficiencies which were confirmed by the analytical evaluation. Only one of the 9 maintenance-free projects when redesigned had a greater

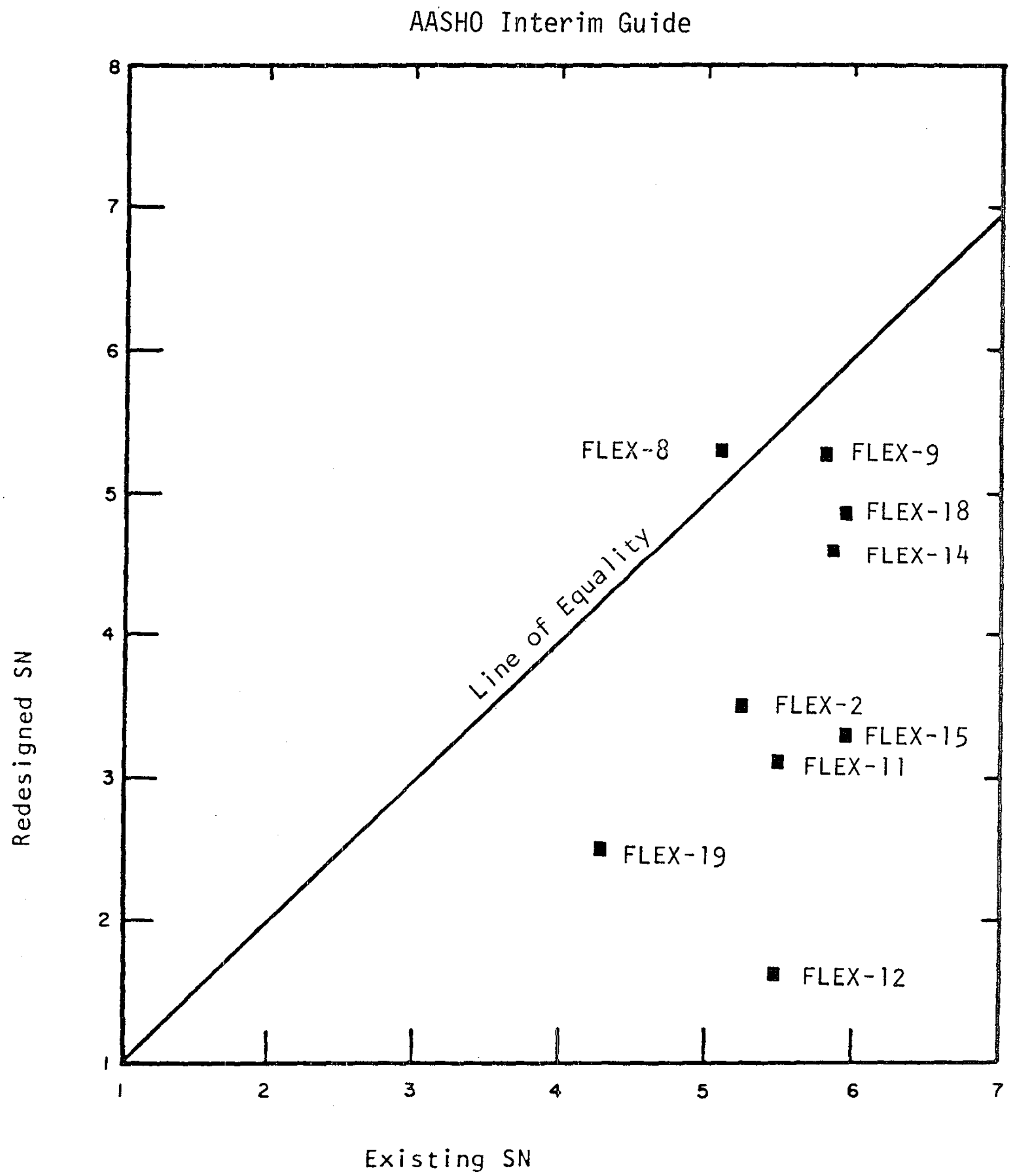


Figure 4.8. Flexible Redesign Structural Number (SN) versus existing SN (all projects shown provided maintenance-free performance).

SN than the existing pavement. Hence, the Guide will not provide maintenance-free pavement structures. In addition, the method provides no recommendations for preventing low-temperature cracking, which is a problem in many areas of the U. S.

4.4.2 Asphalt Institute Design Procedure

The Asphalt Institute design method for flexible pavements (9) "was developed using the observations at the AASHO Road Test, the WASHO Road Test, and experience with the design procedures of several states." The relationship between design and load applications has been based on the data from AASHO Road Test.

Conceptual Evaluation

The Asphalt Institute design method for flexible pavements is based upong the following:

1. Subgrade Strength - The CBR (or plate bearing value, since a general correlation exists) or the resistance value, R, is the measure of subgrade support which is correlated with soil support as in the AASHO Interim Guide.

2. Surface, Base, and Subbase Strength - The design method uses equivalence factors which are recommended as 1 inch of asphalt concrete = 2 inches of granular base = 2.7 inches of subbase.

3. Wheel or Axle Load - For this design method, an 18-kip single axle load is chosen as the base design load. The effects of the loads of other magnitudes are given in terms of equivalent 18-kip single-axle loads as follows:

$$W_{18} = W_L F_L \text{ for a given load } L$$

where

$F_L = 10^{0.11833(L_1 - 18)}$

W_{18} = equivalent 18-kip single axle applications

W_L = application of load L

F_L = load equivalency factor for load L

L_1 = single-axle load or .57 times the gross tandem-axle load, kips

4. Volume of Traffic - Traffic volume is evaluated in terms of W_{18} for a mixed-traffic situation, similar to the AASHO Interim Guide

$$W_{18} = \sum_{L=1}^{L=M} W_L F_L \qquad (4.15)$$

where M is the number of load categories.

5. Environment - Although the Asphalt Institute design method indicates the importance of the environment on the performance of flexible pavements, no effort was made to account for the different environments that could exist in different parts of the U. S.

6. Thickness/Load Relationship - Thickness design equation was developed by a multiple regression analysis of the AASHO Road Test data.

$$T = (-8.50 + 5.53 \log W_{18})\left(\frac{2.5}{CBR}\right)^{0.4} \qquad (4.16)$$

where

T = thickness factor = 2D

D = total thickness in inches of asphalt concrete above subgrade

CBR = California Bearing Ratio for the subgrade

In direct contrast to the Interim Guide, Equation 4.16 includes a significant safety factor which increases T by $0.774 \log W_{18}\left(\frac{2.5}{CBR}\right)^{0.4}$.

The design method allows the substitution of "limited portion" of the total thickness of asphalt concrete by an untreated granular base if conditions that call for consideration of such conversion exist. The equation is further modified to include the addition of a term for subgrade support value, expressed in terms of CBR.

A comparison of Equation 4.16 and the AASHO Road Test Equation 4.14 shows that Equation 4.16 gives considerably greater structural thickness (approximately 25 percent) for a relatively low 18-kip ESAL application (approximately 10^5), but the difference diminishes as the number of 18-kip ESAL applications increases. There is no difference in structural thicknesses given by the two methods at about 60 x 10^6 18-kip ESAL applications.

The safety factor included in Equation 4.16 was designed to envelop much of the data used in the analysis, and a study of Equation 4.16 shows that is envelops almost all the data points that reached a serviceability index of 2.5 during the life of the AASHO Road Test.

Since the Asphalt Institute design method is mainly based on the AASHO Road Test data, the same limitations that were discussed in the AASHO Interim Guide method will be applicable for this method. Therefore, this method also is not expected to provide maintenance-free pavements in all locations in the U. S.

Analytical Evaluation

1. Maintenance-Free Projects - Of the seventeen flexible pavement projects analyzed, seven were maintenance-free as shown in Table 4.11. A plot showing the existing and redesigned equivalent asphalt concrete thickness is shown in Figure 4.9. The "redesigns" of these 7 projects indicate the following:

Table 4.11. Summary of Redesign Using the Asphalt Institute Procedure for Flexible Pavements

Project No.-State		Equivalent Existing AC (in.)	Equivalent Existing AC (cm)	Maintenance-Free Performance		Equivalent Redesign AC (in.)	Equivalent Redesign AC (cm)	Does Redesign Provide Maint.-Free Performance?		Redesign as % of Existing AC
1	NH	11.5	29.21		No*	17.5	44.45	Q		152
2	NJ	11.9	30.23	Yes		6.7	17.02		No	56
3	MN	6.5	16.51		No	12.5	31.75		No	192
4	MN	11.8	29.97		No*	13.2	33.53		No	112
5	MN	11.9	30.23		No*	7.2	18.29		No	61
6	MN	6.1	15.49		No*	6.6	16.76		No	108
7	IL	13.0	33.02		No	16.5	41.91		No	127
8	IL	11.7	29.72	Yes		16.5	41.91	Yes		141
9	IL	13.2	33.53	Yes		16.5	41.91	Yes		125
10	IL	11.5	29.21		No	16.5	41.91		No	143
11	WA	12.5	31.75	Yes		8.2	20.83		No	66
12	WA	12.4	31.50	Yes		4.7	11.94		No	38
13	CA	9.5	24.13		No	11.0	27.94		No	116
14	CA	13.3	33.78	Yes		14.5	36.83	Yes		109
15	CA	13.5	34.29	Yes		8.5	21.59		No	63
16	UT	8.0	20.32		No	13.7	34.80	Q		171
17	AZ	5.1	12.95		No	17.0	43.18	Yes		333

* Filling cracks is the only maintenance received.

Q = Questionable

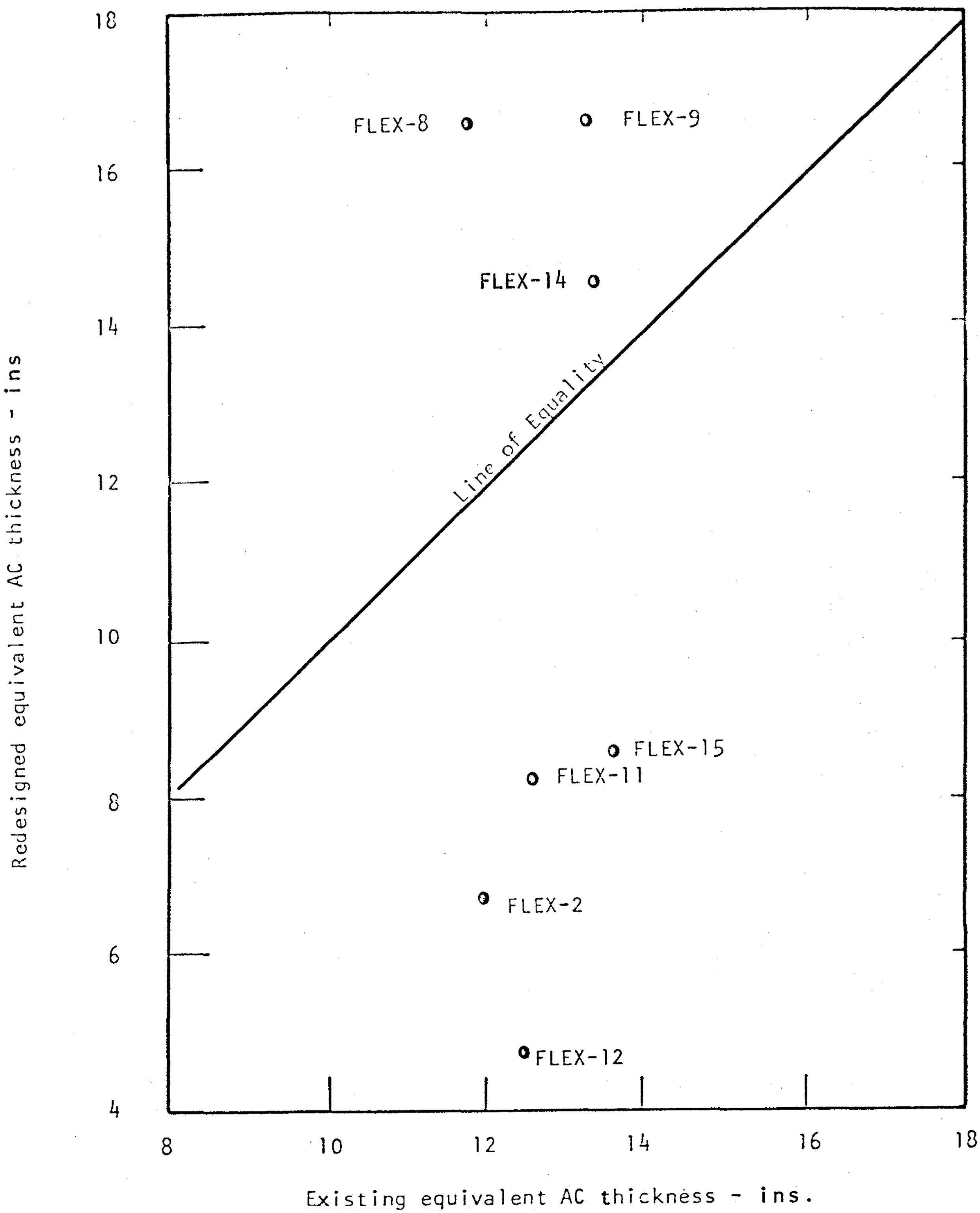


Fig. 4.9 Flexible pavement redesign using Asphalt Institute procedure versus existing equivalent Asphalt Concrete thickness (all projects shown provided maintenance-free performance).

a. Three of the seven zero-maintenance flexible projects redesigned by the Asphalt Institute method provided designs greater than or equal to the existing designs, hence the Asphalt Institute method would have provided zero-maintenance performance in these cases, provided the pavement composition remained relatively the same.

b. Four of the seven redesigned zero-maintenance flexible projects showed less structural sections than the existing. The Asphalt Institute design method provided an equivalent asphalt concrete thickness of 5 to 8 inches less than the existing thickness, and it is doubtful that these redesigns would give zero-maintenance performance.

2. Projects Requiring Maintenance - The ten projects that did not give zero-maintenance performance were also redesigned by the Asphalt Institute method. Nine of the redesigns gave thicknesses greater than the existing, the tenth provided a thickness less than the existing. It is believed that in a few of these cases, where load associated distress occurred, the Asphalt Institute method may have provided zero-maintenance performance. These projects required maintenance because of the damage caused by high loading stresses on inadequate pavement structures and/or due to environmental factors caused by low temperature cracking, etc.

Summary

The Asphalt Institute method of flexible pavement design has essentially all of the limitations of the AASHO Interim Guide. The added safety factor provided somewhat greater thicknesses, especially for the lower traffic volumes, as shown by the redesigns. Overall results indicate that the method usually does not provide maintenance-free pavements in various regions of the U. S. No guidance is given for considering environmentally caused distress, such as low-temperature cracking, which is a cause of distress in many areas of the U. S., and usually requires maintenance. Based upon projects included in the field study, the method's recommendation for using a relatively thick layer of asphalt bound aggregate appears desirable and will definitely help to reduce the traffic associated distress.

4.4.3 Resilience Design Procedure

Several states use a resilience procedure similar to the California DOT procedure (62) for the structural design of flexible pavements. This approach was originally developed by Hveem (62) in 1948, and is based mainly on the results of the Brighton Test Road project. The method has been modified several times on the basis of data from other road tests as well as in-service pavements, particularly in California.

Conceptual Evaluation

The basic design procedure is empirical since it was statistically derived from field test section performance data. The basis for establishing the depth of each pavement layer is to prevent excessive resilient deformation of the pavement layers and the permanent deformation in the subgrade. Both repeated traffic loadings and the strength characteristics of the pavement/subgrade materials as measured by the stabilometer are considered to accomplish this basic design objective. The basis of the design procedure is as follows:

1. A "Traffic Index" is used to express the total traffic. The Traffic Index is a function of the number of repetitions of 5000 lb equivalent wheel load during the design life of the pavement. Traffic data required for a design includes present and estimated future average daily truck traffic. Trucks are classified according to the number of axles, and equivalency factors are likewise assigned. An estimate of the annual number of equivalent 5000 lb wheel load repetitions (EWL) in the design lane expected during the 20-year period following construction is obtained using load equivalency factors. The traffic index is calculated by the following expression:

$$TI = 6.7 \frac{(EWL)^{0.119}}{10^6} \tag{4.17}$$

where

TI = traffic index, and

EWL = total equivalent 5000 lb wheel loads over the design period

2. The shear strength characteristics of the materials used is measured by means of a stabilometer and an expansion pressure test, and is expressed as "resistance value", R. The resistance value R is a coefficient representing the shearing resistance to plastic deformation of a saturated soil at a given density. The expansion pressure apparatus is used to establish the correct density under which the R value is determined. The design thickness and cover must be sufficient to protect the soil or pavement layer material from permanent deformation due to traffic loads as well as to prevent further expansion due to change in moisture content with the resulting loss of stability.

3. Gravel equivalencies are used among various materials types and thicknesses. Gravel equivalency is a function of the tensile property of a material and is relative to a unit thickness of gravel subbase. An equivalent pavement thickness, composed entirely of gravel, is calculated from the basic design equation as an initial step to determine the design thicknesses of the other layers in the section. The equation for equivalent gravel depth is as follows:

$$GE = 0.0032\ (TI)\ (100-R) \tag{4.18}$$

where

GE = gravel equivalent depth, ft

This gravel equivalent thickness is converted into actual thicknesses of surface, base, and subbase through equivalency factors and through calculating the required GE above each layer (starting with the surface) to prevent permanent deformation in each lower layer.

Based upon the above facts, the following analysis of limitations is summarized:

1. Safety Factor - A safety factor is apparently not included in this procedure. This is observed by a comparison of Equation 4.18 to the field test road data used to derive the equation (62).

2. Design Period - The amount of traffic and the time periods for which the procedure is originally developed are significantly less than the 20 to 40 year design period and corresponding traffic applications under consideration for zero-maintenance design. Hence, the use of the procedure for 20 to 40 years design period represents an extrapolation of unknown accuracy.

3. Climate - There is no consideration give to climate effect in Equation 4.18. The procedure was developed essentially for the mild California climate which prevails along much of the west coast, and no method has been suggested to adjust designs for different climates in other parts of the U. S. This is a significant limitation of this design procedure.

4. Variability - The procedure is based mainly on the results of road tests which were very carefully constructed under controlled conditions and consisted of relatively short sections. Typical highway projects contain much greater variability in materials and construction. Since no safety factor is included and material variations are not complately considered (they are considered in part by selecting a low design R value), localized distresses may occur due to the variations in construction and materials.

5. Design Criteria - The major design criteria is to prevent excessive deformation in each pavement layer. A major distress for flexible pavements is fatigue or alligator cracking, which is a result of the high tensile strains at the bottom of the asphalt concrete surface (or AC base). This type of distress is no considered directly in the design procedure.

6. Terminal Design criteria - The design procedure does not contain a definable limiting design criteria for terminal serviceability, such as the extent of cracking or roughness, as the intent is primarily to prevent fatigue cracking of the AC surface. Hence, the use of the method to design high traffic volume pavements for long periods of time may result in pavement structures requiring considerable maintenance before the end

of the design period.

Based upon these limitations, it is doubtful that the California DOT procedure will provide maintenance-free performance in all regions of the U.S.

Analytical Evaluation

1. Maintenance-Free Projects - Of the seventeen flexible pavement projects, seven were maintenance-free as shown in Table 4.12. A plot showing the existing and redesigned equivalent gravel thicknesses for these seven projects is shown in Figure 4.10. The redesign of these 7 projects indicated the following:

a. Two of the seven zero-maintenance flexible projects redesigned by the resilience method provided designs greater than the existing designs, hence, the resilience method would have provided maintenance-free performance in these cases.

b. Five of the seven redesigned zero-maintenance flexible projects showed less structural sections than the existing pavements. The resilience design method provided an equivalent thickness of gravel of 0.6-2 feet less than the existing thickness, and it is doubtful that the redesigns would give zero-maintenance performance.

2. Project Requiring Maintenance - The ten projects that did not give zero-maintenance performance were also redesigned by the resilience method. Two of the redesigns gave thicknesses greater than the existing, the other eight provided thicknesses less than the existing. Hence, it is believed the resilience method therefore would not consistently provide zero-maintenance performance for these cases.

Table 4.12. Summary of Redesigns Using the California DOT Procedure for Flexible Pavements.

FLEX Project No.	State	Equivalent Existing Gravel (ft)	Equivalent Existing Gravel (m)	Maintenance Free Performance	Equivalent Redesign Gravel (ft)	Equivalent Redesign Gravel (m)	Does Redesign Provide Maint. Free Perf.?	Redesign as % of Existing Gravel
1	NH	2.82	0.86	No	2.42	0.74	No	86
2	NJ	4.35	1.33	Yes	1.19	0.36	No	27
3	MN	2.40	0.73	No	1.83	0.56	No	76
4	MN	1.87	0.57	No	1.83	0.56	No	98
5	MN	3.14	0.96	No	1.10	0.34	No	35
6	MN	1.05	0.32	No	0.88	0.27	No	84
7	IL	3.44	1.05	No	2.15	0.66	No	63
8	IL	1.88	0.57	Yes	2.15	0.66	Yes	114
9	IL	2.08	0.63	Yes	2.15	0.66	Yes	103
10	IL	2.53	0.77	No	2.15	0.66	No	85
11	WA	2.985	0.91	Yes	1.55	0.47	No	52
12	WA	2.84	0.87	Yes	1.20	0.37	No	42
13	CA	2.20	0.67	No	2.20	0.67	No	100
14	CA	3.03	0.92	Yes	2.30	0.70	No	76
15	CA	2.83	0.86	Yes	2.05	0.62	No	72
16	UT	1.66	0.51	No	2.20	0.67	Q	133
17	AZ	1.92	0.59	No	2.00	0.61	Q	104

Q = Questionable

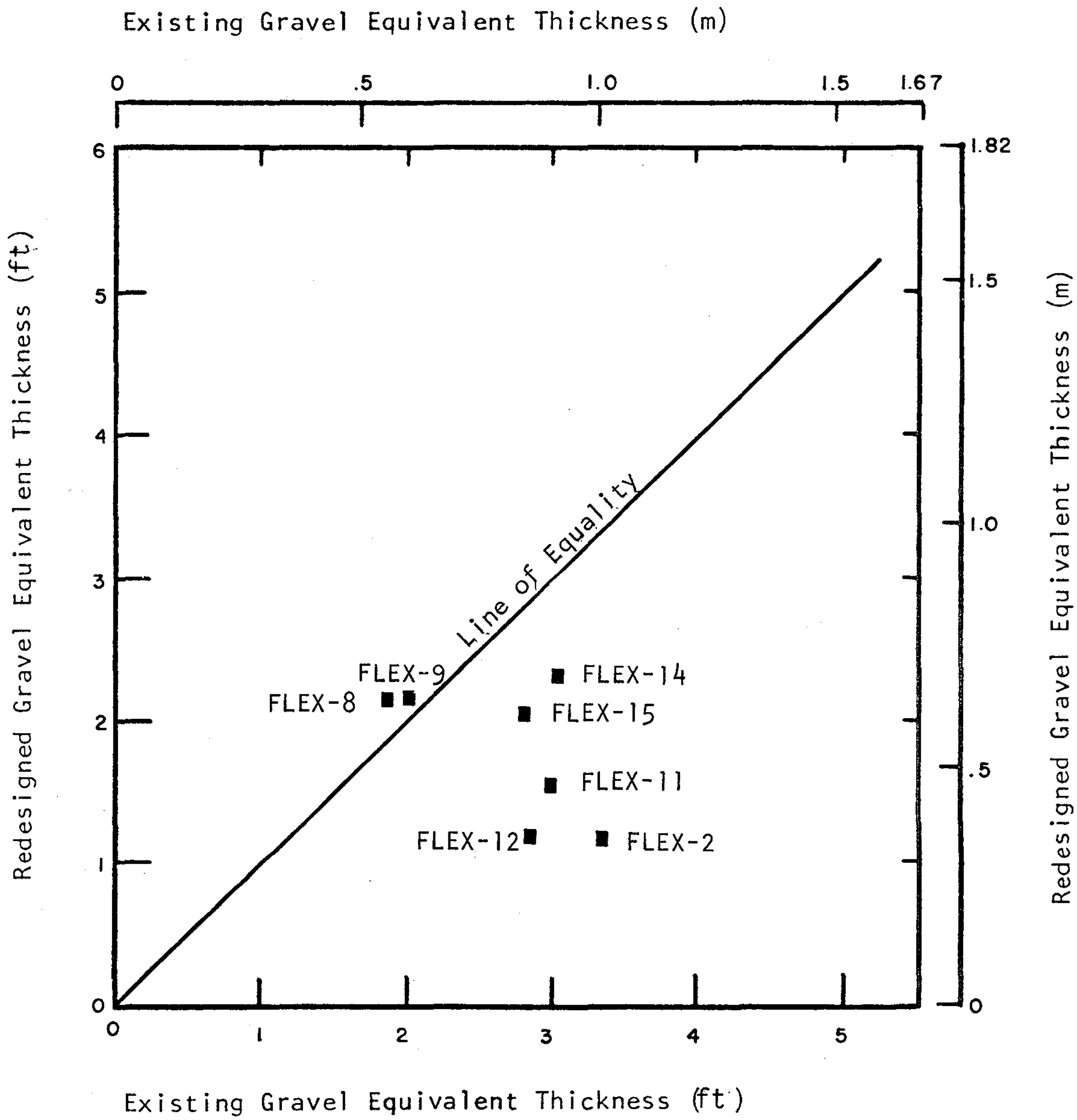


Fig. 4.10 Flexible Pavement Redesign Using A Resilient Procedure Versus Existing Gravel Equivalent (all projects shown provided maintenance-free performance.)

Summary

Results from both the conceptual evaluation and the analytical evaluation show that there are several significant limitations in the resilience design procedure analyzed. Hence, the method will not provide a structural system for reliable maintenance-free pavements under high volume traffic. The procedure provides for minimum stability values for the AC surface materials. Since rutting was not a major distress in the areas using this approach, it is assumed that the stability criteria are adequate.

4.5 COMPOSITE PAVEMENTS

There are no existing procedures for the design of composite pavements. There are however, procedures for designing asphalt concrete overlays over portland cement concrete pavements that can be used for this purpose. The AASHO Interim Guide for the design of flexible pavements is perhaps the most commonly used design procedure for asphalt overlay design, and will be briefly evaluated herein for its adequacy to design composite pavements.

4.5.1 AASHO Interim Guide

The design of asphalt concrete overlays using the AASHO Interim Guide is accomplished by determining the difference between the structural number required by a new design analysis and the structural number calculated for the existing pavement. Use of the AASHO Interim Guide to design composite pavements is similar to that for the design of asphalt concrete overlays. Layer coefficients are assigned to the subbase and the PCC slab, and the thickness of asphalt concrete is determined based on the required structural number. The procedure is quite similar to the design of flexible pavements, except that PCC is used for the base instead of the conventional base material.

Conceptual Evaluation

All limitations previously discussed for the use of the AASHO Interim Guide are applicable to composite pavement design. In addition, the following limitations which are fundamental in the entire concept are applicable:

1. Use of the structural number concept is not applicable since the specific makeup of a composite pavement will have significant effect on its performance (i.e., the use of a bond breaker minimizes the reflection cracking which is a major distress in composite pavements, but it cannot be accounted for in structural number computations.)

2. Field data are not available for determining the layer coefficient of the PCC slab in a composite pavement.

3. The AASHO Road Test equation for flexible pavements pertains to a specific configuration of layers, i.e., the strongest material at the top and the weakest at the bottom. A composite pavement is a considerable deviation from this configuration, the middle layer of concrete being the strongest and containing a high flexural resistance as compared to a minimal flexural strength for a flexible pavement, therefore the stress pattern in various layers will be considerably different. Hence, it is highly unlikely that the performance of a composite pavement will be successfully predicted by the use of a model which was primarily developed for flexible pavements.

Based on these and previously listed limitations, the AASHO Interim Guide is probably not valid for designing maintenance-free composite pavements.

Analytical Evaluation

1. Maintenance-Free Projects - Of the seven composite pavement projects analyzed, five were zero-maintenance as shown in Table 4.13. A plot showing the existing and redesigned SN values is shown in Figure 4.11 for these projects. The "redesigns" of these 5 projects indicate the following:

a. Two of the 5 zero-maintenance composite projects redesigned by the AASHO method provided structural numbers greater than of equal to values for the existing pavements, hence the AASHO procedure provides zero-maintenance design for these projects.

b. The remaining 3 designed zero-maintenance composite projects showed lesser structural sections than in the existing pavements. The AASHO design method provided structural numbers of 2.4 to 3.5 less than the existing SNs, and it is doubtful that these redesigned pavements would have given zero-maintenance performance.

2. Projects Requiring Maintenance - The two projects (COMP-2 and 3) that did not give zero-maintenance performance were also redesigned and gave structural numbers approximately equal to the existing. The redesigns therefore do not give zero-maintenance performance.

Summary

Both the conceptual analysis and the analytical redesign study indicate

Table 4.13 Summary of Composite Redesigns Using the AASHO Interim Guide

COMP Project No.-State		Existing SN	Maintenance Free Performance	Redesign SN	Does Redesign Provide Maintenance Free Performance?	Redesign As % Of Existing SN
1	MI	9.46	Yes	5.90	No	62
2	CAN	5.95	No	6.30	No	106
3	CAN	5.95	No	6.30	No	106
4	CAN	6.70	Yes	7.30	Yes	109
5	IL	5.18	Yes	5.30	Yes	102
6	NJ	6.52	Yes	4.25	No	68
7	WA	5.48	Yes	2.80	No	51

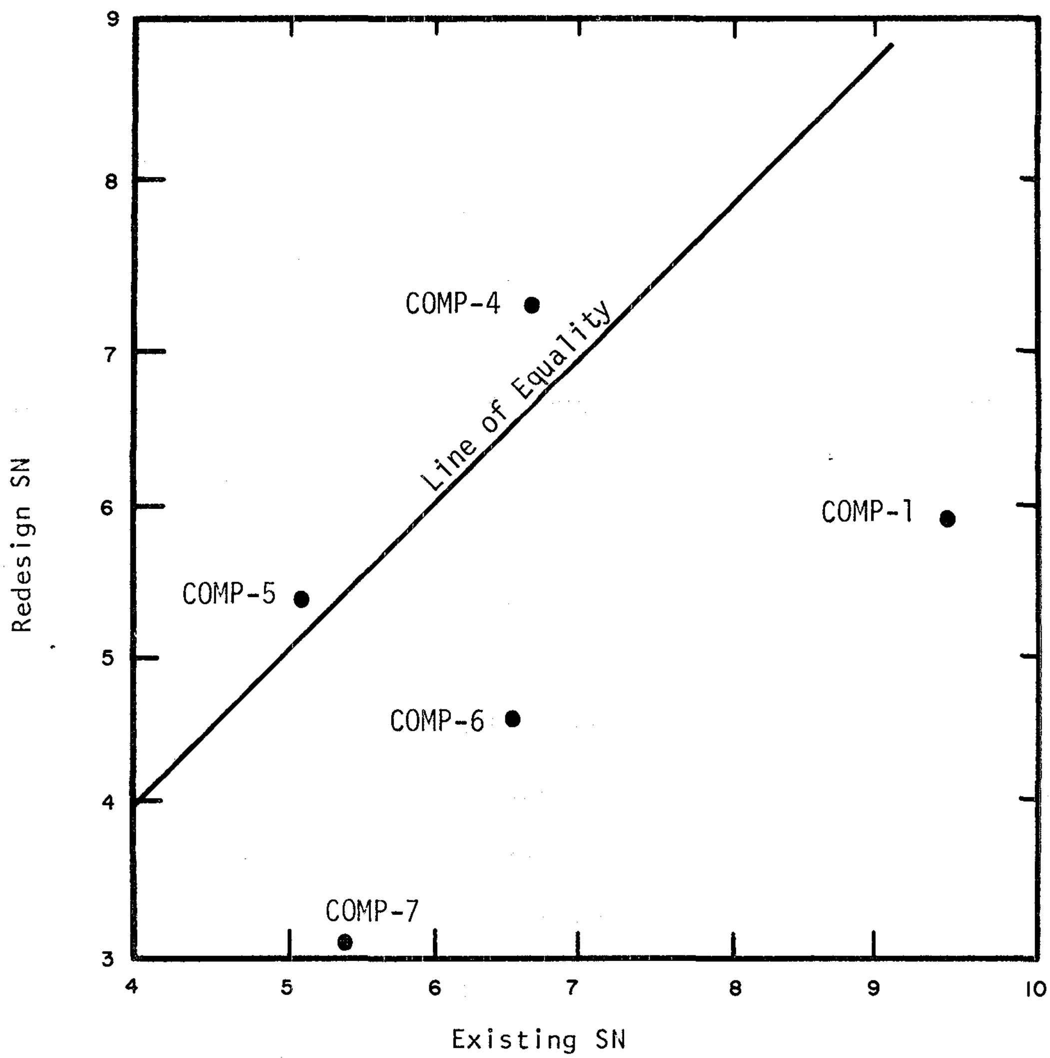


Figure 4.11. Composite Pavement Design SN Versus Existing SN Using AASHO Interim Guide (each project provided maintenance-free performance).

the AASHO Interim Guide methodology cannot be used to provide maintenance-free composite pavements. There are serious limitations to the use of the method. It is doubtful if the structural number has any applicability in the design and performance of composite pavements. Only two out of seven redesigns provided a structure that is adequate for maintenance-free performance.

4.6 SUMMARY OF EVALUATION RESULTS

A detailed evaluation of several design procedures for jointed, non-reinforced concrete, jointed reinforced concrete, continuously reinforced concrete, flexible, and composite pavements was conducted. The specific design procedures evaluated are listed in Table 4.1. The design procedures evaluated for jointed concrete pavements are based on either the results of the AASHO Road Test or the theoretical approach of the Portland Cement Association. The design procedures evaluated for continuously reinforced concrete pavements are based on the results of AASHO Road Test. The design procedures evaluated for flexible pavements are derived from the results of the AASHO Road Test with the exception of the resilience procedure. No design procedure developed specifically for composite pavements was found. A commonly used overlay design method was evaluated for applicability to composite pavement design.

The purpose of this chapter was to evaluate the adequacy of each design procedure in providing zero-maintenance pavements. The evaluation was conducted using two different approaches. First, each procedure was evaluated conceptually, and all theoretical deficiencies and limitations were determined. Second, each procedure was evaluated for actual design situations by "redesigning" several projects selected from the field study projects. The procedures were thus evaluated as to their ability to redesign pavement structures that have provided maintenance-free performance in specific geographical regions over long time periods.

Conceptual Evaluation Summary

Several theoretical limitations have been identified for each design method. Due to these limitations, the procedures may not be able to provide maintenance-free performance in many situations. The following represents a summary of the major limitations and deficiencies in various design procedures as they are currently used:

1. Lack of consideration of variability (in material properties, subgrade, construction, strength, fatigue life, etc.).

2. Lack of consideration for loss of foundation support (due to pumping, disintegration, spring thaw, densification, etc.).

3. Extrapolation of empirical design equations beyond the limits for which they were originally developed.

4. Inadequate joint design guidelines.

5. Inadequate reinforcement design procedures.

6. Little or no consideration of climatic factors (moisture, temperature, precipitation, etc.).

7. No recommendations for pavement subsurface drainage.

8. Extension of load equivalency factors beyond the limits and conditions for which they were originally developed.

9. Use of limiting criteria values (i.e., terminal serviceability index or percent fatigue consumed) in design that result in considerable maintenance before the end of the design period.

10. Lack of consideration of swelling subgrades.

11. Lack of procedures and adequate guidelines for shoulder design.

12. Lack of guidance on pavement structure composition (for a given overall structural requirement).

13. Lack of adequate durability requirements for pavement materials.

14. Lack of design flexibility due to fixed, or lack of, design safety factors.

15. Inadequate traffic loading analysis methods.

The ability of each design procedure to design against each significant distress type as identified in Chapter 3 can be evaluated based upon the results of the conceptual evaluation and the observations of in-service pavements. A summary of major distress types that may occur or have been observed to occur in pavements designed using each design procedure are as follows. In many instances, a procedure did not give any recommendations concerning a particular design aspect (i.e., joint spacing), and hence, unless other information was used, a design error could easily occur. This possibility is included in the distress types listed below:

1. JCP

a. AASHO Interim Guide: Transverse cracking, joint faulting, slab cracking from swelling subgrades, joint sealant damage, and "D" cracking.

b. PCA: Transverse cracking, joint faulting, slab cracking from swelling subgrade, joint sealant damage, and "D" cracking.

2. JRCP

a. AASHO Interim Guide: Transverse and longitudinal cracking, crack faulting, joint sealant damage, joint spalling, "D" cracking, slab cracking due to swelling subgrade, and blowups.

b. PCA: Transverse cracking, crack faulting, joint sealant damage, "D" cracking, and slab cracking due to swelling subgrade.

3. CRCP

a. AASHO Interim Guide: Crack spalling, surface depressions, interconnecting cracking and breakup, edge cracking, punch-outs, "D" cracking, roughness due to swelling subgrades, terminal joint and construction joint failure.

b. ACI: Crack spalling, surface depressions, interconnected cracking and breakup, edge cracking, punch-outs, "D" cracking, and roughness due to swelling subgrade.

4. FLEX

a. AASHO Interim Guide: Rutting, transverse cracking, fatigue (alligator) cracking, longitudinal cracking, surface depressions, reflection cracking, and roughness due to swelling subgrade.

b. Asphalt Institute: Rutting, transverse cracking, fatigue (alligator) cracking, longitudinal cracking, surface depressions, reflection cracking, and roughness due to swelling subgrade.

c. Resilience Procedures: Transverse cracking, fatigue (alligator) cracking, longitudinal cracking, surface depressions, reflection cracking, and roughness due to swelling subgrade.

5. COMP

a. AASHO Interim Guide: Rutting, transverse reflection and/or shrinkage cracking, longitudinal cracking, deterioration cracking, and edge cracking.

Analytical Evaluation Summary

Each design procedure was evaluated for actual design situations by "redesigning" several projects included in the field study. One of the important conclusions that may be reached from the analytical evaluation is the extent to which a design procedure "redesigns" a structure that provides maintenance-free performance over long time periods. Details of the results for each design procedure and each project have been provided in this chapter. A summary of the percentages of projects "redesigned" to provide an apparently adequate structure, to give maintenance-free performance for a certain period of time (equal to the in-service life of the project being redesigned in each case), is given in Table 4.14. These percentages have been calculated as follows: consider the evaluation of the AASHO Interim Guide for the design of JCP which showed (Table 4.2) 7 out of 10 zero-maintenance projects redesigned giving structures equal to or greater than the existing structures. All other factors being constant, the Guide would therefore have provided 70 percent of these projects with structures that have been proven to give maintenance-free performance for the time in service. It should be noted that this evaluation is considering the structural design of the pavement only, and not other aspects of the pavement performance. As another example,consider the evaluation of AASHO Interim Guide design procedure for FLEX pavement which showed (Table 4.11) 1 out of 9 zero-maintenance projects redesigned giving an overall structure greater than the existing structure. Therefore, the guide provided maintenance-free performance for only 11 percent of the projects.

Table 4.14 Summary of the Percentage of Projects Redesigned by Various Design Procedures, with Structures that have Provided Zero-Maintenance Performance

Design Methods	Percent of Zero-Maintenance Projects Redesigned as Zero-Maintenance Projects
1. JCP	
a. AASHO Interim Guides	70
b. PCA	33
c. State Application of AASHO Procedure	67
d. State Application of PCA Procedure	33
2. JRCP	
a. AASHO Interim Guides	12
b. PCA	0
c. State Application of AASHO Procedure	31
d. State Application of PCA Procedure	0
3. CRCP	
a. AASHO Interim Guides	0
b. ACI	0
4. FLEX	
a. AASHO Interim Guides	11
b. Asphalt Institute	43
c. Resilience Procedure	29
5. COMP	
a. AASHO Interim Guides	40

These results are only valid over time periods equal to the existing lives of the in-service projects surveyed. The time periods ranged from 10 to 25 years with an average of 15 years. Extension of these results to longer time periods of 20 to 40 years would probably show even less adequacy of the design procedure to provide maintenance-free designs.

The design of zero-maintenance pavements for 20 to 40 years under heavy traffic represents a significant extrapolation of the design procedures beyond the limits of their derivation. A considerable limitation relative to the validity and accuracy of this extrapolation is the inadequate consideration of long term traffic and time that is inherent in the design procedure. Most design procedures result in only 5 to 10 percent change in the structural section requirements for the facility with a 100 percent increase in traffic applications. This small effect by traffic on the structural design is the subject of much controversy among pavement engineers. Also, the design methods do not take into account the wide variation of time over which the traffic will be applied. Consider the design of a pavement for 40 years with the same traffic. In both cases, a commonly used design procedure will give the same pavement structure, provided all other factors are identical. This is unrealistic since time greatly affects the performance of a pavement because of the materials. Hence, the lack of consideration of time or design life in these design procedures leads to inadequate designs for zero-maintenance pavements.

Conclusions

The commonly used design procedures evaluated in the chapter are incapable of designing maintenance-free pavements under heavy traffic for periods of 20 to 40 years. Many limitations and deficiencies of these procedures have been documented, and the key factors which must be modified and/or added to the procedures to improve their ability to design maintenance-free pavements have been identified. The actual modifications of one or more of these procedures for zero-maintenance pavement design will be conducted during Phase II of this study.

CHAPTER 5

LIMITING DESIGN CRITERIA FOR USE IN DESIGNING ZERO-MAINTENANCE PAVEMENTS

5.1 INTRODUCTION

Limiting design criteria, such as maximum working stress or maximum deflection, are required in structural engineering to arrive at rational and adequate designs. The levels of such limiting criteria used in structural engineering are generally based on safety considerations or the consequences of failure. Since pavement failure occurs gradually over a relatively long period, and not as catastrophic occurrence, limiting criteria and their levels used in structural design of pavements involve, in addition to safety considerations, the ride quality of the pavement and its structural integrity.

The primary objective of this chapter is to determine appropriate limiting critieria levels that can be used to design zero-maintenance pavements subject to high traffic volumes. The analyses presented in this chapter show that currently used limiting criteria levels used for conventional pavement designs are not adequate for high traffic volume zero-maintenance pavement desings. The difference in the two limiting criteria is described as follows:

1. Limiting criteria for zero-maintenance: The limit is a pavement condition which requires structural maintenance due to any of several reasons, including safety, structural integrity or physical deterioration, and roughness. Structural maintenance consists of activities such as patching, crack filling, slab jacking, overlay, joint replacement, surface

grinding, etc.

2. Limiting criteria for major rehabilitation: The limit is a pavement condition where structural integrity is significantly impaired, roughness has become unacceptable, and therefore, excessive maintenance or a major rehabilitation has to be applied.

The limiting criteria actually used by maintenance personnel in determining when high traffic volume pavements should be maintained are based upon two factors:

1. User considerations.
2. Maintenance management policies.

The limiting criteria which provide direct evaluation of these factors, and are also amenable for use in zero-maintenance pavement design are:

1. Pavement serviceability.
2. Observable pavement distress.

These two criteria were identified as the most significant criteria by many of the state personnel interviewed during the field visits. This chapter presents a comprehensive analysis of these limiting criteria based upon extensive field surveys and interviews, analyses of data, and previous research results.

The limiting criteria currently being used in the structural design of pavements are first identified, the limiting serviceability criteria for zero-maintenance design is then determined, and finally, the limiting distress criteria for zero-maintenance design is developed.

5.2 IDENTIFICATION OF LIMITING CRITERIA USED IN DESIGN

Limiting criteria used in several currently used design methods are summarized in Table 5.1. Each of these limiting criteria is discussed, and values used for conventional pavement designs are presented in the following section.

Present Serviceability Index

Since the development of the serviceability-performance concept at the AASHO Road Test, the present serviceability index (PSI) has been widely used in several design procedures as a design parameter. Specific definitions used in the serviceability-performance concept are given by Carey and Irick (22). Basically, the present serviceability rating (PSR) is a subjective rating made by highway users of the ability of a pavement to serve traffic, and ranges from 0 to 5 as shown in Figure 5.1. The mean subjective evaluation of highway pavements as defined by the PSR was correlated with physical measurements taken on many in-service highways and AASHO Road Test pavements, and predictive equations were developed to estimate PSR. The estimated PSR value was termed present serviceability index (PSI) to distinguish it from the actual PSR. However, it must be remembered that PSI is nothing more than an estimate of PSR. The deterioration of each test section at the Road Test was measured in terms of roughness, cracking, rutting, patching, spalling, etc., and a corresponding PSI performance curve computed using the predictive equations. The basic data resulting from the AASHO Road Test have been used in developing several existing design procedures, and therefore, the PSI is widely used as a fundamental design limiting criteria.

TABLE 5.1. LIMITING CRITERIA USED IN CURRENT DESIGN METHODS

Design Methods	Limiting Criteria
(a) Jointed Rigid Pavements	
(1) AASHO Interim Guide	minimum acceptable serviceability index
(2) AASHO Interim Guide procedure applied by State DOT	" "
(3) Texas Rigid Pavement System	" "
(4) Portland Cement Association	percent fatigue consumption (125% maximum)
(5) PCA Procedure applied by State DOT	" "
(b) Continuously Reinforced Concrete Pavements	
(1) AASHO Interim Guide	min. acceptable serviceability index
(2) Texas Rigid Pavement System	" "
(3) American Concrete Institute	" "
(4) USS Steel Corporation	" "
(c) Flexible Pavements	
(1) AASHO Interim Guide	min. acceptable serviceability index
(2) Asphalt Institute	" "
(3) Texas Flexible Pavement Systems (FPS-11)	" "
(4) Resilient Procedure	Not defined
(d) Composite Pavements	
(1) AASHO Interim Guide (Overlay)	min. acceptable serviceability index

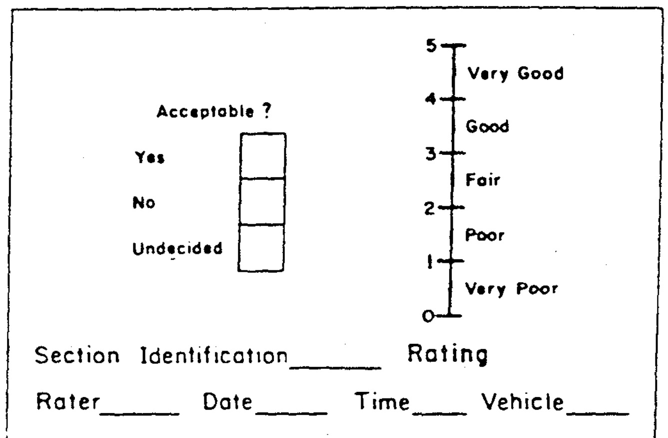


Figure 5.1. Present Serviceability Rating Form Used at the AASHO Road Test (Ref. 22).

The limiting or terminal values of PSI have been recommended in the AASHO Interim Guide for the design of different highways as follows:

2.5 for major highways,

2.0 for secondary highways.

These values have been adopted by almost every agency using the AASHO Interim Guide. The only known exception to these recommendations is the Texas Highway Department which uses a terminal PSI of 3.0 as a limiting criteria for the design of major highways.

Fatigue Resistance and/or Consumption

Another widely used design parameter is the fatigue consumption concept similar to that used by Portland Cement Association for the design of concrete pavements (105). The fatigue consumption concept assumes the following:

1. A fatigue failure occurs under continued repetitions of loads which may cause stress/strength ratios of less than unity. The stress/strength ratio is the ratio of maximum critical stress to the ultimate strength of a material. For example, the flexural stress is critical in a rigid pavement, hence the stress/strength ratio will be the ratio of the maximum flexural stress to the modulus of rupture of concrete.

2. As per Miner's hypothesis (87), the fatigue resistance not consumed by repetitions of one load is available for consumption by the repetitions of other loads.

The limiting value of total fatigue consumption has been recommended by the PCA to be 100 to 125 percent for concrete pavement design. This limit is also used by several other agencies using similar procedures, and in various theoretically based procedures for the design of flexible pavements.

Deflection

The past studies have shown that pavement deflection, and especially spring time deflection, is a good indicator and estimator of the loss of serviceability (57, 66). The AASHO Road Test data and the derived relationships indicate that pavement deflection may be used as a limiting design criteria.

The California D. O. T. developed a semi-empirical criteria for flexible pavements by setting the maximum deflection levels for various pavement layer combinations, as shown in Table 5.2. The values developed through a series of experimental tests, are applicable only to flexible pavements in California.

Other Limiting Criteria

It was found during the literature review that some empirical design procedures do not have identifiable limiting criteria and use "engineering judgment" as a limiting criteria.

5.3 LIMITING SERVICEABILITY CRITERIA FOR ZERO-MAINTENANCE PAVEMENTS

The acceptability of a pavement by the users (including automobile, bus, and truck occupants) is a major factor to be considered in determining the limiting criteria for zero-maintenance pavement design. Several aspects are discussed here, including minimum acceptable

TABLE 5.2 TENTATIVE MAXIMUM DESIGN DEFLECTIONS FOR PAVEMENTS (Ref. 155)

Thickness, In.	Type of Pavement	Max. Deflection for Design Purposes, In. (tentative)
8	Portland cement concrete	0.012
6	Cement-treated base (surface with bituminous pavement)	0.012
4	Asphalt concrete (plant mixed) on untreated aggregate base	0.017
3	" "	0.020
2	" "	0.025
1	Asphalt concrete (road mixed) on untreated aggregate base	0.036
1/2	Surface treatment	0.050

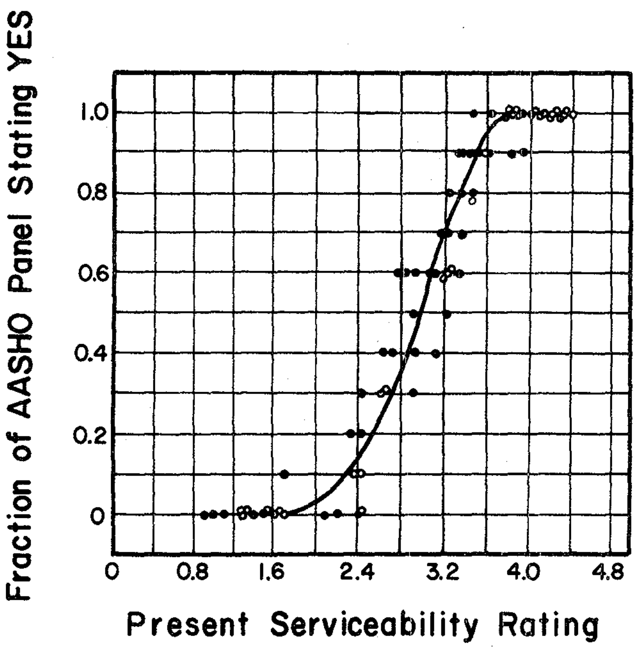
serviceability, limiting serviceability/maintenance criteria, and limiting serviceability/user cost criteria.

Minimum Acceptable Serviceability

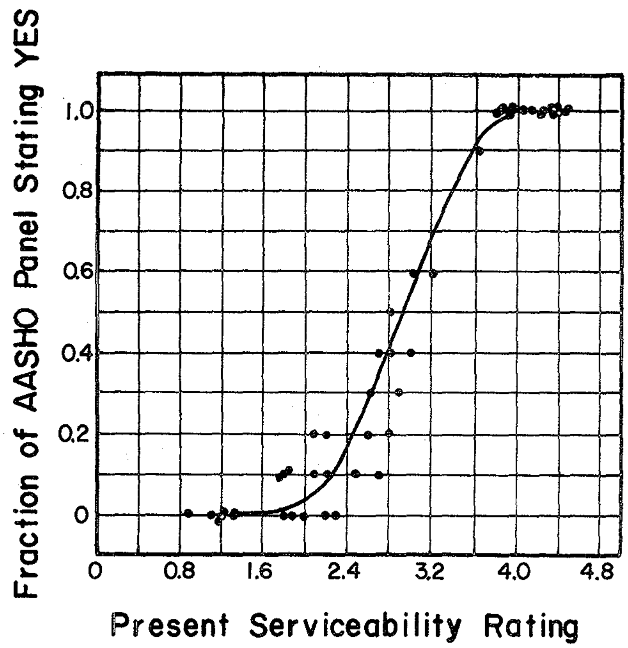
A pavement's "acceptability" is subjectively judged by a highway user based on comfort, safety, and extent of possible damage to the user's vehicle. Results from three studies are available which help to assess minimum acceptability levels.

1. The most extensive study was conducted during the AASHO Road Test by Carey and Irick (22) where 74 flexible and 49 rigid pavements in three states were rated by a panel of highway users. Results showing the fraction of the panel rating the pavements as acceptable and unacceptable are shown in Figure 5.2. These pavements ranged from secondary to primary highways, and no differentiation was made between highway types in determining the acceptability. Figure 5.2 gives the 50th percentile for acceptability of 2.9 and the 50th percentile for unacceptability of 2.5. The difference between the two values is due to several panel members being undecided.

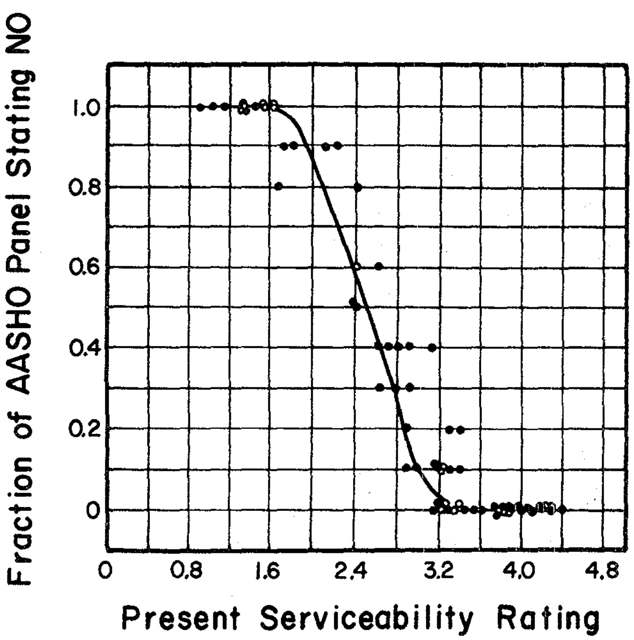
2. A similar study was conducted by the Texas Highway Department in 1968 (106). The acceptability levels were determined separately for interstate and secondary highway pavements. Results showing the percent of the panel rating the pavement as acceptable are shown in Figure 5.3 for interstate and in Figure 5.4 for secondary highways. The general trend of the data is similar to the AASHO study, but the 50th percentile levels for acceptability are significantly different, being 3.4 for interstate and 1.9 for secondary highways.



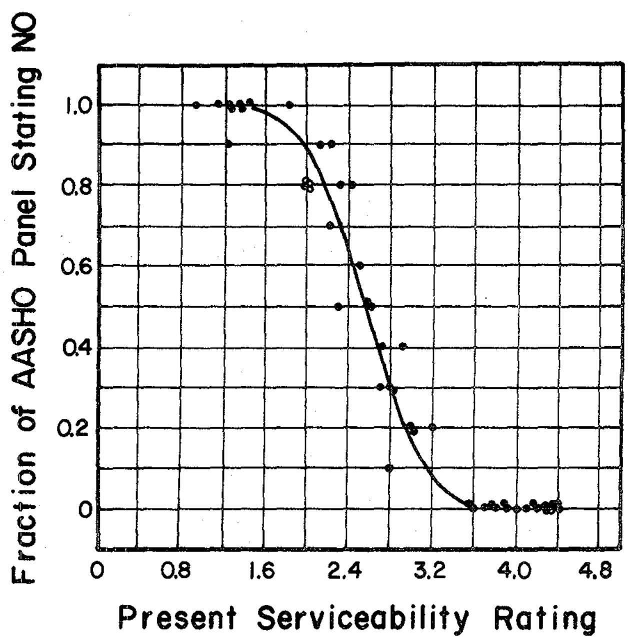
(a) Acceptability vs present service-ability rating: 74 flexible pavements.



(b) Acceptability vs present service-ability rating: 49 rigid pavements.



(c) Unacceptability vs present service-ability rating: 74 flexible pavements.



(d) Unacceptability vs present serviceability rating: 49 rigid pavements.

Figure 5.2. Acceptability and Unacceptability Versus Serviceability for Flexible and Rigid Pavements (Ref. 22).

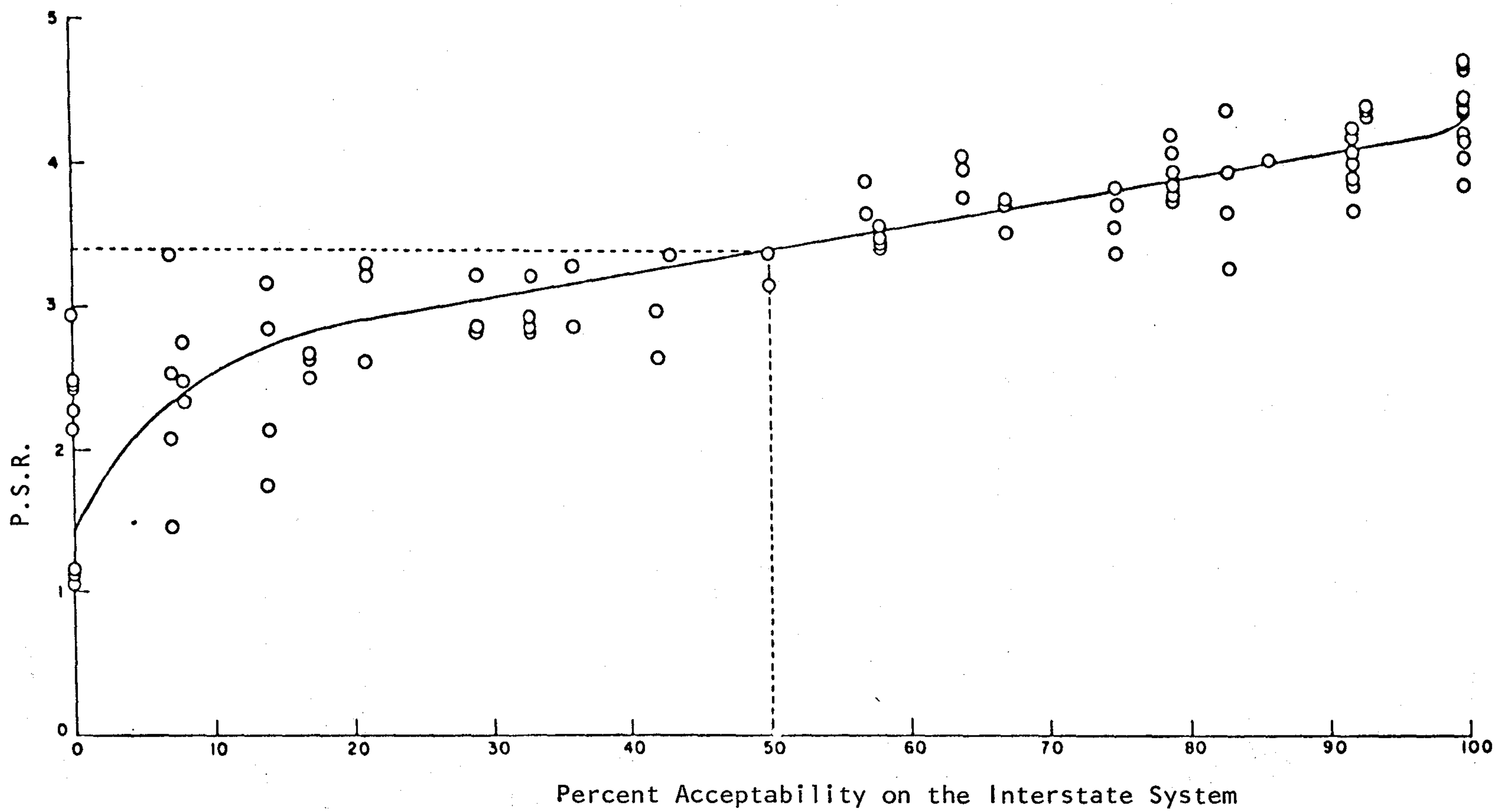


Fig. 5.3 Acceptability versus PSR for Interstate Highways (Ref. 106)

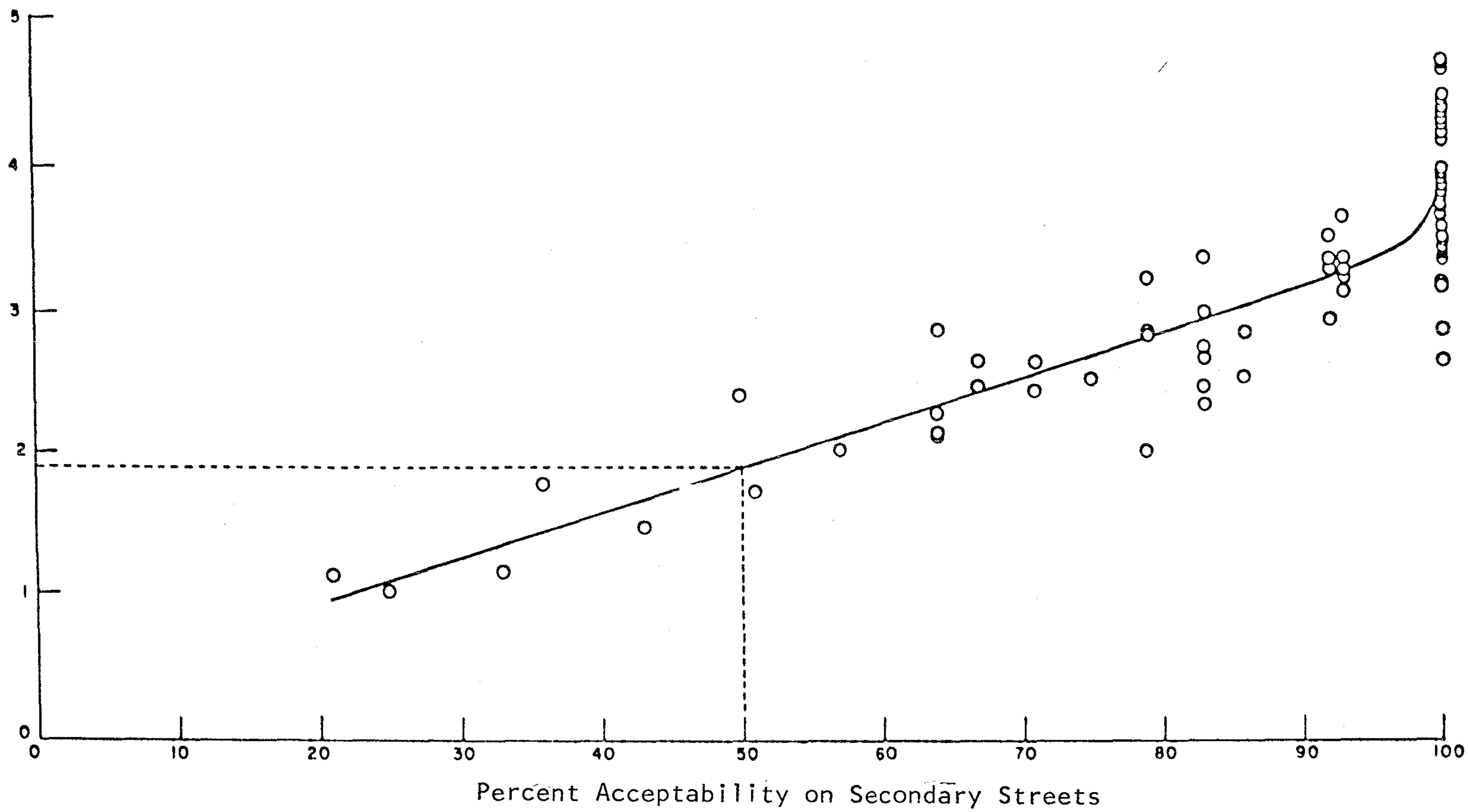


Fig. 5.4. Acceptability versus PSR for SEcondary Highways (Ref. 106).

3. Another study was conducted by Rogers, Cashell, and Irick (108) during 1961-62 to determine the serviceability index.

> "Its purpose was to establish a serviceability index for pavements with surfaces no longer considered acceptable to to traffic. This index, called the terminal serviceability index, was needed to make the AASHO Road Test equations usable in predicting the life of new pavements and the remaining life of pavements in-service - Nationwide, the average terminal serviceability index was determined to be 2.2 for primary rigid pavements, 2.1 for primary flexible pavements, and 1.9 for secondary flexible pavements. Secondary rigid pavements were not included in the survey because of the scattered locations of the relatively few such pavements scheduled for resurfacing."

The values arrived at in the above studies are the limiting levels when major rehabilitation should be provided and not the limiting values for maintenance-free pavements as will be shown later. As per AASHO Interim Guide,

> "Selection of the terminal serviceability index is based on the lowest index that will be tolerated before resurfacing or reconstruction becomes necessary." (1)

The state highway personnel interviewed during the field survey indicated a minimum acceptable serviceability index of 2.7 to 3.0 for high traffic volume pavements before major rehabilitation should be performed. A particular case that supports these recommendations is the rehabilitation of the local lanes of the Dan Ryan Expressway in Chicago, Illinois, in 1971. The expressway carries about 250,000 ADT in 12 lanes. The pavement exhibited considerable distress and received extensive maintenance before major rehabilitation took place in 1971. Justification of the rehabilitation was that there were significant public complaints about the condition of the pavement, even though the average present serviceability index at the time of rehabilitation was 2.7.

It appears that despite the findings of Rogers, Cashell, and Irick, and the recommendations of the AASHO Interim Guide, the present terminal serviceability index for rehabilitation of major freeways is in the range of approximately 2.7 to 3.0 or greater. Based upon the results of the Texas study where the 50th percentile PSR for acceptance on interstate highways was 3.4, the major rehabilitation at serviceability levels of 2.7 or greater is not surprising. Based upon the available results, it is concluded that if the serviceability level of a high traffic volume pavement, equivalent to the interstate system, is permitted to drop below 3.0, many users will not subjectively rate the pavement as "acceptable" and considerable public complaints will be received if the pavement serviceability drops below this level.

Minimum Serviceability for Zero-Maintenance Design (Field Visits)

The 68 high traffic volume projects surveyed during the field visits provide valuable information to determine an approximate minimum serviceability level to which a pavement may be allowed to deteriorate before structural maintenance is applied. The definition of "zero-maintenance" is somewhat different for this section in that a pavement is considered zero-maintenance even though some joint sealing (but not crack filling) and shoulder maintenance may have taken place. Also, a few projects that had received very minor maintenance work (e.g., one small patch) are still considered zero-maintenance.

The present serviceability indices of the projects were estimated by the project staff during the field visits, and values were also obtained from agency personnel where possible. Also, each project was classified as maintenance-free or maintenance-required as discussed and data was obtained indicating the proportions of projects at a given serviceability value that were maintenance-free. The relationship between pavement serviceability and the proportion of projects with maintenance-free performance is shown in Figure 5.5 through Figure 5.9 for different pavement types. The plots show an increase in the proportion of maintenance-free pavements with an increase in serviceability level. A plot showing a combination of data for all pavement types is shown in Figure 5.10. The number of projects contained within each data point is also shown in this figure.

The following serviceability levels are obtained from these plots to cover 50 percent of the projects (probability of 50 percent) for each pavement type:

JCP	3.2
JRCP	3.2
CRCP	3.6
FLEX	3.5
COMP	3.4
ALL TYPES	3.4

For serviceability levels greater than about 3.8, 100 percent of the projects exhibited zero-maintenance performance whereas for serviceability levels below 3.0, no project exhibited zero-maintenance performance.

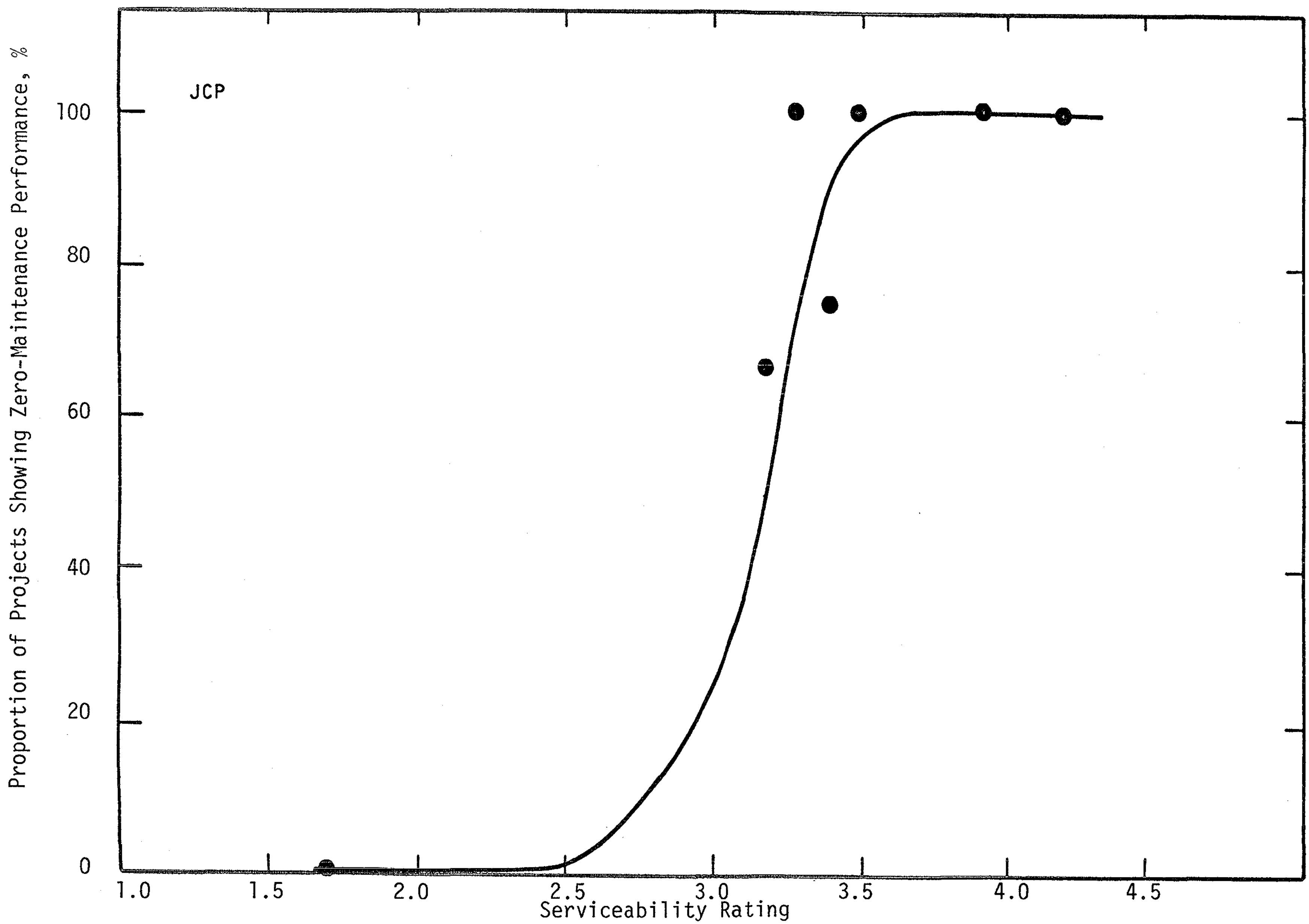


Figure 5.5. Subjective rating and maintenance needs for non-reinforced jointed concrete pavements.

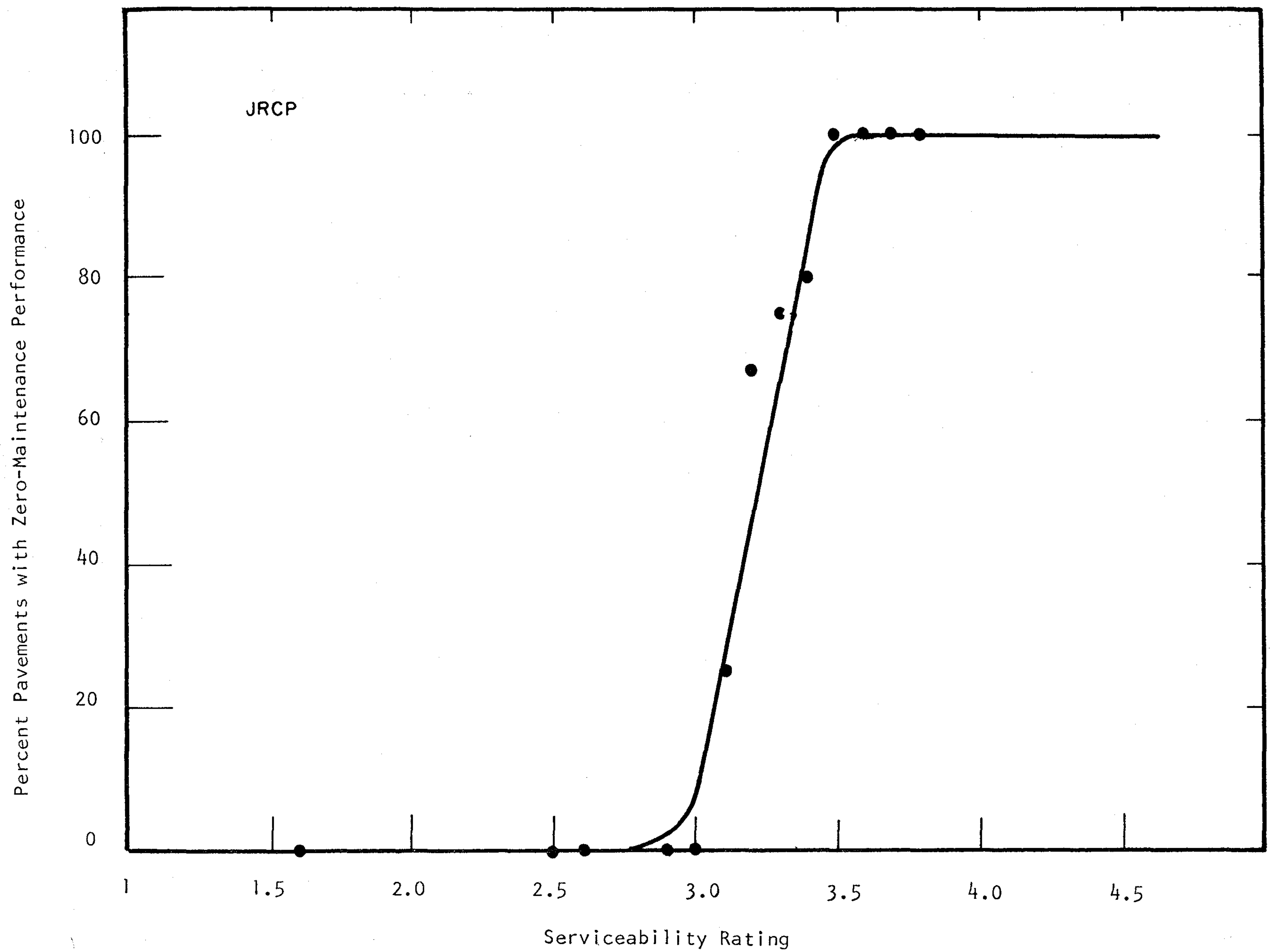


Fig. 5.6 Subjective rating and maintenance needs for jointed reinforced concrete pavements.

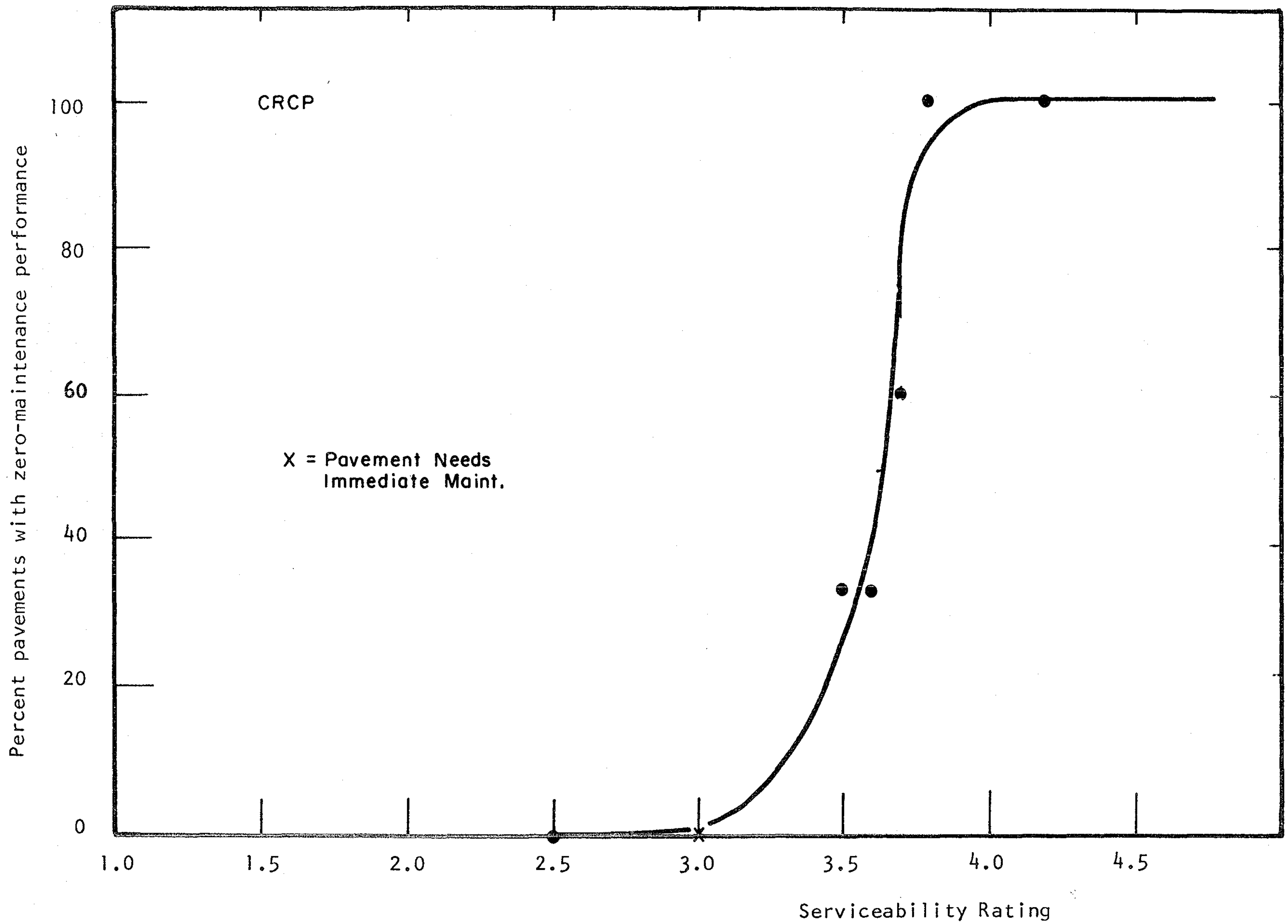


Fig. 5.7. Subjective rating and maintenance needs for continuous reinforced concrete pavements.

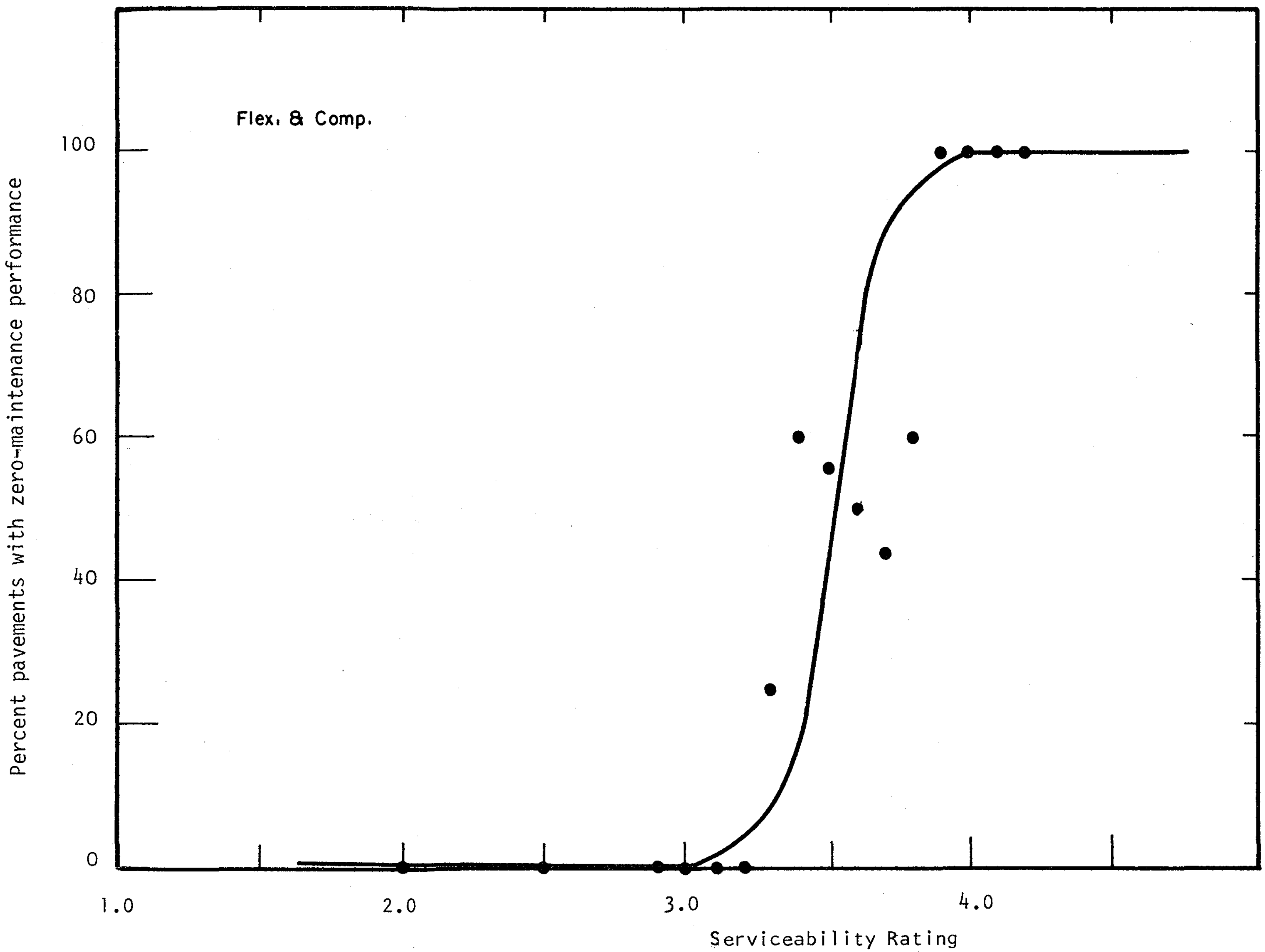


Fig. 5.8 Subjective rating and maintenance needs for flexible and composite pavements.

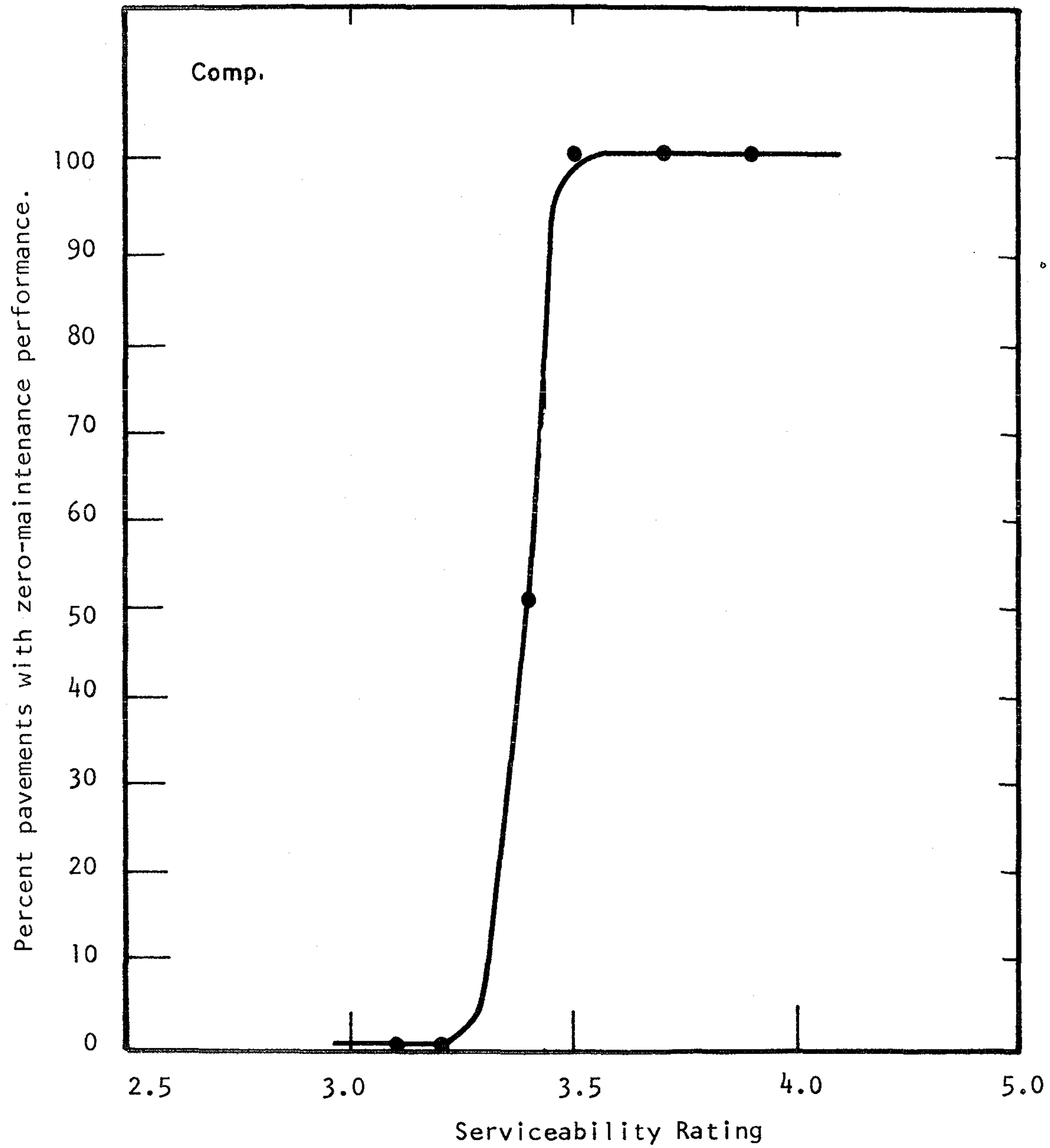


Fig. 5.9. Subjective rating and maintenance needs for composite pavements.

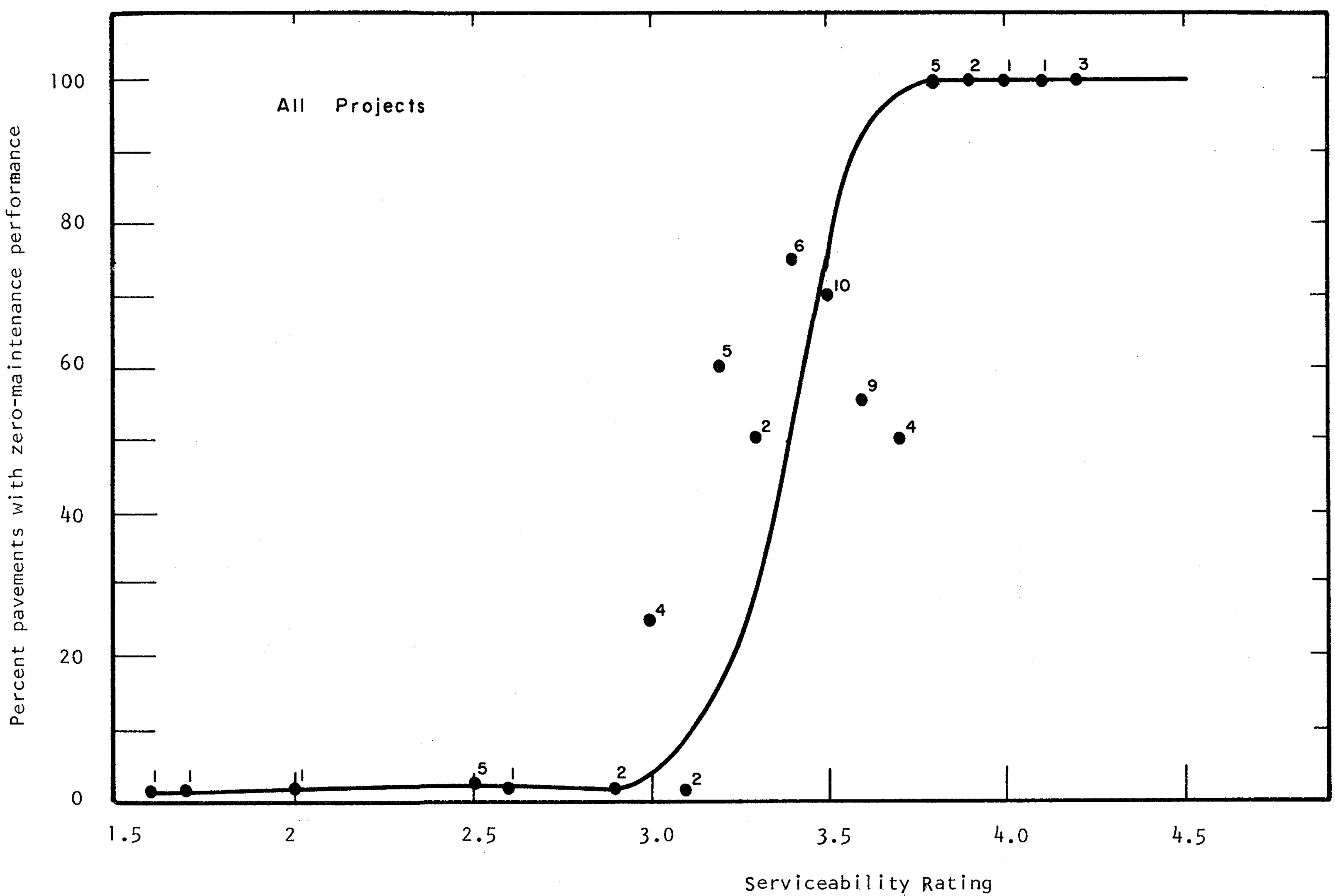


Fig. 5.10. Subjective rating and maintenance needs for all types of pavements.

The results indicate that the level of serviceability (PSR or PSI) can be used as a reasonable indicator of when maintenance is usually applied. There are cases when this will not be valid, such as a severe localized "pothole" on a new project initiated from a construction deficiency and then deteriorating rapidly with traffic. However, these data provide substantial information to help select minimum serviceability levels for design of zero-maintenance pavement systems.

Minimum Serviceability for Zero-Maintenance Design (User Cost)

Another effect of terminal serviceability level on the highway user can be determined in terms of "costs" to the user. This section presents results of an analysis of user costs/serviceability conducted by Dr. R. K. Kher of the Ontario Ministry of Transportation and Communications.

User costs to the public occur in many categories of which two are the most important: vehicle operation and travel time. The vehicle operating cost includes fuel consumption, oil consumption, tire wear, vehicle repair and maintenance, and vehicle depreciation. Travel time cost includes the value of user time for both work and non-work trips. A detailed analysis of user costs is given in this section with applications to zero-maintenance design examples.

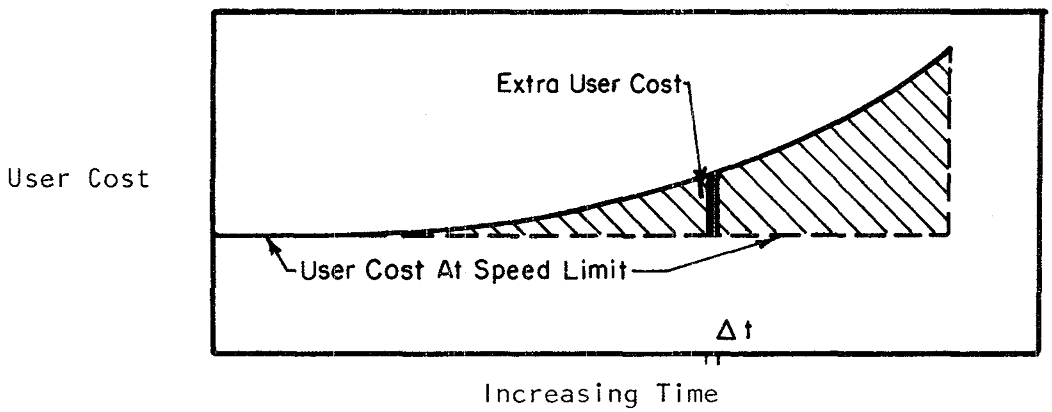
Rationale for User Cost

As a pavement becomes rough, the operating speeds of the vehicles are normally reduced. The effect on user cost is as follows:

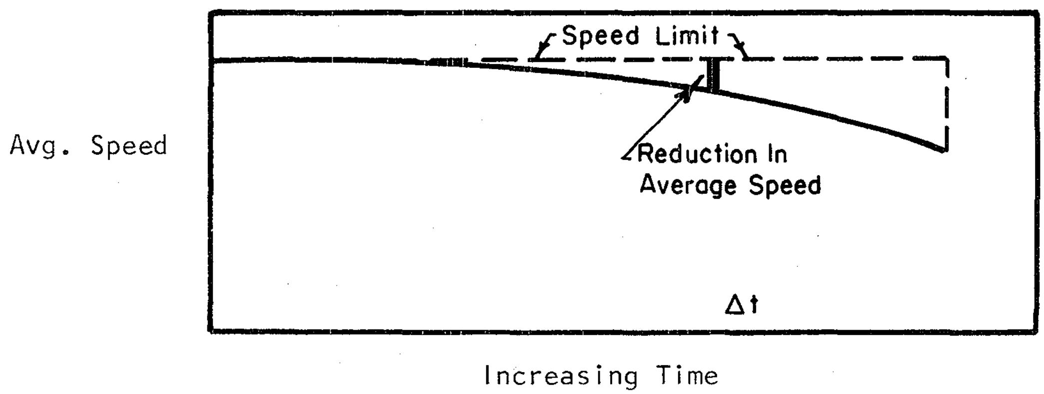
1. Increased roughness increases vehicle operating costs;
2. Reduced speed increases user time costs; and
3. Reduced speed reduces vehicle operating costs.

In general, the overall effect of increased roughness is an increase in user cost. The effects due to the above three items are of different magnitudes, and the overall increase depends on how the user adjusts his speed to accomodate pavement roughness. For example, if users reduce their speeds significantly to adjust for the increased roughness (generally the case on low volume secondary roads), item 3 cancels out most of the increase due to items 1 and 2, and therefore a low overall increase in user cost is observed. In cases where users do not reduce their speeds to compensate for increased roughness (generally the case on expressways serving metropolitan areas), items 2 and 3 do not exist, and a high overall increase in user cost is observed.

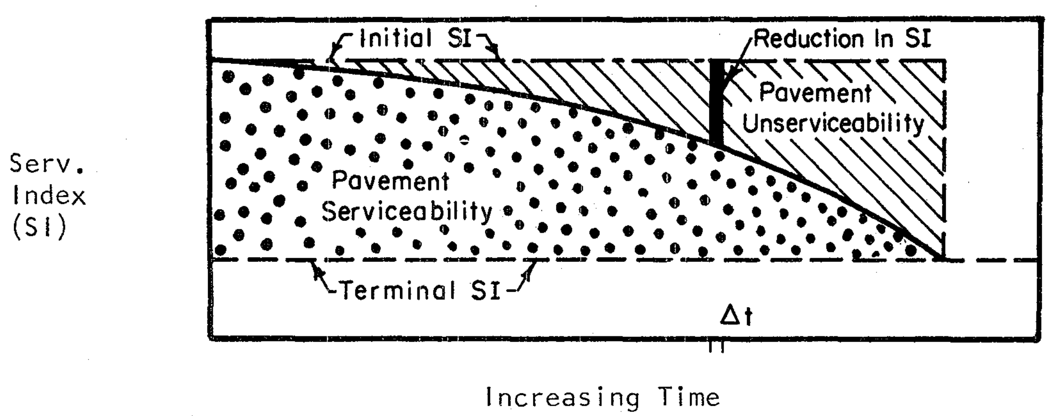
Figure 5.11 gives an illustration of roughness, speed, and user cost profiles for a pavement strategy. Since pavement design should consider only those user costs which may exist as a consequence of pavement deterioration, only that part of user cost which is attributed to pavement roughness is important. This part of user cost is the excess, over that which would occur assuming the pavement stays at initial serviceability level. This part is called "Extra User Cost", and is obtained by subtracting, from total user cost at any point in time, that part which is incurred due to the initial serviceability index level. If a pavement stays at the initial SI level at all times, and if users can maintain their average operating speeds at speed limit, no user cost will be attributed to pavement design, and extra user cost will be zero.



(a) User Cost Profile



(b) Speed Profile



(c) Performance Profile

Figure 5.11 Illustrative performance, speed and user cost profiles for a pavement strategy.

Vehicle Operating Cost

Vehicle operating costs include,

1. Fuel
2. Oil
3. Tire wear
4. Repair and maintenance
5. Depreciation

Results from numerous research studies (4, 10, 18, 26, 35, 152) document vehicle operating cost. Data from all available sources have been studied, and specific rates of fuel consumption, oil consumption, tire wear, repair, maintenance, and depreciation as a function of speed have been selected for road user cost analysis (97). An example of fuel consumption rates for automobiles, single unit trucks (two-axle, six-tire trucks), and transport trucks (tractor semi-trailer truck combinations) used in the present analysis is given in Table 5.3. Data represents the consumption rates on straight tangents of highways in rolling terrain, and was taken from an NCHRP study report (26).

Consumption rates are multiplied by respective unit costs to obtain each component of vehicle operating cost. Unit costs used for the present analysis are shown in Table 5.4. These unit costs represent actual unit costs minus taxes which are not considered since the present study deals with public investment decision making.

The sum of fuel, oil, tires, maintenance, and depreciation costs gives the total vehicle operating costs, as a function of speed, for automobiles, single unit trucks, and transport trucks. Table 5.5 also

SPEED (MPH)	FUEL CONSUMPTION RATE (GALLONS/1000 MILES)		
	AUTOMOBILES	SINGLE UNIT TRUCKS	TRANSPORT TRUCKS
10	68.0	98.0	336.0
20	49.0	87.0	262.0
30	42.0	92.0	263.5
40	44.0	107.0	312.5
50	50.5	126.0	397.5
60	60.5	146.5	542.5
70	68.0	N/A	N/A

Table 5.3. Fuel Consumption Rates as Affected by Speed on Straight Tangents of High Type Pavements in Rolling Terrain.

COST COMPONENT	UNIT	COST PER UNIT, $		
		AUTOMOBILES	SINGLE UNIT TRUCKS	TRANSPORT TRUCKS
Fuel	gallon	0.50 (gasoline)	0.50 (gasoline)	0.47 (Diesel)
Oil	quart	1.30	1.30	1.30
Tire	tire	40.00	160.00 (includes one retread)	313.00 (includes 2-1/2 retreads)
Vehicle Depreciable Value	vehicle	4140.00 (excluding tires)	5100.00 (excluding tires)	28,000.00 (excluding tires)
Labour for Repair & Maintenance	hour	11.60	11.60	11.60

Table 5.4. Unit Costs for Vehicle Operating Components.

gives a composite vehicle operating cost based on a traffic distribution of 85 percent automobiles, 5 percent single unit trucks, and 10 percent transport trucks.

Based on studies by Kher (71) and McFarland (83), effects of roughness of vehicle operating costs are shown in Table 5.6. Composite values of vehicle operating cost as shown in Table 5.5 are multiplied by a roughness effect matrix (71) to obtain Table 5.6 which gives vehicle operating cost as a function of speed and pavement roughness.

Time Cost

An important part of user cost is the value of time users pay due to a reduction in average operating speed. Numerous studies (53, 75, 90, 131) have been carried out to evaluate the dollar value of time savings, and the following two concepts are generally used (53) to determine the value of time:

1. Willingness to pay.
2. Trade-off of additional dollar costs against savings in travel time.

In the present study, the willingness to pay concept has been adopted. As a result, value of travel time is measured as a function of the wage rate of the travellers.

Table 5.7 gives unit values of travel time used for automobiles, single unit trucks, and transport trucks. Table 5.7 also gives the time cost as a function of speed and a composite time cost based on a traffic distribution of 85 percent automobiles, 5 percent single unit trucks, and 10 percent transport trucks.

SPEED	COST (CENTS/MILE)			
	AUTOMOBILES	S. UNIT TRUCKS	TRANSPORT TRUCKS	COMPOSITE VEHICLE*
10.0	8.28	14.63	32.78	11.06
15.0	7.59	13.71	29.95	10.13
20.0	7.08	13.40	28.70	9.55
25.0	6.85	13.71	28.73	9.38
30.0	6.80	14.37	29.79	9.48
35.0	6.92	15.34	31.75	9.83
40.0	7.40	17.20	35.84	10.73
45.0	7.93	19.14	39.79	11.68
50.0	8.49	21.04	42.72	12.54
55.0	9.41	23.82	47.69	13.94
60.0	10.35	26.37	51.67	15.28
65.0	11.70	29.72	56.40	17.07
70.0	12.92	32.71	60.53	18.67

*Composite vehicle represents a traffic distribution of 85% automobiles + 5% S. Unit trucks + 10% trans. trucks.

Table 5.5. Vehicle Speed Versus Operating Cost.

SPEED (MPH)	COST (CENTS/MILE) FOR SI OF						
	1.5	2.0	2.5	3.0	3.5	4.0	4.5
10.0	11.21	11.21	11.04	11.04	11.04	11.04	11.04
15.0	10.54	10.44	10.23	10.18	10.13	10.13	10.13
20.0	10.32	10.08	9.75	9.65	9.55	9.55	9.55
25.0	10.55	10.13	9.66	9.52	9.38	9.36	9.38
30.0	11.19	10.52	9.96	9.67	9.48	9.43	9.39
35.0	12.28	11.25	10.51	10.07	9.83	9.73	9.73
40.0	14.22	12.66	11.79	11.05	10.73	10.62	10.57
45.0	16.46	14.25	12.90	12.14	11.68	11.56	11.44
50.0	18.81	15.80	14.11	13.18	12.54	12.41	12.22
55.0	22.16	18.26	16.03	14.70	13.94	13.67	13.50
60.0	25.82	20.78	17.95	16.23	15.28	14.84	14.67
65.0	30.64	24.16	20.49	18.30	17.07	16.47	16.05
70.0	35.48	27.45	22.78	20.17	18.67	17.83	17.37

Table 5.6. Composite Vehicle Operating Cost as a Function of Vehicle Speed and Pavement Roughness.

Unlike vehicle operating cost, travel time cost is not a function of roughness, i.e., at a uniform average speed, travel time cost will not change at varying levels of roughness.

Total User (Vehicle Operating and Time) Cost

Adding composite time cost values of Table 5.7 to vehicle operating cost values of Table 5.6, final total user cost values are arrived in Table 5.8 as function of vehicle speed and pavement roughness.

Extra User Cost

As described in the rationale for user cost, only a part of total user cost which occurs due to the inability of the design to retain initial serviceability index (and thereby allow the vehicles to travel at the speed limit) is significant in pavement analysis. This part, called "Extra User Cost", is obtained by subtracting from total user cost matrix (Table 5.8), the user cost associated with the initial pavement conditions.

Two examples of extra user cost are given in Table 5.9, first for initial serviceability index of 4.5 and speed limit of 70 mph (corresponding cost = 23.64 c/mile), and second for initial serviceability index of 4.5 and speed limit of 55 mph (corresponding cost = 21.48 c/mile). Extra user cost for any roughness and average operating speed level can be obtained (for the above two initial conditions) from Table 5.9a and 5.9b, respectively.

Application to Zero-Maintenance Pavements

Four performance histories which result in terminal serviceability indices of 2.0, 2.5, 3.0, and 3.5 at 20 years have been selected for

SPEED (MPH)	COST (CENTS/MILE)			
	AUTOMOBILES	S. UNIT	TRANSPORT	COMPOSITE VEHICLE
10.0	40.00	66.00	66.00	43.90
15.0	26.67	44.00	44.00	29.27
20.0	20.00	33.00	33.00	21.95
25.0	16.00	26.40	26.40	17.56
30.0	13.33	22.00	22.00	14.63
35.0	11.43	18.86	18.86	12.54
40.0	10.00	16.50	16.50	10.97
45.0	8.89	14.67	14.67	9.76
50.0	8.00	13.20	13.20	8.78
55.0	7.27	12.00	12.00	7.98
60.0	6.67	11.00	11.00	7.32
65.0	6.15	10.15	10.15	6.75
70.0	5.71	9.43	9.43	6.27

Rates: Automobile $1.90-$4.00/hr; S Unit and Transport Trucks $6.60/hr.

Table 5.7. Travel Time Cost As A Function of Speed

SPEED (MPH)	COST (CENTS/MILE) FOR SI OF						
	1.5	2.0	2.5	3.0	3.5	4.0	4.5
10.0	55.11*	55.11	54.94	54.94	54.94	54.94	54.94
15.0	39.81	39.70	39.50	39.45	39.40	39.40	39.40
20.0	32.27	32.03	31.70	31.60	31.50	31.50	31.50
25.0	28.11	27.69	27.22	27.08	26.94	26.92	26.94
30.0	25.82	25.16	24.59	24.30	24.11	24.07	24.02
35.0	24.82	23.79	23.06	22.61	22.37	22.27	22.27
40.0	25.19	23.64	22.67	22.03	21.71	21.60	21.55
45.0	26.22	24.00	22.66	21.90	21.43	21.32	21.20
50.0	27.59	24.58	22.89	21.88	21.32	21.19	21.00
55.0	30.14	26.24	24.01	22.68	21.92	21.65	21.48
60.0	33.14	28.10	25.27	23.54	22.60	22.15	21.99
65.0	37.40	30.91	27.24	25.06	23.83	23.23	22.80
70.0	41.75	33.72	29.05	26.44	24.94	24.10	23.64

*Sum of corresponding values from Table 5.6 and the last column from Table 5.7.

Table 5.8. Total User Cost as a Function of Vehicle Speed and Pavement Roughness (Composite Vehicles).

TABLE 5.9 - EXAMPLES OF EXTRA USER COST CALCULATIONS

5.9a. INITIAL SI = 4.5, SPEED LIMIT = 70 MPH

SPEED (MPH)	COST (CENTS/MILE) FOR SI OF						
	1.5	2.0	2.5	3.0	3.5	4.0	4.5
10.0	31.47	31.47	31.30	31.30	31.30	31.30	31.30
15.0	16.17	16.07	15.86	15.81	15.76	15.76	15.76
20.0	8.63	8.39	8.06	7.96	7.87	7.87	7.87
25.0	4.48	4.05	3.59	3.44	3.30	3.29	3.30
30.0	2.18	1.52	0.95	0.67	0.48	0.43	0.38
35.0	1.19	0.16	-0.58	-1.02	-1.27	-1.37	-1.37
40.0	1.56	0.00	-0.96	-1.61	-1.93	-2.04	-2.09
45.0	2.58	0.36	-0.98	-1.74	-2.20	-2.32	-2.44
50.0	3.95	0.94	-0.75	-1.75	-2.32	-2.44	-2.63
55.0	6.50	2.60	0.37	-0.95	-1.72	-1.98	-2.15
60.0	9.50	4.96	1.63	-0.09	-1.04	-1.48	-1.65
65.0	13.76	7.27	3.60	1.42	0.19	-0.41	-0.83
70.0	18.11	10.08	5.41	2.80	1.31	0.47	0.0

5.9b. INITIAL SI = 4.5, SPEED LIMIT = 55 MPH

SPEED (MPH)	COST (CENTS/MILE) FOR SI OF						
	1.5	2.0	2.5	3.0	3.5	4.0	4.5
10.0	33.62	33.62	33.46	33.46	33.46	33.46	33.46
15.0	18.32	18.22	18.02	17.97	17.91	17.91	17.91
20.0	10.78	10.55	10.21	10.12	10.02	10.02	10.02
25.0	6.63	6.21	5.74	5.60	5.46	5.44	5.46
30.0	4.34	3.67	3.10	2.82	2.63	2.58	2.53
35.0	3.34	2.31	1.57	1.13	0.88	0.78	0.78
40.0	3.71	2.15	1.19	0.54	0.22	0.11	0.06
45.0	4.73	2.52	1.17	0.41	-0.05	-0.17	-0.29
50.0	6.10	3.09	1.40	0.40	-0.17	-0.29	-0.48
55.0	8.65	4.75	2.52	1.20	0.43	0.17	0.0
60.0	11.65	6.61	3.79	2.06	1.11	0.67	0.50
65.0	15.91	9.43	5.76	3.57	2.34	1.74	1.32
70.0	20.26	12.23	7.57	4.95	3.46	2.62	2.15

this analysis, and are designated designs 1, 2, and 3. Total user costs for 20 years are calculated for each of the designs. An example to demonstrate the calculation procedure is given in Figure 5.12, whereas total user costs for three different traffic situations, two discount rates, and two different speed limits, as shown below, are given in Table 5.10. A plot of user cost vs terminal serviceability level (55 mph speed limit) is given in Figure 5.13.

Traffic Case Number	ADT Initial	ADT 20 Year
1	30,000	60,000
2	50,000	100,000
3	75,000	150,000

The designs were evaluated for discount rates of 0 and 6 percent, and for speed limits of 55 and 70 MPH.

It is assumed for the analysis that users will not reduce their speed to adjust for roughness during the 20 year analysis period. This is a reasonable assumption considering the following:

1. That no pavement deteriorates to a very high roughness level (worst case being SI = 2.0 in the 20th year).

2. That in and around metropolitan areas where zero-maintenance pavements will be generally considered, little reduction in average operating speed is expected to compensate for increasing pavement roughness.

Depending upon the available capacity of a highway, there may be a reduction in average speed as traffic reaches the highway capacity, particularly during the latter years of the analysis period. As

Figure 5.12. Example Calculation of Total User Cost

1 Year	2 SI Avg. for the year	3 Avg. Speed mph	4 Extra U. Cost c/mile/veh. (from Table 5.9a)	5 Avg. Traffic	6 Cost/yr/mile Column 4 x Column 5 x 365/100	7 Discounted Cost/yr/mile $\frac{\text{column 6}}{(1.06)^t}$
1	4.485	70	0.01	30,750	1,122	1,058
2	4.455	70	0.04	32,250	4,708	4,190
3	4.420	70	0.07	33,750	8,623	7,240
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
12	3.870	70	0.68	47,250	117,275	58,282
-	-	-	-	-	-	-
16	3.275	70	1.98	53,250	384,838	151,487
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
20	2.000	70	8.34	59,250	1,803,629	562,386
				TOTAL	5,716,093	2,106,149

Example pertains to the following:

1. Performance history as indicated in Column 1,
2. Initial ADT = 30,000; 20 year ADT = 60,000.
3. Speed limit = 70 mph, Avg. operating speed = 70 mph
4. Discount rate = 6%

$$\text{Total discounted user cost/mile for n years} = \sum_{t=1}^{t=n} ADT_t \times E_t \times \frac{365}{100} \times \frac{1}{(1+D)^t}$$

Where ADT_t = Average traffic for year t

E_t = Excess user cost in year t, cents/mile/vey (from Table 5.9a)

D = Discount rate

Table 5.10. Total Extra User Cost* for Four Zero Maintenance Designs

	Design	Terminal SI at 20th year	Speed Limit = 70 mph Total 20-yr U.Cost/Mile Undiscounted	Disc.@ 6%	Speed Limit = 55 mph Total 20-yr U.Cost/Mile Undiscounted	Disc.@ 6%
ADT 30,000 to 60,000 in 20 years	Design 1	2.0	5,716,000	2,106,000	2,473,000	893,000
	Design 2	2.5	3,738,000	1,416,000	1,522,000	565,000
	Design 3	3.0	2,324,000	910,000	859,000	333,000
	Design 4	3.5	1,367,000	556,000	472,000	193,000
ADT 50,000 - 100,000 in 20 years	Design 1	2.0	9,527,000	3,510,000	4,122,000	1,488,000
	Design 2	2.5	6,230,000	2,360,000	2,537,000	942,000
	Design 3	3.0	3,873,000	1,517,000	1,432,000	555,000
	Design 4	3.5	2,278,000	927,000	787,000	322,000
ADT 75,000 - 150,000 in 20 years	Design 1	2.0	14,290,000	5,265,000	6,183,000	2,233.000
	Design 2	2.5	9,345,000	3,540,000	3,805,000	1,413,000
	Design 3	3.0	5,810,000	2,275,000	2,148,000	833,000
	Design 4	3.5	3,418,000	1,390,000	1,180,000	483,000

* Unit Prices as shown in Table 5.4

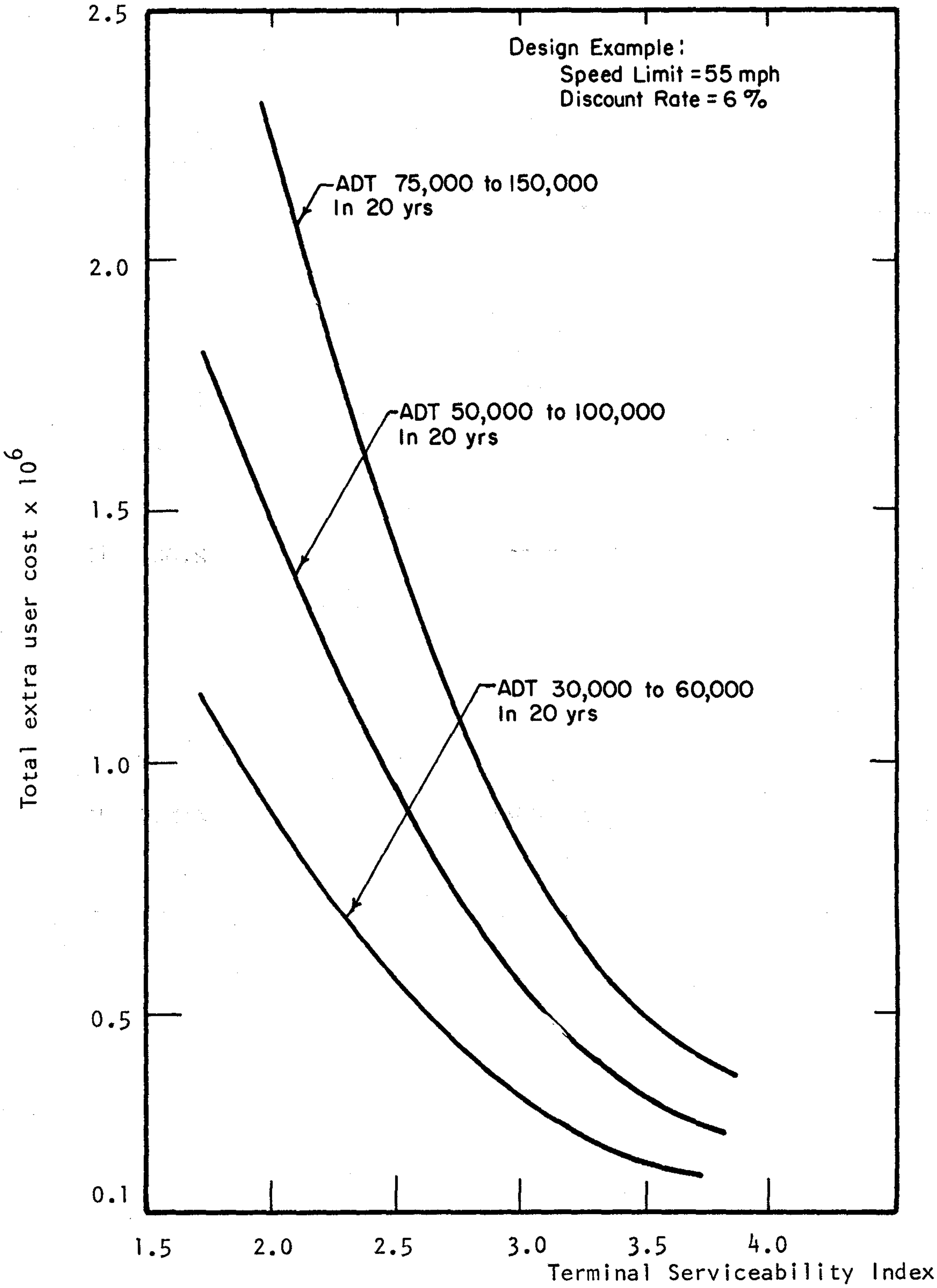


Fig. 5.13 Total extra user costs vs. terminal serviceability for different traffic conditions. (From Table 5.10).

described in the rationale for user cost, this will result in a reduction in vehicle operating cost and an increase in the user time cost. This may result in a slight net change in user cost. This net effect may not be considered in pavement design analysis since it is or should be considered for decisions regarding major reconstruction such as addition of lanes or widening of shoulders, etc.

Summary

1. User costs can be calculated for any time period during the life of a pavement and provide an important index of user effects related to selection of critical limiting serviceability for design of zero-maintenance pavements.

2. Total user costs (vehicle operating and user time) over a 20-year life span increases dramatically as the terminal serviceability at the 20th year is decreased from 3.5 to 2.0 and such a reduction is highly nonlinear. (Figure 5.13).

5.4 LIMITING DISTRESS CRITERIA FOR ZERO-MAINTENANCE DESIGN

The distress in terms of type, severity, and amount that is visually observed on a pavement is a very significant factor in influencing maintenance management of a pavement. This section summarizes maintenance policies and procedures, analyzes relationships between distress and serviceability, and determines critical distress levels when maintenance is performed.

Maintenance Management Policies

The policies and procedures of maintenance management responsible for a particular pavements have a strong influence on when structural maintenance is performed. Each highway agency has various maintenance policies and requirements. Generally, pavement distresses affecting safety, ride quality, and structural integrity are given the highest priority, whereas distresses having long range effect on ride quality and structural integrity are usually given the second priority. Levels of severity for each distress type are also sometimes specified to decide the time when maintenance must be applied. An example of these levels for typical distress types occurring on high-traffic volume pavements are specified by the California D.O.T., and are summarized in Table 5.11 for flexible and Table 5.12 for rigid pavements. Some states provide very limited guidance to the maintenance personnel with regard to the time when maintenance should be applied, therefore much variation in policies and practices exists across the U. S.

Several surface evaluation systems for maintenance management have been developed by various state highway departments and other agencies (45, 73, 146). Basically, both the riding quality and visible structural distresses of the pavement are considered in determining a condition index. One significant procedure was developed by the Washington Highway Commission, whereby a numerical rating for the pavment is determined according to the following formula (73):

$$R_R = \sqrt{G_R \times G_D} \tag{5.1}$$

where

R_R = final rating

G_R = ride rating = 100 - (10 x ride score)

G_D = defect rating = 100 - Σ negative values for pavement defects

Table 5.11. Critical Distress Levels when Maintenance is Recommended for Flexible Pavements (California DOT, Reference 20).

Distress Type	Critical Distress Level When Maint. Recommended
1. Cracks	$\geq$ 1/4 in. wide
2. Rutting (wheel path)	> 1 in. depth or when water is impounded
3. Potholes	Immediately
4. Depressions/Swells	$\geq$ 1 1/2 in. vertical deviation in 50 ft longitudinal, or when riding quality is objectional
5. Raveling	Before safety is impared
6. Lane/Shoulder Dropoff	> 3/4 in or edge failure is apparent

Table 5.12. Critical Distress Levels When Maintenance is Recommended for Rigid Pavements (California DOT, Ref. 20).

Distress Type	Critical Distress Level when Maint. Recommended
1. Cracks	$\geq$ 1/4 in. wide
2. Spalling (transverse)	> 4 in. in length in direction of travel
(longitudinal)	when affects ride quality
3. Lane/Shoulder Dropoff	> 3/4 in. or edge failure becomes apparent
4. Depressions/Swells	$\geq$ 1 1/2 in. vertical deviation in 50 ft. longitudinal, or when riding quality is objectional
5. Joint Separation	> 1/4 in. wide

The procedure is described as follows:

> "The ride rating and defect rating are obtained in this system through subjective evaluation of the roadway by one or more qualified observers. They first drive over the section of pavement at normal driving speed to determine the ride rating. In scoring the ride, a scale of 0 to 10 is used; a rating of 0 indicates a perfect ride, and a rating of 10 would indicate a roadway which is virtually impassable. The raters then retrace the route to detect, evaluate, and list any defects on prepared rating data forms according to established categories and guidelines. The numerical values assigned to pavement defects increase with the seriouness and severity of the distress." (73)

Limiting values for the final rating R_R were proposed for various types and classes of pavements as follows:

Asphalt Concrete and Bituminous Pavements

Highway Class	Points
Interstate	60
Principal	60
Major	55
Collector	50
Other	45

Portland Cement Concrete

All classes	50

Although the priority of major maintenance or rehabilitation can be determined easily, indices such as this cannot be used in design since there exists no relationship relating the index to design factors.

Other existing evaluation systems have similar deficiencies making it impossible to use them for setting limiting criteria for zero-maintenance pavements.

Relationships Between Distress and Serviceability

Pavement serviceability is highly related to pavement distress. Relationships between distress types which require maintenance and the serviceability can be very useful in ultimately selecting levels of distress as a limiting criteria for zero-maintenance pavement design. This section summarizes existing relationships, and present a newly-developed relationship between distress and serviceability.

The first relationships between serviceability and distress were developed by Carey and Irick (22) as follows:

$$PSI = C + A_1R_1 + B_1D_1 + B_2D_2 \tag{5.2}$$

where

C = coefficient (5.03 and 5.41 for flexible and rigid pavements, respectively)

A_1 = coefficient (-1.91 and -1.78 for flexible and rigid pavements, respectively)

R_1 = function of profile roughness (log (1 + SV), where SV = mean slope variance given by CHLOE profilometer)

B_1 = coefficient (-1.38 for flexible and 0 for rigid pavements, respectively)

D_1 = function of surface rutting (RD^2, where RD = mean rut depth along a pavement)

B_2 = coefficient (-0.01 for flexible and -0.09 for rigid pavements, respectively)

D_2 = function of surface deterioration ($\sqrt{C+P}$, where C+P = amount of cracking (in linear feet for rigid and square feet for flexible pavements), and patching (in square feet per 1000 ft^2 of a pavement).

The accuracy of these models can be judged by the following statistics:

Flexible: R^2 = multiple correlation coefficient = 84 percent

S_e = standard error of estimate of PSR = 0.38

Rigid: R^2 = 92 percent

S_e = 0.32

Work by Yoder and Milhous provided correlations between objective measurements given by various pieces of equipment and subjective panel ratings. Their results show that the standard error in predicting panel ratings from equipment measurements alone was only slightly larger than when pavement condition factors were included (such as cracking, patching, etc.).

Similar regression equations to predict the subjective serviceability ratings from objective measurements on a pavement were developed by the Texas Highway Department in 1968 (106). Similar R^2 and standard error of estimates were obtained. It was determined that a model containing only either slope variance or a roughness index explains about 67 percent of the mean rating panel opinion (R^2 = 67 percent).

Walker and Hudson developed a regression model using profile wavelength/amplitude characteristics. The form of equation is (147):

$$PSR = B_0 + B_1X_1 + B_2X_2 + B_3X_3 + \ldots B_NX_N \quad (5.3)$$

where

B_i = the linear model coefficients determined from regression, and

X_i = the average amplitude of i^{th} wavelength.

This regression model has an R^2 of 0.89, and the standard error of estimate of 0.33, which indicates this model can estimate the users' subjective PSR with about the same accuracy and confidence of the models that also include a pavement surface deterioration characteristic of rutting, cracking, and patching. The model has been used extensively by the Texas Highway Department, and demonstrates that it is possible to accurately estimate the subjective user rating of a highway pavement by the wave length/amplitude characteristics of the longitudinal pavement profile.

Studies have shown that major distresses have an influence in reducing serviceability rating (154). The correlation coefficients shown in Table 5.13 indicate the effects of these distresses to serviceability rating. For example, a correlation coefficient of -0.8 between transverse faulting and pavement serviceability indicates that serviceability decreases significantly as faulting increases. However, no correlation exists directly relating this or any similar distress to the serviceability of the pavement as all previous studies have included a general roughness measurement which usually "dominates" the regression equation and makes the specific distress seem relatively insignificant. This leads to an erronesous interpretation since there exists a high correlation between distress and roughness. Hence, an attempt is made in the present study to develop relationships between various major distress types and present serviceability rating (PSR).

Table 5.13. Summary of Simple Correlation Coefficients (Ref. 154).

PHYSICAL MEASUREMENT	COEFFICIENTS		
	RIGID	OVERLAY	FLEXIBLE
Average transverse fault	-0.80	---	---
Blowups	-0.74	---	---
Bituminous patching	-0.65	---	---
Joints (number)	0.65	---	---
Maximum transverse fault	-0.61	---	---
Average longitudinal fault	-0.40	---	---
Cracking & patching	-0.37	-0.63	---
Maximum rut depth	---	-0.90	-0.74
Bituminous patching	---	-0.69	-0.57
Average rut depth	---	-0.63	---
Bleeding*	---	---	-0.69
Absolute minimum value**	0.35	0.37	0.30

* Sum of major, intermediate, and minor bleeding.

** Minimum correlation coefficient for 90 percent probability that coefficient in non-zero.

A multiple regression analysis was made to find a mathematical model of the relationship between the amount of distress and the serviceability of a pavement using data collected in previous studies (22, 106), and in some of the current field visits. The resulting expressions are as follows:

Flexible pavement:

$$PSI = 4.5 - 0.49394\, \overline{RD} - 1.16171\sqrt{\overline{RDV}}\,(1 - 0.08737\sqrt{\overline{RDV}}) - 0.13044 \log (1 + TC) - 0.03439\sqrt{AC+P} \tag{5.4}$$

$R^2 = 0.7689$

$s_e = 0.455$

Sample size = 95

where

$\overline{RD}$ = the mean rut depth in both wheel paths of the pavement where rut depth is the depression under the center of a 4-ft straightedge (inches)

$\overline{RDV}$ = mean rut depth variance (in^2 x 100)

AC = class 2 and class 3 alligator or fatigue cracking ($ft^2/1000\ ft^2$) as per AASHO definition (57)

TC = transverse and longitudinal cracking ($ft/1000\ ft^2$)

P = patching ($ft^2/1000\ ft^2$)

Rigid pavement:

$$PSI = 4.5 - 0.03648F - 0.0396S - 0.01496P - 0.08717\sqrt{C+S} - 0.78144 \log (1 + F + C) \tag{5.5}$$

$R^2 = 0.8664$

$S_e = 0.4187$

Sample size = 65

where

F = faulting in wheel path (in/1000 ft)

S = spalling for areas greater than 3 in. diameter (ft^2/1000 ft^2)

C = class 3 and 4 cracking (ft/1000 ft^2) as per AASHO definition (57)

P = patching (ft^2/1000 ft^2)

The data used to derive these equations are given in Appendix B. The analyses were conducted using stepwise regression techniques. The standard error of estimate is higher than desirable for both expressions. It is noted, however, that the flexible pavement equation has 67 percent of the data within an interval of $\pm$ 0.45 of the calculated PSI value, and the rigid pavement equation has 67 percent of the data within an interval of $\pm$ 0.42 of calculated PSI value. Both equations contain only those parameters which are significant at the 5 percent significance level.

Plots illustrating the effect of the extent of each distress type on PSR are shown in Figures 5.14 through 5.20. Two curves are shown in each figure. The upper curve represents the predicted loss of serviceability when only the particular distress type under consideration is varied. The lower curve represents the effect on the loss in serviceability when the specific distress type under consideration is varied and specific values for all other distress types are held constant. The lower curves are more representative of actual distress situations since there usually exists more than one type of distress as a pavement deteriorates.

The results obtained from these expressions were checked against the data obtained from the field visits and from the AASHO data. In most cases,

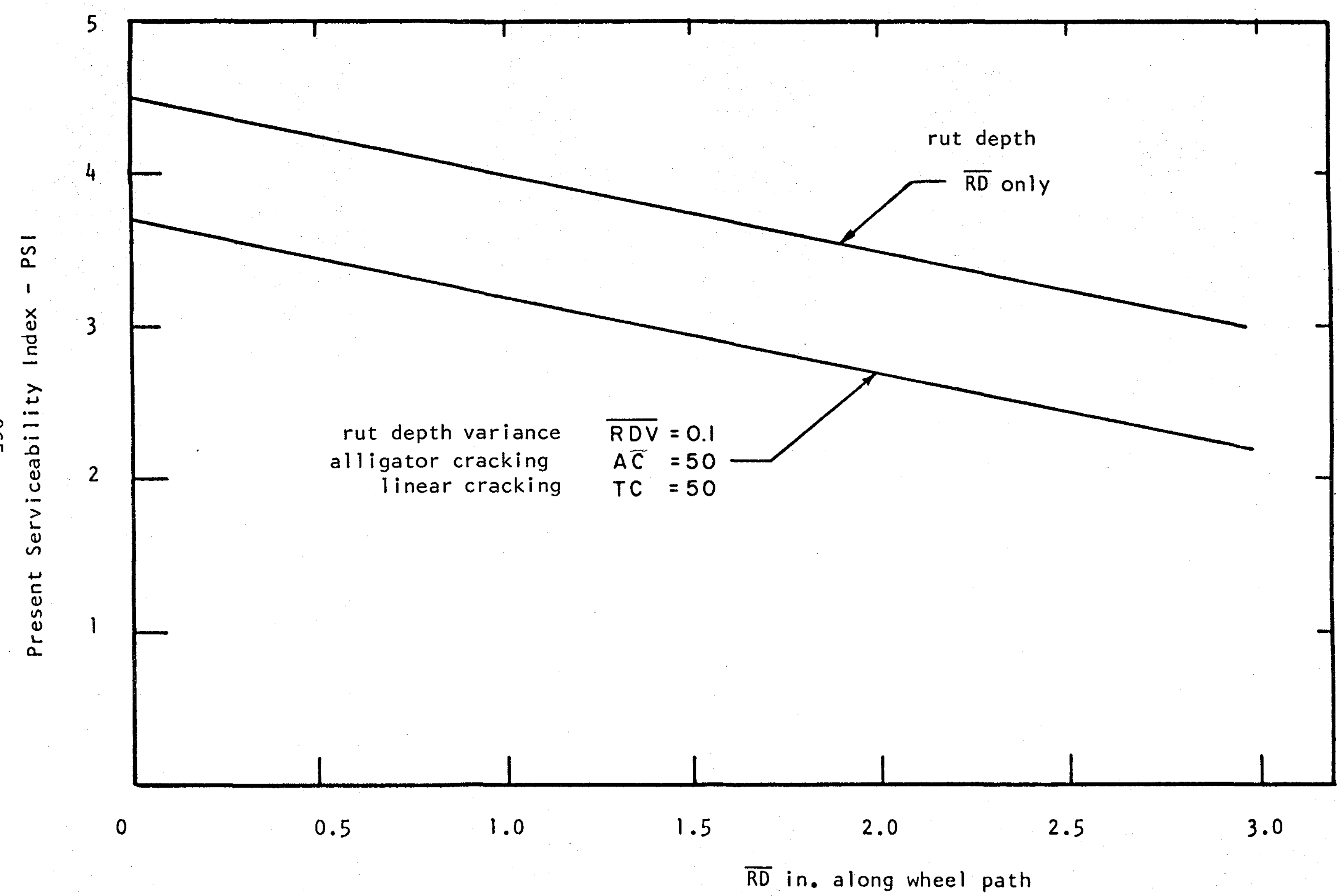


Fig. 5.14 . Mean rut depth versus serviceability for flexible pavements.

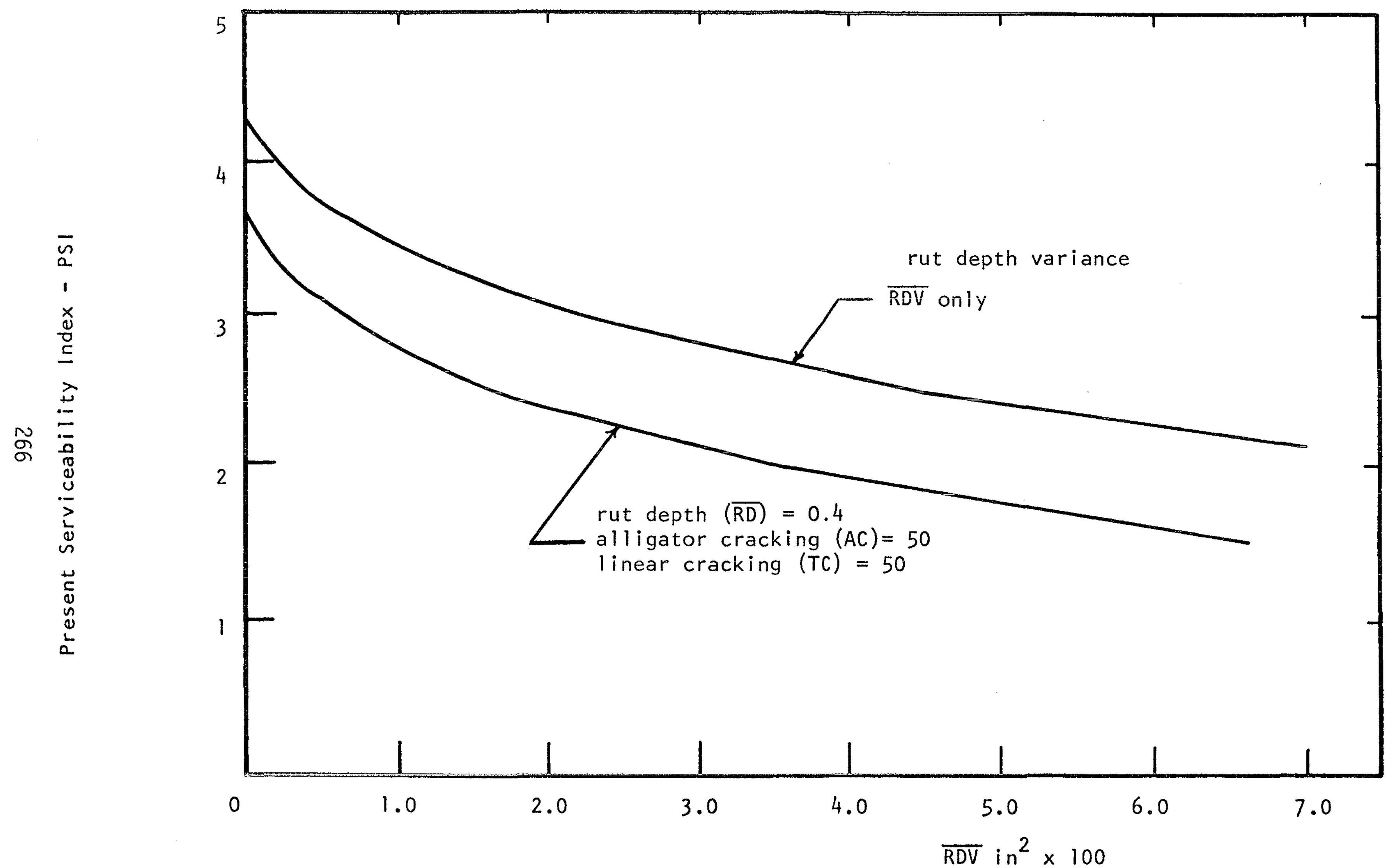


Fig. 5.15 Rut depth variance versus serviceability for flexible pavements.

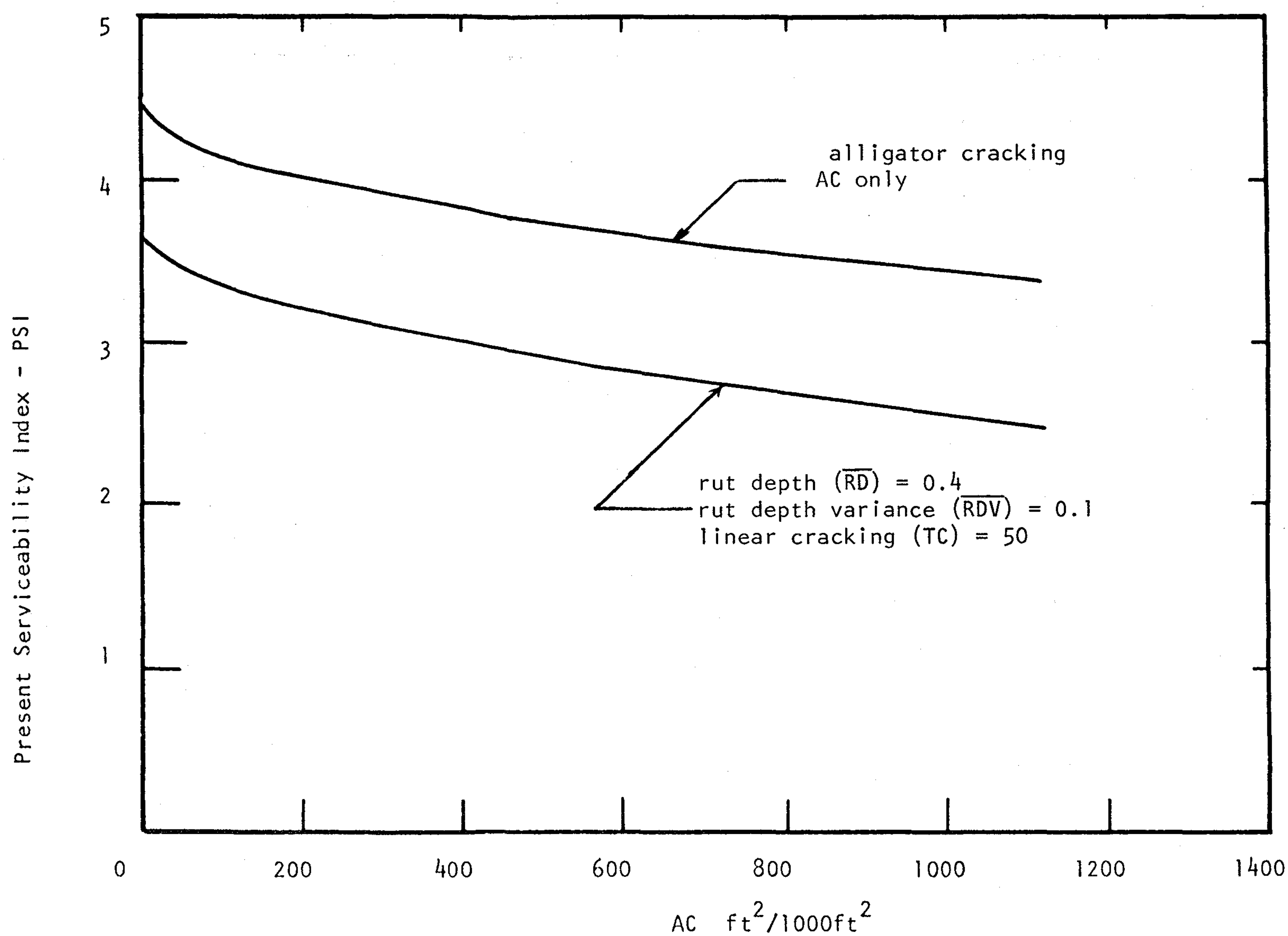


Fig. 5.16 Alligator cracking versus serviceability for flexible pavements.

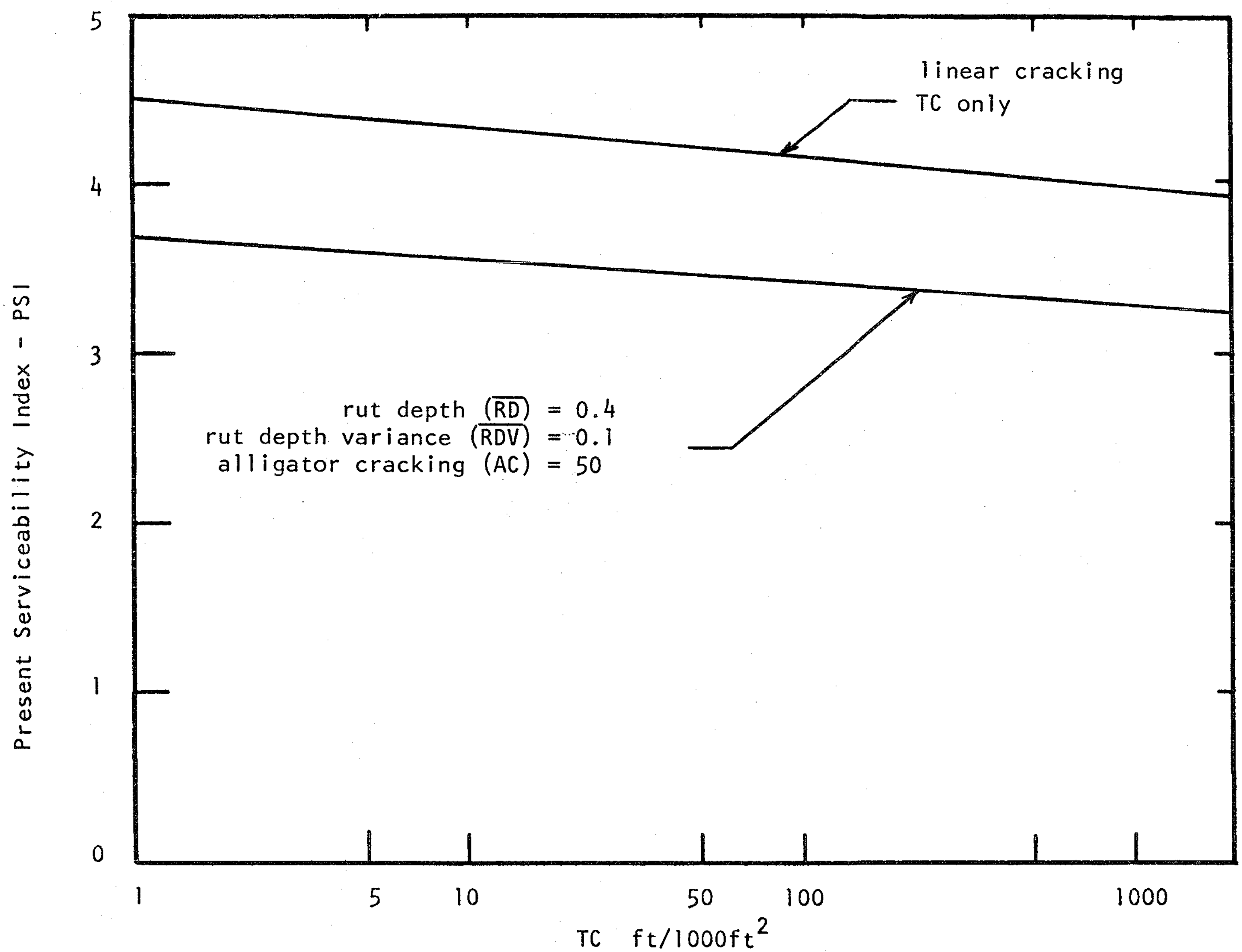


Fig. 5.17 Transverse and longitudinal cracking versus serviceability for flexible pavements.

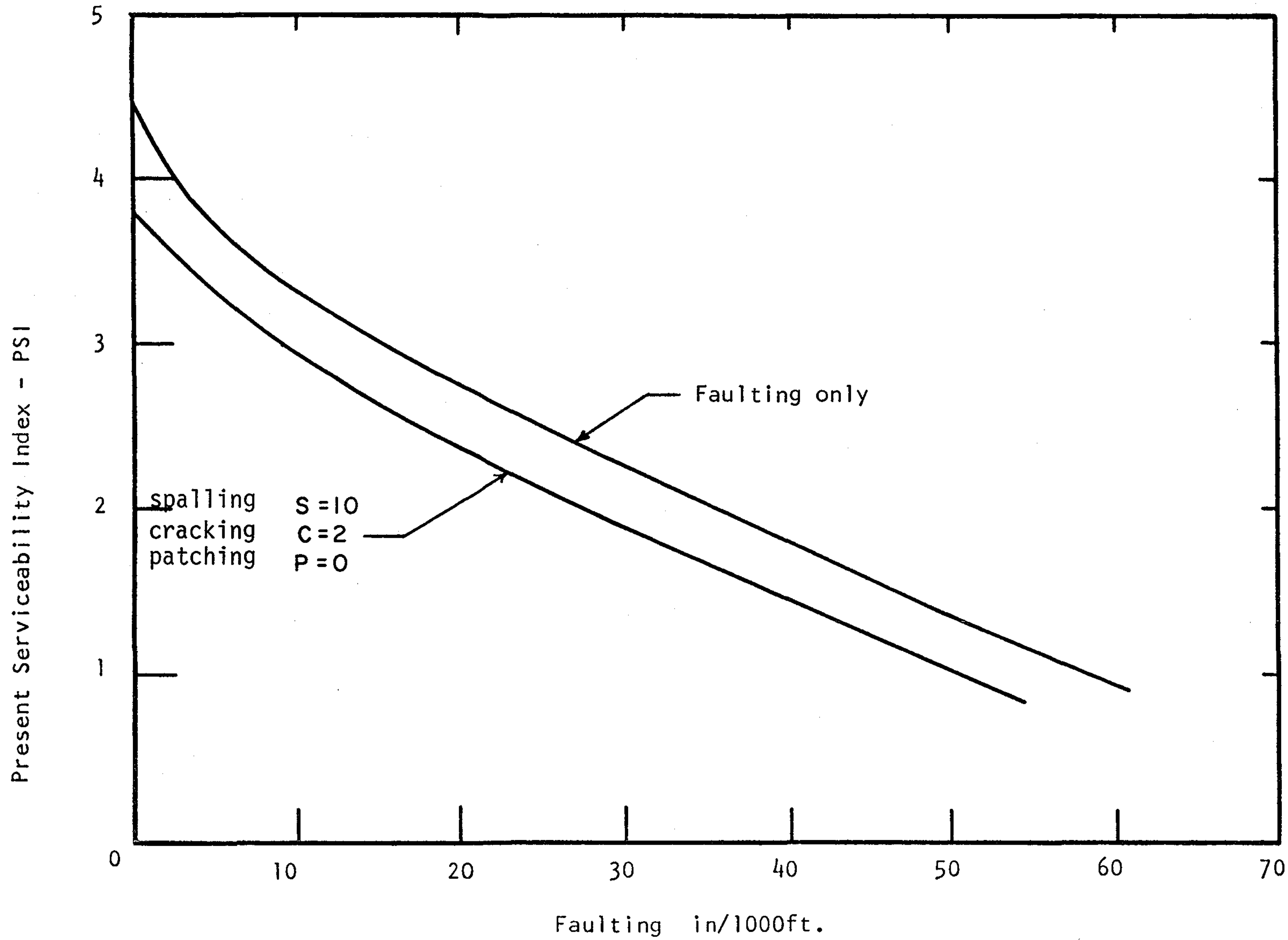


Fig. 5.18 Faulting versus serviceability for rigid pavements.

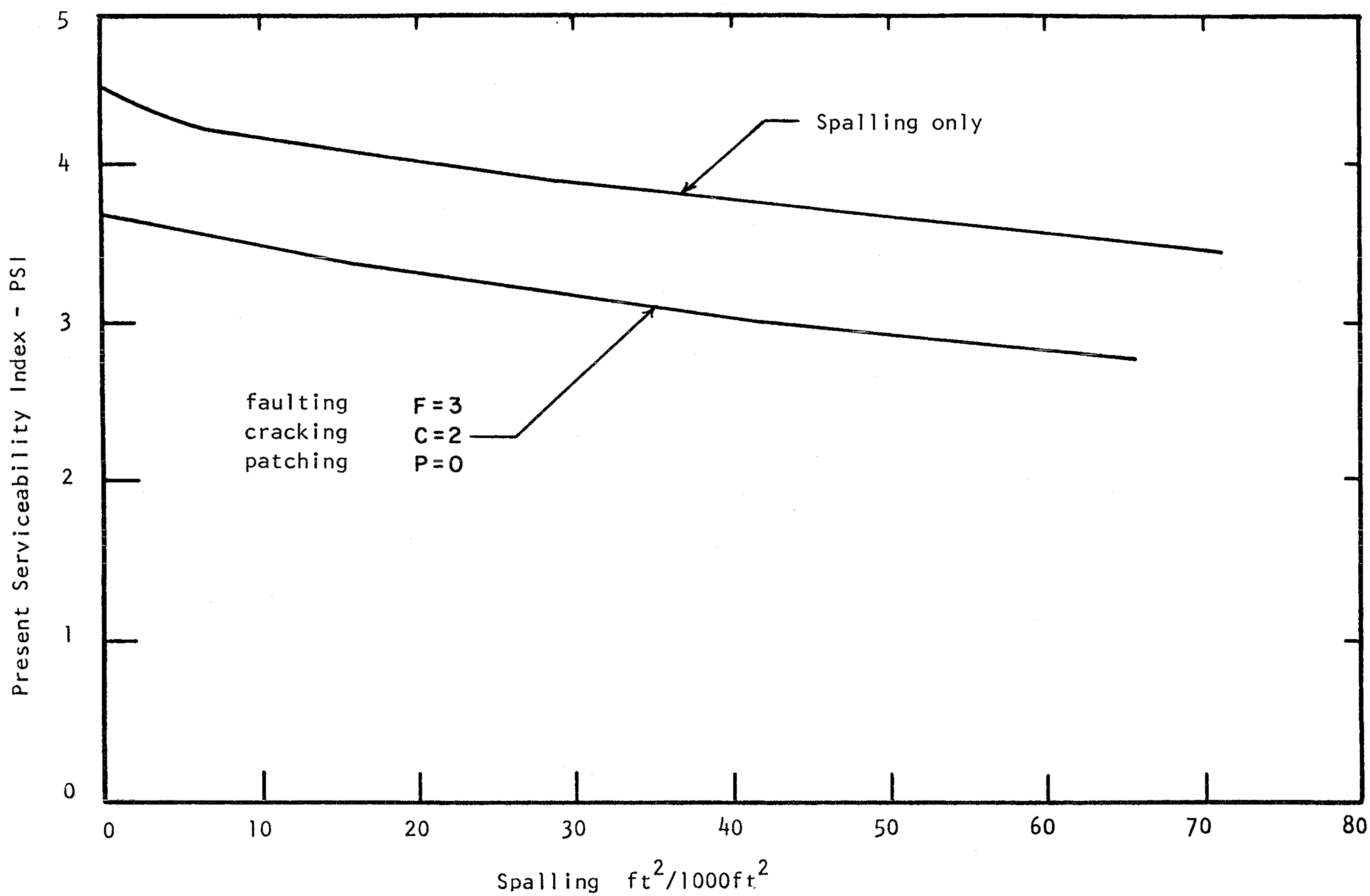


Fig. 5.19 Spalling versus serviceability for rigid pavements.

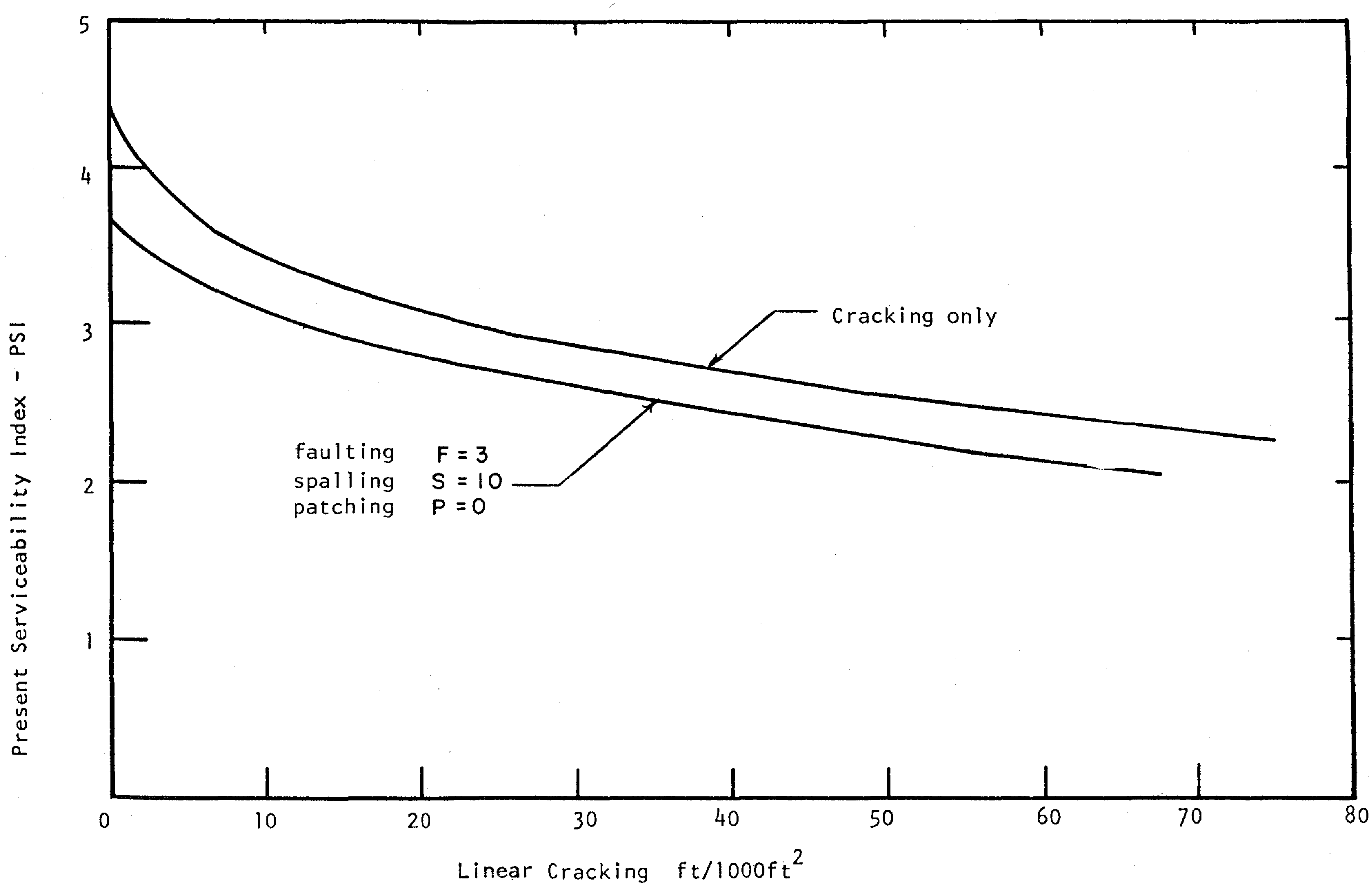


Fig. 5.20 Cracking versus serviceability for rigid pavements.

the results seem comparable. Attempts were made to develop similar expressions for CRCP, but adequate data were not available. These expressions are used in the following section to estimate how much distress can be allowed before the pavement serviceability drops to a limiting level.

Critical Distress Amounts

The major types of distress occurring on the five pavement types for high traffic volumes are identified in Chapter 3. Based upon information obtained from the field visits and the recommendations contained in the states' maintenance management policies, levels of distress when maintenance should be applied are given in Table 5.11 for flexible and composite pavements, and Table 5.12 for rigid pavements. The critical limits of distresses in terms of quantity at a given level of severity are estimated using Equations 5.4 and 5.5 at a prescribed terminal serviceability level ranging from 3.0 to 3.5. Terminal serviceability of between 3.0 and 3.5 for zero maintenance pavements are realistic values as shown in Figures 5.3 to 5.9. The critical levels of distress are summarized in Table 5.14 for single types of distress only, and for typical combinations of distresses. Critical levels of distress can be interpreted as follows:

Rigid Pavements:

1. Faulting - For faulting distress only, the limiting level of distress, as shown in Figure 5.18, varies from 7.5 to 15 inches per 1000 linear feet of pavement as the terminal serviceability varies from

Table 5.14. Critical Levels of Major Distresses for Rigid and Flexible Pavements for terminal serviceability ranging from 3.5 to 3.0.

Distress Type	One Distress Only	Typical Combinations of Distress **
(1) Rigid Pavements		
(a) Faulting in/1000 ft	7.5-15-0	3-9
(b) Spalling $ft^2/1000\ ft^2$	65-120	10-43
(c) Cracking $ft/1000\ ft^2$	10-25	2-12
(2) Flexible Pavements		
(a) $\overline{RD}$ in	*	0.4-1.4
(b) $\overline{RDV}$ $in^2 \times 100$	*	0.07-0.60
(c) AC $ft^2/1000\ ft^2$	*	25-410
(d) TC $ft/1000\ ft^2$	*	15-1000

* The single distress values omitted because they always correlated with other distresses, hence, the combined values give more reasonable results.

** Typical distresses combinations used to calculate the critical level of each distress are as follows:

(1) Rigid Pavements: Faulting = 3 in/1000 ft
Spalling = 10 $ft^2/1000\ ft^2$
Cracking = 2 $ft/1000\ ft^2$

(2) Flexible Pavements: $\overline{RD}$ = 0.5; $\overline{RDV}$ = 0.1; AC = 50; TC = 50

3.5 to 3.0, respectively. Assuming a joint spacing of 15 feet gives 66 joints per 1000 feet of pavement. Thus, the average faulting per joint would vary from 0.11 inches to 0.22 as the terminal serviceability varies from 3.5 to 3.0. These values are consistent with observations from the field survey as two pavements, JCP-7 and 11, with faulting in the range of .10 per joint were not maintained, whereas most engineers interviewed indicated an average fault of 1/4 inch should definitely be maintained.

2. Spalling - For spalling only distress, a check of Figure 5.19 indicates spalling of the magnitude of 65 ft^2 and approximately 120 ft^2 per 1000 ft^2 for terminal serviceability of 3.5 and 3.0, respectively. For spalling combined with other types of distress, the magnitude of spalling varies from about 10 $ft^2/1000\ ft^2$ to 43 $ft^2/1000\ ft^2$ as the terminal serviceability goes from 3.5 to 3.0. For a joint of 40 ft, there is approximately 24 lineal feet of joint per 1000 ft^2 of pavement. Thus, if each joint spalled for a distance of 6 inches to 1 foot on either side of the joint, the pavements would need maintenance. As intermediate cracking develops, the amount of spalling at each joint needed to require maintenance obviously goes down. These values are consistent with maintenance activities observed on pavements in service.

3. Cracking - For cracking distress only, the critical limit as seen from Figure 5.20 is between 10 and 25 ft/1000 ft^2 as the final serviceability goes from 3.5 to 3.0. This represents about one to two transverse cracks (greater than ¼ in. width) per 83 ft lane of a pavement. At the AASHO Road Test and the field surveys, this extent of cracking has been shown to reduce the serviceability to about 3.5 to 3.0.

Flexible Pavements:

For flexible pavement, each type of distress is correlated with all other major types, hence, only the combined distresses need be considered. The results are more difficult to interpret for flexible than for rigid pavements. The following observations can be made from Figures 5.14 through 5.14 for a terminal serviceability of 3.5.

1. Rut Depth - The critical limit for mean rut depth is approximately 0.4 inch for typical combined distresses.

2. Rut Depth Variance - Rut depth variance is the most significant parameter in influencing loss of serviceability index. The critical limit for the variance of rut depth is 0.07, i.e., standard deviation of rut depth along a flexible pavement should not exceed about $\sqrt{.07} = 0.26$ inch.

3. Alligator or Fatigue Cracking - The critical limit is 25 ft^2/1000 ft^2 of pavement in typical combination with other distresses. This is equivalent to one wheel path 1.5 ft wide and 17 ft long (within a pavement 12 ft wide by 83 ft in length (1000 ft^2)) with Class 2 or Class 3 cracking.

4. Transverse Cracking - The critical limit is 15 ft/1000 ft^2 of pavement in typical combination with other distresses. This cracking is equivalent to one transverse crack every 66 ft of a lane.

5.5 LIMITING FATIGUE CONSUMPTION CRITERIA FOR ZERO-MAINTENANCE PAVEMENTS

A study conducted by Darter (29) using the results from the rigid jointed pavements at the AASHO Road Test shows a correlation between PCC slab fracture damage and the cracking index. The loss of support due to pumping and thermal warping stresses as well as traffic loading stresses were considered in computing total fatigue damage. The cracking index at a fatigue consumption of 100 percent (using Equation 4.3) was predicted as approximately 80 ft/1000 ft^2. According to Equation 5.5, this cracking would reduce the serviceability index considerably below 3.0, and the pavement would require extensive maintenance. Another study described in Chapter 6 analyzes several sections at the Road Test that are still in-service on I-80 and pavement sections in the U. S., and arrives at similar conclusions, that is, designing for 100 percent fatigue consumption results in a considerable cracking, which requires maintenance. These studies and others to be extended during Phase II of this project will be used to determine an appropriate level of fatigue consumption to limit cracking to that recommended in Table 5.14.

5.6 SUMMARY

This chapter identified and quantified the major limiting criteria for use in designing zero-maintenance pavements. Three limiting criteria that are currently used in conventional design methods are identified as terminal serviceability, fatigue consumption, and maximum deflection.

The limiting criteria used by maintenance personnel on high volume pavements in determining the time when maintenance should be applied are based partly upon user considerations (i.e., safety, comfort, etc.), and partly upon maintenance management policiies. The two limiting criteria that provide direct evaluation of these factors and also are amenable for use in zero-maintenance pavement design are pavement serviceability and observable pavement distress. Comprehensive analyses of these factors and their effects is conducted, and the following limiting criteria selected.

1. Terminal Serviceability - A value of between 3.0 and 3.5 is recommended for all pavement types. Fifty percent of the projects surveyed during the field visits showed maintenance-free performance at a level of serviceability of 3.4 as shown in Figure 5.10. Also, user costs on urban freeways increase dramatically when the terminal serviceability drops significantly below this value, as shown in Figure 5.13. Such a high terminal serviceability level is not unusual, as the 50th percentile for acceptance of interstate highways in Texas is 3.4.

2. Distress Severity and Amount - Recommended amounts of major distresses in flexible and rigid pavements are given in Table 5.14 based upon their detrimental effects in reducing the pavement serviceability level to 3.5-3.0. These distresses are the major types occurring on high-traffic volume pavements, as shown in Chapter 3, and if they can be prevented or limited to the amounts shown in Table 5.14, the probability of obtaining a pavement with maintenance-free performance over a long time period is high.

CHAPTER 6

MAXIMUM MAINTENANCE-FREE LIVES OF CONVENTIONAL PAVEMENTS

6.1 INTRODUCTION

The purpose of this chapter is to establish the maximum maintenance-free life that can be reasonably expected from conventional pavements under high-traffic volumes, and located in various environments. The analysis contained in this chapter relates specifically to conventional pavements, which are defined as typical designs for JCP, JRCP, CRCP, FLEX, and COMP systems that have been constructed on high traffic volume pavements in the U. S. Specific definitions of "conventional designs" for each pavement type are provided in the analysis.

A few projects of each pavement type were found that exhibited maintenance-free performance under heavy traffic over relatively long time periods. An example of each pavement type is provided to illustrate the best conventional pavement performance found during the field survey. Some basic data for five such projects are summarized in Table 6.1, and photos of these pavements are shown in Figures 6.1 - 6.5. Details of their structural sections are given in Table 2.2 - 2.6 in Chapter 2. All of these pavements are structurally maintenance-free excepting some joint sealing and shoulder repair. The structural section of these pavements is somewhat greater than used in normal design (such as the 10-inch slab on CRCP-2), but these pavements have given excellent service for periods ranging from 10 to 23 years.

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Table 6.1. Maintenance-free life of selected pavements under heavy traffic.

Location, Pavement Type	Age, Years	1974 ADT	Total 18-kip ESAL, (x 10^6) *	18-kip ESAL x 10^6/ year
JCP-7 L.A. Calif.	20	129,000	32.70	1.63
JRCP-5 New Jersey	23	17,000	26.57	1.16
CRCP-2 Chicago, Ill.	10	71,000	10.30	1.03
FLEX-2 New Jersey	23	32,800	12.42	0.54
COMP-4 Toronto Ont.	14	81,500	27.93	2.00

* In heaviest traveled lane.

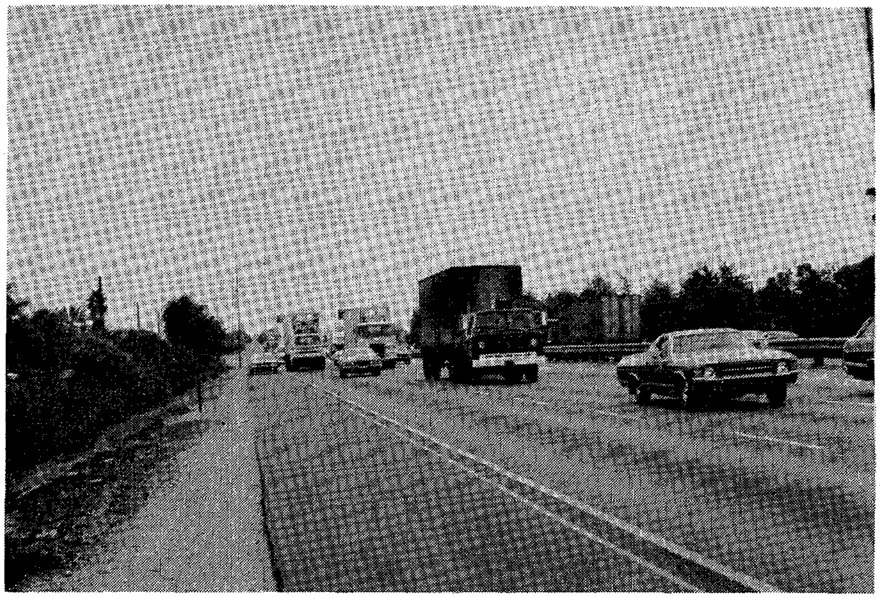

Figure 6.1. JCP-7, Jointed Concrete Pavement.

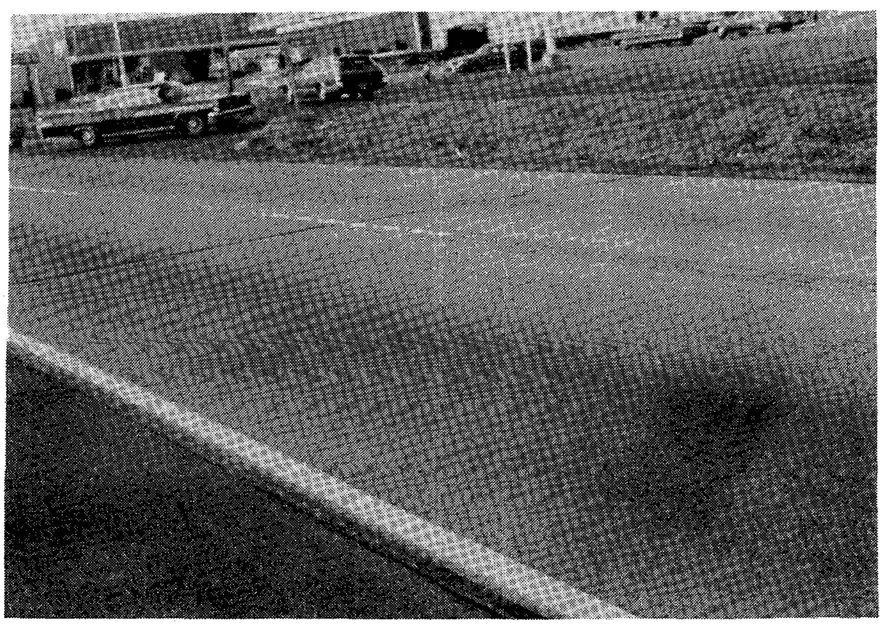

Figure 6.2. JRCP-5, Jointed Reinforced Concrete Pavement.

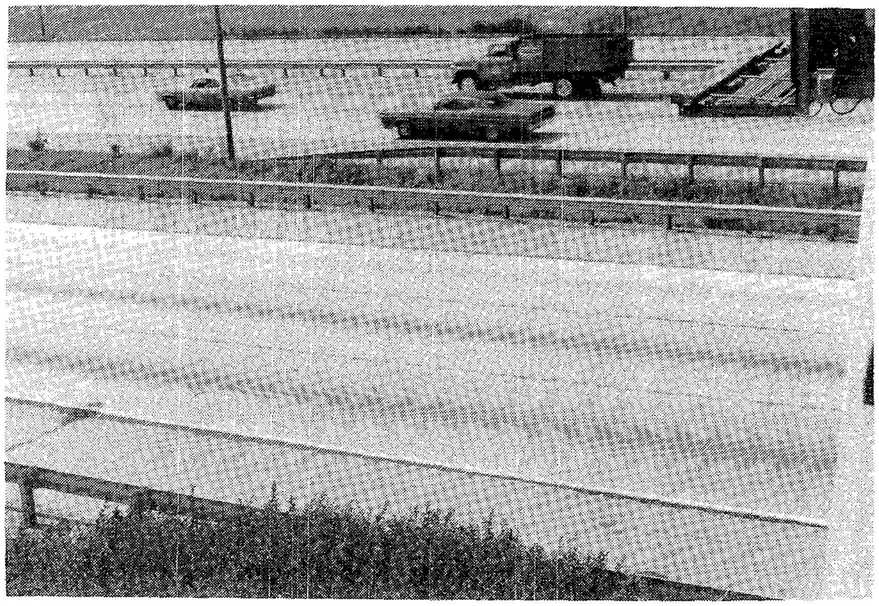

Figure 6.3. CRCP-2, Continuously Reinforced Concrete Pavement.

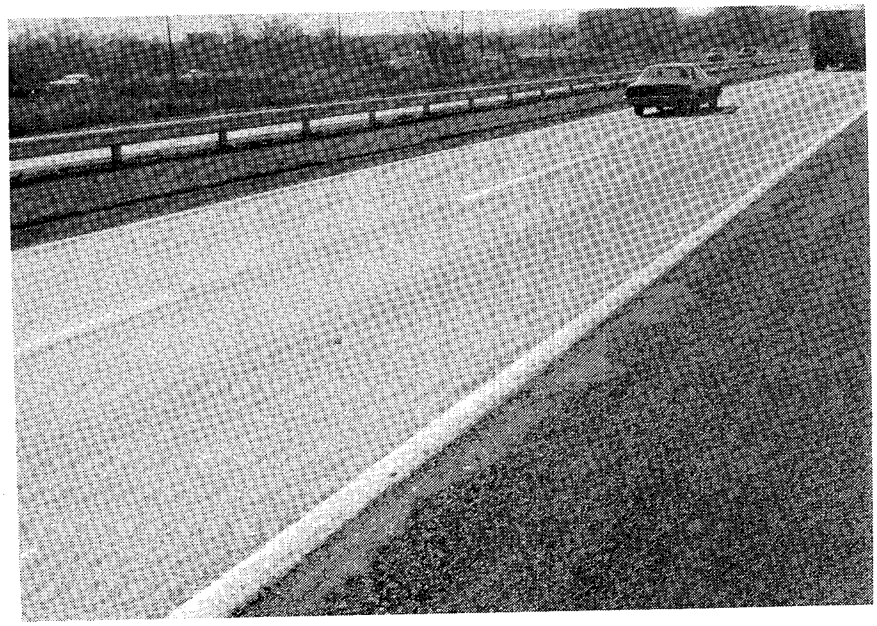

Figure 6.4. FLEX-2, Flexible Pavement.

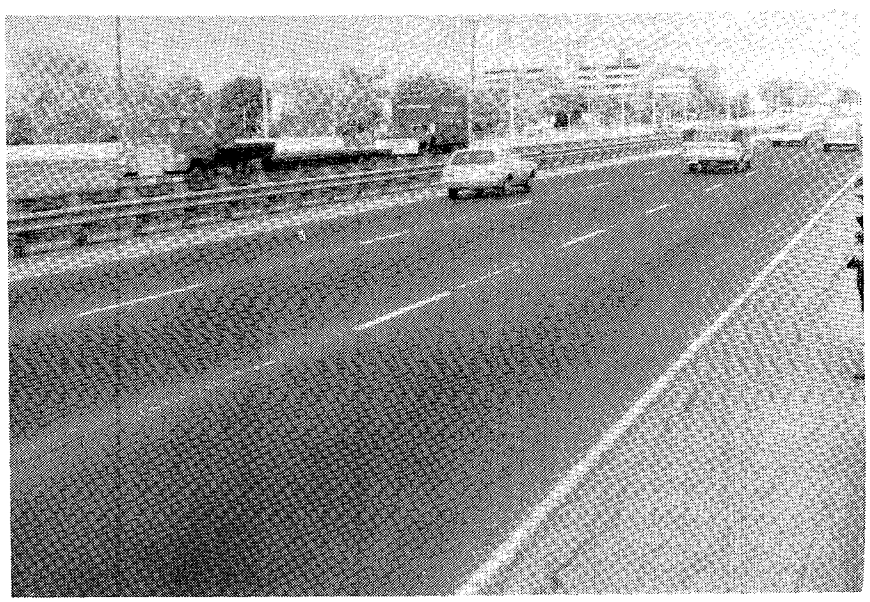

Figure 6.5. COMP-4, Composite Pavement.

6.2. JOINTED CONCRETE PAVEMENTS (JCP AND JRCP)

Conventional plain jointed (JCP) and jointed reinforced concrete (JRCP) pavements are defined as those typically found in past and current design and construction practice. Such pavements normally have an 8 to 10 inch slab thickness and non-stabilized or stabilized granular subbase. The portland cement concrete flexural strength usually ranges from 500 to 700 psi at 28 days, under 3rd point loading. Non-reinforced jointed pavements (JCPs) are built with or without LTDs (usually dowels) at contraction joints and with an average joint spacing of 15 ft, but some range up to 30 ft. Recent designs have incorporated skewed joints (1 to 6 skews) into the systems. Jointed reinforced concrete pavements (JRCPs) always have dowels or other mechanical load transfer devices at contraction and expansion joints, which are normally spaced in a range of 30 to 100 ft. The steel reinforcement ranges from approximately 0.10 to 0.20 in^2/ft depending upon joint spacing.

Field Performance

Results from the field survey are documented in Chapters 2, 3, 4, and 5, and provide useful information concerning the maintenance-free lives of JCP and JRCP.

Eleven of the 14 JCP projects showed maintenance-free performance. These 11 pavements ranged in age from 9 to 25 years with an average traffic loading of 1.0×10^6 18-kip ESAL per year in the heaviest traveled lane. Of the 18 JRCP projects, only 8 showed maintenance-free performance ranging in age from 10 to 24 years, and an average traffic loading of 0.9×10^6 18-kip ESAL per year.

The maintenance-free life, type of distress that required maintenance, and the maintenance actually performed on each of the JCP and JRCP projects are summarized in Tables 6.2 and 6.3. Joint sealing and shoulder repair were not considered in determining maintenance-free life in these tables. The range for maximum maintenance-free lives that can be obtained considering the best conventional designs and the existing traffic for each project is as follows:

Environment	JCP	JRCP
Wet-Freeze	5-25+	< 5-23+
Wet-Non/Freeze	12-24+	24+
Dry-Freeze	9-15+	no projects found
Dry-Non/Freeze	14-20+	< 5-15

Factors Affecting Maintenance-Free Life

Pavements are extremely complex physical systems involving many factors and interactions which affect maintenance-free life. The many factors influencing the maintenance-free life of JCP and JRCP can be placed in several broad categories.

1. Traffic-Environment: Stresses in concrete pavements results from several causes including traffic loadings, cyclic change in environmental conditions (temperature and moisture), volumetric changes of concrete (shrinkage and expansion), and loss of support. If the combination of stresses due to these causes becomes too large, fatigue fracture of the PCC slab will result. This phenomenon was found on 10 out of 18 conventional JRCPs and 3 out of 14 JCPs which ended their maintenance-free life. An analysis was conducted to determine the effect of traffic on the maintenance-free life of 6 JCP sections and 5 JRCP sections.

Table 6.2. Maintenance-Free Life of JCP Projects Included in Field Survey

Environment	Project No.	Maintenance-Free Life, (years)*	Average 18-kip ESAL/year**	Distresses Requiring Maintenance	Types of Maintenance Received
WET + FREEZE	1	< 10	0.60	CF, JF, TC, D	P
	2	16+	1.05		
	3	16+	1.09		
	4	16+	1.16		
	11	25	1.18	JF	
WET + NON-FREEZE	5	12+	0.36		
	12	24+	0.71		
DRY + FREEZE	9	9+	0.45		
	13	9	0.87	TC, CC	CS
	14	15+	0.87		
DRY + NON-FREEZE	6	< 15	0.65	TC	P, CS
	7	20+	0.64		
	8	14+	0.20		
	10	14+	1.97		

* Joint filling and shoulder repair have not been considered.

** 18-kip ESAL x 10^6. Heaviest traveled lane.

Abbreviations

Distresses:

JF: Joint Faulting
JS: Joint Spalling
CF: Crack Faulting
CS: Crack Spalling
TC: Transverse Cracking
B: Blowups
CC: Corner Cracking
LC: Longitudinal Cracking
D: "D" Cracking
DC: Diagonal Cracking

Maintenance:

CS: Crack Sealing
P: Patching with PCC or AC
JR: Joint Replacement
SR: Slab Replacement
M: Mudjacking
L: Limejacking

Table 6.3. Maintenance-Free Life of JRCP Projects Included in Field Survey

Environment	Project No.	Maintenance-Free Life (years)*	Average 18-kip ESAL/year**	Distresses Requiring Maintenance*	Types of Maintenance Received
WET + FREEZE	5	23+	1.16		
	6	< 10	0.60	TC,CF,CS,CC,D	P
	7	< 10	1.05	TC,CF,JS,CS,D	P
	8	< 10	1.09	TC,CF,CS	P
	9	16	1.16		
	10	12+	0.64		
	11	10+	2.45		
	12	< 10	0.70	TC,CF,JS,B	P
	13	< 5	0.35	CF,JS,TC,CC	P, JR, SR
	14	10+	0.91		
	15	< 10	0.35	TC,CF,JS,LC,B	P, L, SR
	16	< 10	0.48	JS,JF,TC,B	CS, P, SR
	17	14+	0.33		
	18	< 5	0.82	TC,B,CC,JF,CF,JS	P, CS
WET + NON-FREEZE	1	24+	0.16		
	2	22+	0.08		
DRY + NON-FREEZE	3	< 15	0.21	CC,LC,JS	P
	4	< 5	0.34	TC,LC,DC	M, CS, SR

* See Table 6.2

** Average 18-kip ESAL x 10^6. Heaviest traveled lane.

Detailed information about the traffic, materials, and climate were obtained, and a comprehensive fatigue analysis was conducted. Load associated stresses were computed by Westergaard's edge stress model (149) and by a finite element model (175). Extensive analyses using the finite element model have shown that the critical fatigue damage location in the PCC slab is at the edge of the slab, midway between joints. Results show that even though only a relatively small proportion of truck wheel loads occur within about 6 inches of the slab edge, these loads result in the highest accumulative fatigue damage at the slab edge. The accumulated fatigue damage at the slab edge resulting from a typical lateral wheel distribution (with a mean lateral distance of 24 inches from the slab edge to the outside of the truck duals) is equivalent to damage caused at the edge, calculated by taking the proportion of traffic in the outer 6 inches and considering it all as true edge loads on the slab. This proportion of traffic in the outer 6 inches varied between 5 and 10 percent of the total traffic over the range of variables studied.

Warping stresses were calculated using Bradbury's model for edge stress (19) and with a finite element model previously mentioned (175). The analyses showed that when the modulus of foundation support (k) was less than about 100 pci, the warping stress calculated with the Bradbury model and load stresses calculated with the Westergaard model could be added directly to produce approximately the same stress that was obtained using the finite element model which calculated a combined curl and load. The variation of thermal gradients during the days and nights for each month of the year was considered in the analysis. The change of several factors over the life of the pavements was also considered, including: the modulus of foundation reaction due to moisture changes and freezing temperatures, and the modulus of elasticity and flexural strength of the concrete.

Total fatigue damage was computed using Miner's damage model (87) summed over the traffic load distribution and time (including change in material properties and foundation support) on a monthly basis over the entire life of each pavement.

$$D = \sum_{j=1}^{m} \sum_{i=1}^{k} \frac{n_{ij}}{N_{ij}} \tag{6.1}$$

where

D = total accumulated fatigue damage of PCC slab over life of pavement

n_{ij} = number of load applications of magnitude i applied during the j^{th} time period at the edge of the slab (10 percent of total load applications passing over the traffic lane was used).

N_{ij} = number of allowable load applications of magnitude i occurring during j^{th} time period (determined from PCC fatigue curve by Kesler (19)).

k = total number of load ranges considered in the analysis

m = total number of months in pavement life

A cracking index (linear feet of Class 3 and 4 cracking* per 100

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*Class 3 crack - any crack opened or spalled at the surface to a width of ¼" or more over a distance equal to at least ½ the crack length.

Class 4 crack - any class 3 crack that has been sealed.

square feet of pavement) was measured on each section and the summary of the results is given in Table 6.4. A plot of cracking index versus concrete damage summation due to fatigue is shown in Figure 6.6. There are several interesting aspects to this plot and Table 6.4:

a. As total accumulated fatigue damage increases, cracking index does not increase until a level of fatigue damage is reached (i.e., 10^{-4}), but then increases rapidly with additional cumulation of fatigue damage. The dashed line is intended only to illustrate the general increase in cracking.

b. All projects having a fatigue damage summation less than a particular amount (10^{-4}) were maintenance-free, and most projects greater than this amount required maintenance.

c. A fatigue analysis such as this can provide a reasonable estimation of maintenance-free performance of pavements considering load-environment factors only. Pavements included in this analysis were located mainly in Illinois, but one was in California, and two in New Jersey. Pavements analyzed showed no evidence of serious joint deterioration, and therefore these results are only valid for pavements with sound joints. These results at least illustrate that fatigue distress can be estimated with reasonable accuracy using a mechanistic approach which can be used as a basis for developing thickness design procedures for jointed concrete pavements. Much more development of the methodology is necessary, however, before such procedures can be used with confidence.

2. <u>Environment - Material:</u> Another important factor affecting maintenance-free lives of JCP and JRCP is environment-material. Environ-

Table 6.4. Summary of results from comprehensive fatigue analysis of JCP and JRCP projects.

Project No.	Damage Summation Σ n/N	Cracking Index (Class 3 + 4) ft./1000 ft.^{2}
JCP-1	0.121708	25
-2	0.000102	3
-3	0.0000003	0
-4	0.000000008	0
-7	0.008754	3
-11	0.000060	0
JRCP-5	0.058144	0
-6	119.479196	40
-7	0.560316	21
-8	0.005111	23
-9	0.000770	4

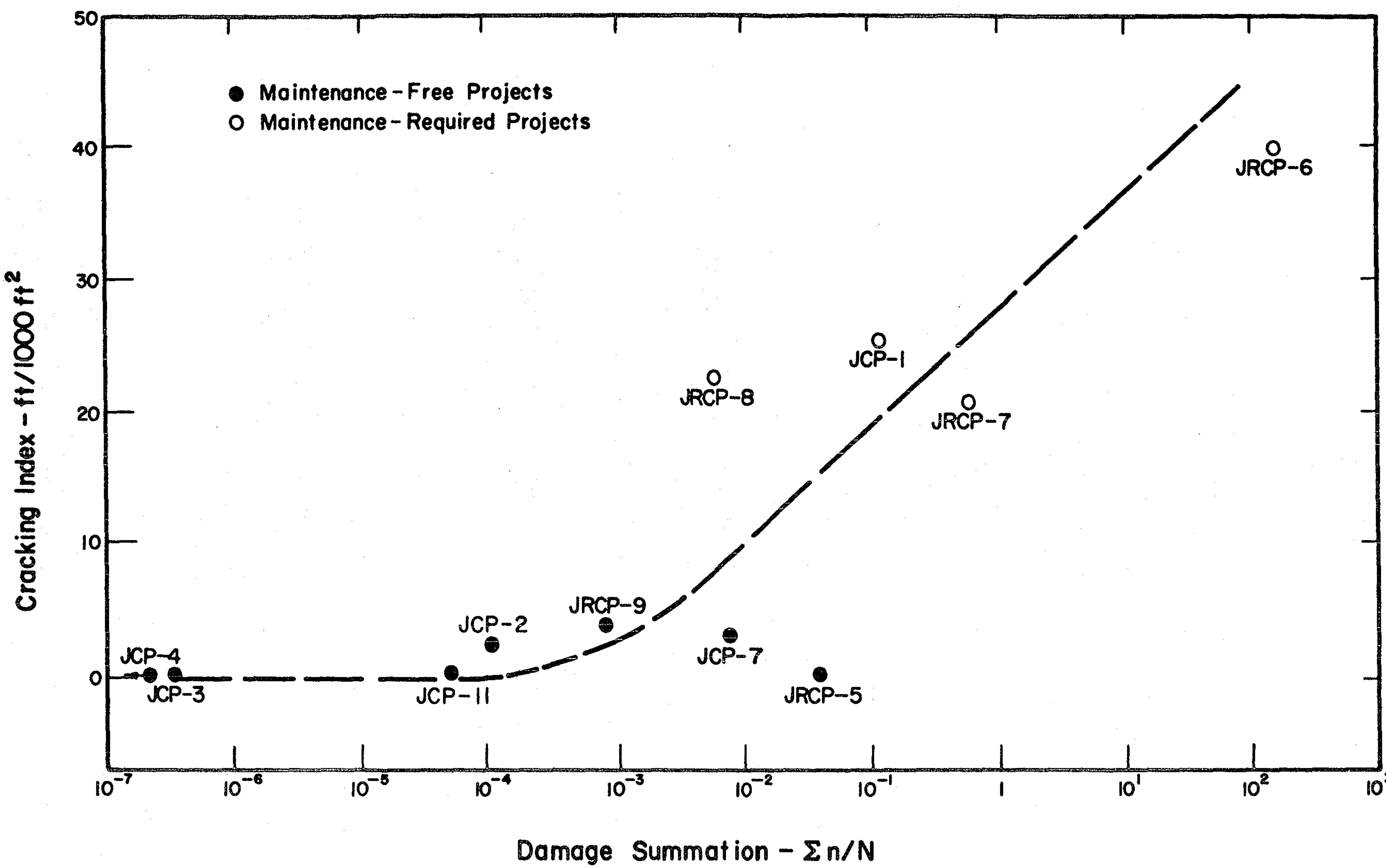

Figure 6.6. Fatigue damage summation versus cracking index for 11 JCP and JRCP projects.

mental factors such as temperature, moisture, and freeze-thaw have significant effect on material durability, strength, and resiliency properties. Shrinkage and swell of certain soils have significant effect on cracking of the slab, and hence, on maintenance-free life of the pavement. In some areas, this has reduced maintenance-free life to less than 10 years (i.e., Dallas-Fort Worth Turnpike, JRCP-4).

Durability of materials, especially concrete, is also a major factor to be considered in determining maintenance-free life of the concrete pavements. This becomes very important in freeze areas that use large amounts of deicing salt. Use of deicing salt not only may result in concrete deterioration, but is one of the important factors causing corrosion of dowels and subsequent lockup and rupture of reinforcing steel. This will cause pavement distress requiring maintenance in less than 15 years as shown in Table 6.3. Associated with the durability of concrete is the distress known as "D" cracking. The "D" cracking of portland cement concrete pavement is noted to be more prevalent and most severe in heavy freeze environments. This can cause severe deterioration which may require maintenance of concrete pavements in far less than 20 years.

3. Variability: There are many variations and uncertainties associated with load, materials, and climate, which can cause distress requiring maintenance. These have been identified, analyzed, and summarized by Darter (19), Kher and Darter (70), and Barenberg (12, 74) for concrete pavements. Load variations include such items as possible increases in axle loadings, changes in axle configurations, and increases in repetitions over that expected. Variations in material properties include such items as strength,

resiliency, fatigue life, and change in support. These variations have been found to cause localized failures which have a significant effect on the maintenance-free life of pavement both in the field surveys and analytically (30).

4. <u>Geometry:</u> Slab dimensions (length and thickness) and slab reinforcement, joint design (width, depth, load transfer, skewing, and sealants) are basic factors affecting maintenance-free lives of JCP and JRCP, as previously discussed.

5. <u>Construction:</u> Construction factors include possible errors which may occur including dowel misalignment, lack of consolidation, inadequate curing, rough surface, etc. These factors usually result in failures within a short time period, and hence, have significant effect on maintenance-free life.

Expected Maintenance-Free Life

Several factors that affect performance and consequently maintenance-free lives of JCP and JRCP have been identified. Unfortunately, there is no available predictive technique which can take into account the effects of all factors affecting maintenance-free life of concrete pavements. Therefore, the expected maximum maintenance-free life of conventional JCP and JRCP is estimated in the four climatic regions using three approaches:

1. Best available predictive models;
2. Actual project life based on observations from the field survey;
3. And, estimates of the project staff based on general performance found in the various climatic regions for particular designs.

Each of these approaches has limitations, and hence, all three estimates should be considered in arriving at final conclusions.

The predictive model derived from the results of the AASHO Road Test may be used to estimate pavement maintenance-free life using the serviceability-performance approach (1). Findings from Chapter 5 show that a terminal serviceability of between 3.0 and 3.5 is a reasonable limit for maintenance-free design criteria. If this criteria is used along with the AASHO predictive equation, the maintenance-free life may be roughly estimated for certain pavement designs including the effect of slab thickness, modulus of rupture and elasticity of concrete, modulus of foundation support, terminal serviceability, load transfer at joints, and number of equivalent 18-kip single axle load applications.

A summary of estimations of the maintenance-free life of various designs of JCP and JRCP are given in Tables 6.5, 6.6, 6.7, and 6.8. These estimates are made for specific conventionaldesign situations with the following constants and assumptions:

Traffic: heavy traffic at 1×10^6 18-kip ESAL/year in design lane

Construction: good quality with no significant errors that would limit life

Concrete flexural strength = 700 psi (28 days, 3rd point loading)

Concrete modulus of elasticity = 4.2×10^6 psi

Subbase = 6 inches thick, granular and stabilized

Table 6.5. Estimated Maximum Maintenance-Free Life for JCP with Slab Thickness of 8 Inches.

Stabilized Subbase / Load Transfer / Environment *****	NO		YES	
	Aggregate J = 3.3	Dowels J = 3.2	Aggregate J = 3.3	Dowels J = 3.2
WET + FREEZE		<5 JCP-1*		
	2	3**	3	5
	<5	<5***	<5	<10
WET	**** (17+)JCP-12			
	2	3	4	5
	<10	5-10	10-15	10-15
DRY + FREEZE	9 JCP-13 15+ JCP-14			
	2	4	3	6
	<5	<5	5-10	5-10
DRY			(10) JCP-6 (32+) JCP-7	
	3	4	5	7
	<10	5-10	10-15	10-15

* Maintenance-free life of JCP-1, in years.

** Computed pavement life in years to a serviceability index of 3.5 using Equation 4.2.

*** Estimated maintenance-free life in years by project staff.

**** 17+ is the traffic maintenance-free life of JCP-12 at a rate of 1×10^6 18-kip ESAL/year. Actual maintenance-free life of the pavement is 24 years at an average traffic application of $17/24 = 0.71 \times 10^6$ 18-kip ESAL/year.

***** Modulus of foundation support used for each environment region is as follows: Wet-freeze: non stabilized k = 30 pci; stabilized k = 175 pci; Wet: non-stabilized k = 40 pci; stabilized k = 200 pci; Dry-Freeze: non-stabilized k = 60 pci; stabilized k = 230 pci; Dry: non-stabilized k = 75 pci, stabilized k = 230 pci.

Table 6.6. Estimated Maximum Maintenance-Free Life for JCP with Slab Thickness of 9 inches. *

Stabilized Subbase	NO		YES	
Load Transfer / Environment	Aggregate	Dowels	Aggregate	Dowels
WET + FREEZE	 4 5	16+ JCP-2** 6 5-10	 5 5-10	 10 10-15
WET	(4+) JCP-5 5 5-10	 7 10-15	 7 10-15	 10 10-20
DRY + FREEZE	 5 <5	 8 5-10	(4+) JCP-9 6 5-10	 10 10-15
DRY	27+ JCP-10 7 10-15	 8 10-15	 9 15-20	 10 15-20

* This slab was 9.5 inches thick.
** See Table 6.5 for description of values.

Table 6.7. Estimated Maintenance-Free Life for JRCP with Slab Thickness of 9 Inches.

Joint Spacing	60-100 Feet		30-60 Feet	
Joint Types / Environment	Contraction	Expansion or Expansion + Contraction	Contraction	Expansion or Expansion +Contraction
WET + FREEZE	6 <5	(<4) JRCP-15 (<5) JRCP-16 (4+) JRCP-17 6 <5	<12 JRCP-7 (24+)JRCP-11 6 5-10	6 5-10
WET	6.5 <5	6.5 5-10	6.5 5-10	(4+)JRCP-1 6.5 10-15
DRY + FREEZE	7 <5	7 5-10	7 5-10	7 <10
DRY	8 <5	8 5-10	8 10-15	8 10-15

See Table 6.5 for description of values.

Table 6.8. Estimated Maximum Maintenance-Free Life for JRCP with Slab Thickness of 10 Inches. *

Joint Spacing	60-100 Feet		30-60 Feet	
Environment / Joint Types	Contraction	Expansion or Expansion + Contraction	Contraction	Expansion or Expansion + Contraction
WET + FREEZE	(<7)JRCP-12 <5 JRCP-18	23+ JRCP-5 9+ JRCP-14	(<12)JRCP-7 (8+)JRCP-10	
	13	13	13	13
	<5	5-15	5-10	5-15
WET				
	13	13	13	13
	<5	10-15	10-15	10-15
DRY + FREEZE				
	14	14	14	14
	<5	5-15	10-15	10-15
DRY				
	14	14	14	14
	<5	15-20	15-20	15-20

See Table 6.5 for description of values.

Joint Spacing:	JCP spacing 12 to 18 ft, and JRCP as indicated
Subgrade:	similar to soil at AASHO Road Test ($k_g \approx 60$ pci), no swell problem
Serviceability:	P1 = 4.5, P2 = 3.5 (initial and final, respectively)
Environment:	as previous defined for wet-freeze, wet/non-freeze, dry-freeze, and dry/non-freeze regions
Concrete Durability:	no "D" cracking deterioration

The design variations for JCP include subbase type (stabilized and nonstabilized), slab thickness (8 and 9 inches), load transfer (aggregate interlock and dowels), and environmental region. The design variations for JRCP include slab thickness (9 and 10 inches), joint spacing (60-100 ft and 30-60 ft), joint types (contraction and expansion), and environmental region. Each cell of the tables contains three estimates, in years, of the maintenance-free life of the pavements. The top estimate, if available, is the actual maintenance-free life in years for one or more projects included in the field study that "fit" into the cell (i.e., similar slab thickness, traffic, subbase, joint type, etc.). Sometimes this number is in parentheses, which indicates "traffic life" in years, in terms of 18-kip ESAL at a rate of 1×10^6 per year for those projects where actual traffic is less than 1×10^6 per year. For example, a project 10 years old with approximately 10×10^6 ESAL would be listed as 10, but a project 10 years old with 5×10^6 ESAL would be listed as "(5)". The center estimate is computed life using the AASHO Performance Model (Equation 4.2 in Reference 1.) to a serviceability index of 3.5 at a rate of 1×10^6 18-kip ESAL per year. The bottom estimate is the range of probable maintenance-free performance life determined by the project staff based upon the experience gained from the field surveys and interviews with local pavement engineers.

The following is a summary of conclusions related to the maintenance-free life estimates given in Tables 6.5 to 6.8:

1. The three sets of estimate in each cell are generally in agreement. Consider, for example, in Table 6.5 for an 8-inch JCP slab, nonstabilized subbase, with dowels, and located in a wet-freeze region. Actual performance of JCP-1 shows less than 5 years before maintenance was required, the computed estimate shows 3 years, and the project staff estimate is less than 5 years. An example of fair agreement for JRCP is found in Table 6.8 for a 10-inch slab, 60-100 ft joint spacing, contraction joints, and a wet-freeze region. Here, two projects (JRCP-12, 18) had an actual maintenance-free life of less than 7 years, the computed life expectancy is 13 years, and the project staff estimate was less than 5 years.

2. There are some instances of significant disagreement, particularly between the AASHO equation and the computed estimate in non-freeze regions. For example, consider Table 6.5 for JCP with 8-inch slab, stabilized subbase, aggregate interlock load transfer, and dry/non-freeze region. Two projects (JCP-6 and JCP-7) showed actual maintenance-free lives of 10 and more than 32 "traffic" years, respectively, while the computed life was 5 years, and the staff estimate was 10 to 15 years. The widest disagreement was for non-freeze regions where Equation 4.2 (1) appears very inadequate (it was derived for a wet-freeze climate of Illinois).

3. The maximum maintenance-free life for conventional JCP in each environmental region is estimated as follows for a typical design consisting of a 9-inch slab, stabilized or non-pumping subbase, dowels for load transfer in freeze and in wet regions, and heavy traffic of 1×10^6 18-kip ESAL/year:

Environment	Maintenance-Free Life, Years
Wet-freeze (dowels)	10-15
Wet/Non-freeze (dowels)	15-20
Dry-freeze (dowels)	10-15
Dry/Non-freeze	15-25

4. The maximum maintenance-free life for conventional JRCP in each environmental region is estimated as follows for a typical design consisting of a 10-inch slab, 30-50 ft joint spacing, contraction or expansion joint types, doweled joints, and heavy traffic of 1×10^6 18-kip ESAL/year:

Environment	Maintenance-Free Life, Years
Wet-freeze	5-15
Wet/Non-freeze	10-15
Dry-freeze	10-15
Dry/Non-freeze	15-20

Summary JCP and JRCP

The conventional design for JCP that has shown the longest maintenance-free life under heavy traffic includes a non-pumping granular or stabilized subbase, randomly spaced and skewed transverse joints (less than 18 ft maximum spacing), slab thickness of 9 inches or more, and use of doweled load transfer devices (especially in cold climates). This design, if properly constructed, should provide maintenance-free life from 10 to 20 years depending on environmental deterioration effects of the PCC under heavy traffic.

The conventional design for JRCP that has shown the greatest maintenance-free life under heavy traffic includes contraction joint spacings less than about 50 ft or expansion joint spacing of less than 100 ft, corrosion proof dowels, non-pumping granular subbase, and slab thickness of at least 10 inches. This design, if properly constructed, should show maximum maintenance-free life from 5 to 20 years depending on environmental deterioration of the PCC under heavy traffic.

6.3 CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (CRCP)

Conventional high traffic volume CRCP is considered as that which is typical of past and current design and construction practice. It is typically comprised of 8"-9" thick slabs with 0.5 to 0.7 percent longitudinal reinforcement of either deformed welded wire fabric or deformed reinforcing bars. Subbases are either stabilized aggregates (with asphalt, cement, or lime) or non-stabilized granular materials.

Field Performance

Results from the field visits which included three (WF, W, D) of the four environmental regions included 7 out of a total 12 projects surveyed with maintenance-free performance. Their maintenance-free life ranged from 5 to 19 years, with average traffic loadings of 0.50 x 10^6 18-kip ESAL per year in the heaviest traveled lane. Only two of the maintenance-free CRCP projects were subjected to heavy traffic; CRCP-2 with 1 x 10^6 18-kip ESAL per year and CRCP-4 with 0.86 x 10^6. Traffic loadings ranged from about 0.3 to 4.0 x 10^6 per year for pavements having ADTs ranging from 40,000 to 266,000.

Project data are summarized in Table 6.9 showing the structural components, actual maintenance-free life, distress requiring maintenance, and maintenance activities. One important conclusion drawn from these data is that all projects with a slab thickness of 9 or 10 in. and deformed rebar reinforcement provided long term maintenance-free performance. One exception to this general conclusion is CRCP-12 which had a slab thickness 10 inches, but required maintenance within 10 years,

Table 6.9. Some Characteristics of CRCP Relating to Performance.

Environment	Project No.	State	Slab Thickness (inches)	Longitudinal Reinforcement(%)	Average 18-kip ESAL/year x 10^6	Maintenance-Free Life (years)	Distresses Requiring Maintenance**	Maintenance Received
WET + FREEZE	1	IL	8	0.613	4.00	2-4	IC, LF, SR, B	Patching
	2	IL	10	0.613	1.03	10+		
	3	MN	9	0.619	0.37	5+		
	4	MI	9	0.681	0.88	9+		
	12*	NJ	10	0.737	1.19	< 10	IC, CS, SR	Patching
WET	5	TX	8	0.511	0.78	8-9		
	6	TX	8	0.590	0.16	< 12	IC, LC, CS, LF	Patching
DRY	7	TX	8	0.511	0.22	12+		
	8	TX	8	0.511	0.31	< 13	LF, P	Patching
	9	TX	8	0.511	0.25	13+		
	10	TX	8	0.511	0.30	< 8	LF, P	Patching
	11	TX	9	0.614	0.31	19	CS	Overlay

* This project had unusual reinforcement design which resulted in early rupture that required maintenance. Also, this pavement was subjected to freeze-thaw environment but not to the same extent as pavements in northern Illinois, Minnesota, and Michigan.

** CS: Crack Spalling LF: Localized Failure IC: Interconnecting Cracking SR: Steel Rupture
P: Pumping LC: Longitudinal Cracking B: Blowup

Joint sealant and shoulder distress are not considered in the maintenance activities.

possibly caused by the unusual reinforcement pattern which ruptured early. The longest maintenance-free life was 19 years for CRCP-11 located in Fort Worth, Texas, but this project had an average traffic application rate of only 0.31×10^6 18-kip ESAL per year. Other projects such as CRCP-2 and 4 which were only 10 years old, but carry much heavier traffic were also in excellent conditions at the time of the survey.

The major distresses in high traffic volume CRCP include localized failure, steel rupture interconnecting cracks and crack spalling. All distresses and their causes are discussed in detail in Chapter 3.

Factors Affecting the Maintenance-Free Life of CRCP

There are many factors affecting the performance of CRCP and consequently its maintenance-free life. A very important feature of CRCP is the formation of hairline transverse cracks shortly after construction and probably resulting from a drop in temperature and the drying shrinkage of the concrete. The spacing and width of these cracks apparently have significant effect on the performance of CRCP. While these cracks eliminate the need for regular joints, the reduction of bending stiffness as a result of these cracks can lead to increased deflection and high slab and subgrade stresses under load.

Factors affecting maintenance-free life of CRCP can be divided into several categories which are discussed briefly in the following sections.

System Stiffness: From the field survey and literature review discussed in Chapter 3, the stiffness of the pavement system is found to be a very significant factor affecting its maintenance-free life. This stiffness includes the combined effect of slab thickness; concrete modulus of elasticity; longitudinal reinforcement, both amount and depth; and foundation support. Slab thickness is probably the most important factor (7). Total system stiffness decreases due to thermal cracks which result in higher deflections and high slab stresses (7) which may cause further structural distresses. This point is illustrated by Figure 6.7 which shows how maximum deflection increases rapidly as the slab stiffness decreases due to transverse cracking.

Typical CRCP systems with an 8-inch slab and stabilized or non-stabilized subbase (for example, CRCP-1, 5, 6, 8, and 10) have not performed maintenance-free for long periods of time undery heavy traffic, due in part to inadequate system stiffness resulting in localized failures. Anticipated maintenance-free life is less than 10 years for these designs. Pavements with greater system stiffness, however, have shown much better maintenance-free performance (CRCP-2, 4, and 11).

Construction-Environment: Such factors as inadequate lap length and poor construction joint techniques may result in bond failure and separation of the longitudinal reinforcing steel. Whenever this occurred on the projects surveyed, maintenance was applied as soon as practical (CRCP-1, 12). This maintenance was spurred by a rapid deterioration of the crack under heavy traffic (spalling and faulting). Lack of concrete consolidation is an important factor which can lead to bond

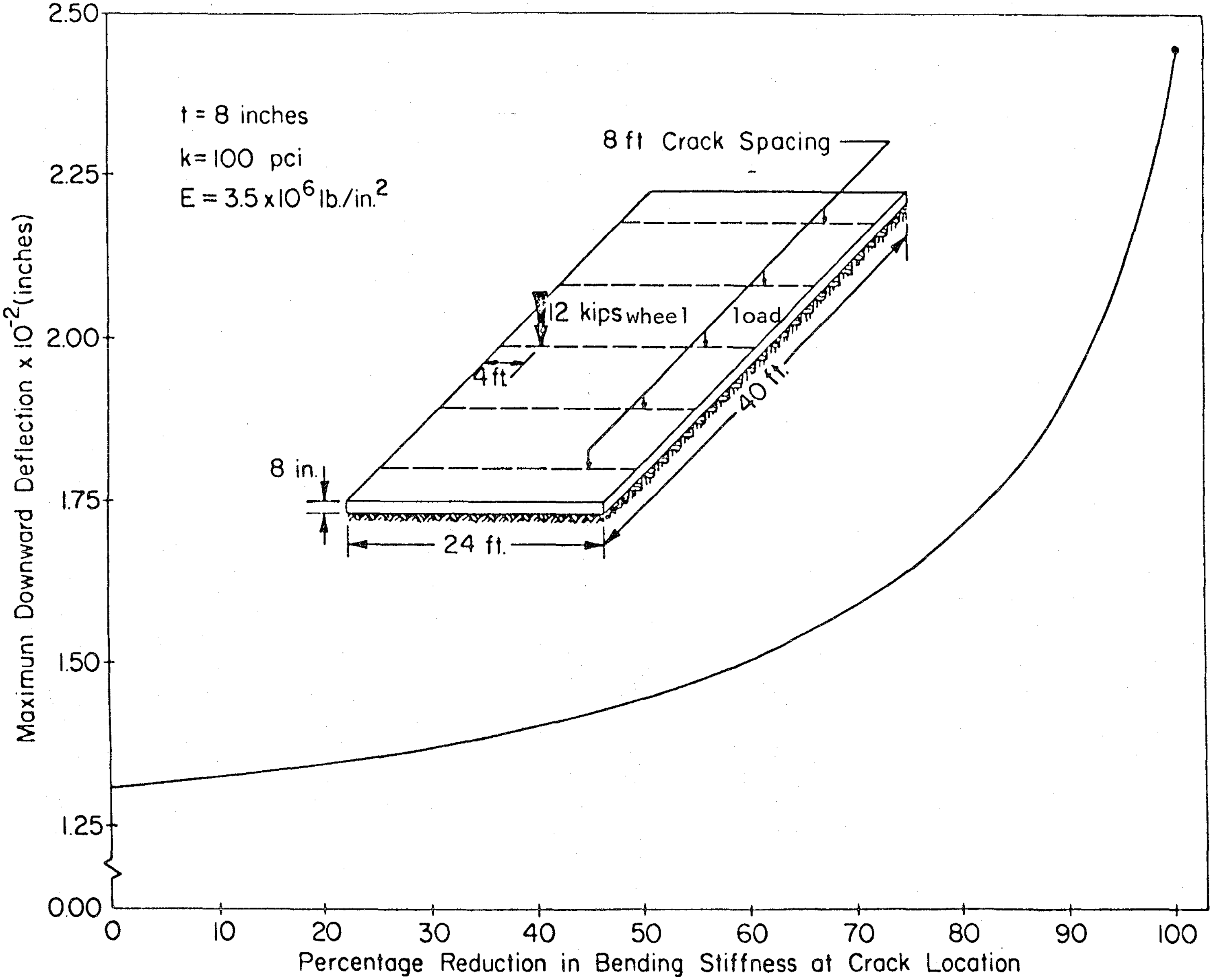

Figure 6.7. Crack severity versus deflection for CRCP (Ref. 7).

failure after only relatively few traffic applications and results in localized failures requiring maintenance. The maintenance-free life of CRCP where such failures occur is very short, usually less than 10 years. Other factors which affect the performance of CRCP are temperature and moisture change during the construction. These environmental factors influence the cracking spacing significantly (6), and therefore have an effect on the maintenance-free life, though no quantitative conclusions can be made at this time on the extent of this effect.

Traffic-Environment: Both traffic and environmental factors have a significant effect on CRCP maintenance-free life. These factors include load magnitude, configuration, lateral distribution, and repetitions of loads; cyclic changes in environmental conditions (temperature and moisture); volumetric changes of concrete (shrinkage and expansion); and loss of foundation support. The interaction of environmental factors with traffic can cause such types of distress as edge pumping and subsequent localized failures which occurred on about one third of the projects surveyed.

Inadequate subsurface drainage allows free moisture to be trapped in the pavement section which, under the action of traffic, causes accelerated pavement failure. The combined effects of traffic and environment result in a fatigue-type deterioration in conventional CRCP which will probably limit its maintenance-free life to less than 15 years, especially under heavy traffic.

Environment-Material: The influence of environmental-material factors affecting concrete pavements were previously discussed under JCP and JRCP. These factors may also limit CRCP maintenance-free life to less than 10 years. Corrosion of steel reinforcement is an added problem in wet/freeze-thaw regions due to the use of large amounts of deicing salts. Engineers in several states expressed concern about this problem and believe it to have the potential for limiting the maintenance-free life of CRCP. More study is clearly needed on this aspect of CRCP.

Variability: The influence of construction and material variations on the maintenance-free life of CRCP is similar or more critical than for jointed concrete pavements which were discussed previously.

Expected Maintenance-Free Life

The same general approach as used for determining the maintenance-free life of JCP and JRCP is also applied to CRCP including:

1. Use of best available predictive model,
2. Actual project life based on data from the field survey, and
3. Estimate of project staff based on general performance found in the various climatic regions for particular designs.

A predictive model (Equation 4.9) developed by Hudson and McCullough (61) from AASHO Road Test data is used to estimate the maintenance-free life using the serviceability-performance approach. Results from Chapter 5 show that a terminal serviceability of from 3.0 to 3.5 is reasonable as limiting design criteria. The maintenance-free life may be roughly estimated for specific designs, including the effect of slab thickness, subbase type, and environmental region. The reasonableness of the predictive Equation 4.9 can be seen in Table 6.10 which summarizes the predicted maintenance-free life and actual maintenance-free life of the 12 CRCP projects included in the field survey. Results indicate that the predicted maintenance-free life is close to the actual values for most projects with the exception of CRCP-6 and 12.

Table 6.10. Predicted and Actual Maintenance-Free Life of CRCPs

Environment	Project No. - State		Present (Years)	Maintenance-Free Life Actual	Maintenance-Free Life Predicted*
WET and FREEZE	1	IL	9	2-4	1.1
	2	IL	10	10+	15.6
	3	MN	5	5+	21.8
	4	MI	9	9+	9.9
	12	NJ	27**	<10	18.4
WET	5	TX	8	8-9	12.2
	6	TX	12	<12	28.6
DRY	7	TX	12	12+	12.9
	8	TX	13	<13	12.9
	9	TX	13	13+	19.8
	10	TX	8	<8	11.0
	11	TX	19	19	23.2

*Based on Equation 4.9.

**This pavement experienced some freeze-thaw activity but not to the same degree as pavements in northern Illinois, Minnesota and Michigan.

The major structural failure of CRCP-6 is localized punchouts which are believed to be mainly the result of lack of consolidation of the concrete in localized areas. Although a small portion of the pavement failed in less than 12 years due to this localized distress, it is believed that this project would have lasted as long as predicted with good quality control. The reason for the early failure of CRCP-12 is that an unusual reinforcement pattern was used which may have caused steel rupture. Hence, this predictive model provides a tool for determining the approximate maximum maintenance-free lives of CRCP, but it clearly has limitation as discussed in Chapter 4.

A summary of estimates of the maintenance-free life of various designs of CRCP is given in Table 6.11. These estimates are made for specific conventional design criteria with the same constants and assumptions for traffic, construction, subgrade, environment, serviceability, and concrete durability as used in the estimates for JCP and JRCP. The design variables include slab thickness (8, 9, and 10 inches*), subbase type (stabilized and non-stabilized), and environmental region. Each cell in Table 6.11 contains three estimates of the maintenance-free life of these pavements. The top estimate, if available, is the actual maintenance-free life, in years, of one or more projects included in the field study that fit into the cell. When this number is in parentheses, it indicates "traffic life" in years in terms of 18-kip ESAL at a rate of 1×10^6 per year instead of actual pavement life. The estimate

*A 10-inch CRCP is not considered as "conventional" herein since very few have been built, but is included for comparison purposes only.

Table 6.11 Estimated Maximum Maintenance-Free Life for CRCP.

Slab Thickness	8"		9"		10"	
Subbase / Environment ***	Nonstabilized	Stabilized	Nonstabilized	Stabilized	Nonstabilized	Stabilized
WET and FREEZE	 3 <5	 5 <5	(8+) CRCP- 4 (2+) CRCP- 3 5 5-10	 9 10-15	10+ CRCP- 2 10 10-15	(16+) Dan Ryan ** 16 15-20
WET	(<2) CRCP- 6 3 <5	(6) CRCP- 5 5 5-10	 6 5-10	 9 10-20	 11 10-20	 16 15-25
DRY and FREEZE	 3 <5	 5 5-10	 6 10-15	 9 10-15	 12 10-20	 17 15-20
DRY	(3+) CRCP- 7 (<4) CRCP- 8 3 <5	(3+) CRCP- 9 (<2.5) CRCP-10 5 5-10	(6) CRCP-11 7 10-15	 9 15-20	 12 15-20	 17 20-25

* See Table 6.7 for description of values.

** This project is not included as one of the Field Survey Projects but is 4 years old carrying 4×10^6 18-kip ESAL per year and is in maintenance-free condition (See Figure 6.8).

*** The foundation modulus of support are as follows: Wet and Freeze: nonstabilized k= 30 pci, stabilized k= 175 pci; Wet: nonstabilized k= 40 pci, stabilized k= 200 pci; Dry and Freeze: nonstabilized k= 60 pci, stabilized k=230 pci; Dry: nonstabilized k= 75 pci, stabilized k= 230 pci.

listed in the mid range is the computed life in years using Equation 4.9 to a terminal serviceability of 3.5. The bottom estimate is the range of probable maintenance-free life determined by the project staff based upon the experience gained from the field surveys.

The following is a summary of conclusions related to the maintenance-free life estimates given in Table 6.11.

1. The three sets of estimates in each cell are generally in agreement. For example, consider a CRCP with an 8-inch slab, stabilized subbase, and located in a wet/non-freeze region. The actual performance of CRCP-5 which fits this description shows an actual maintenance-free life of 8 years, but total 18-kip ESAL of 6.25×10^6, hence a "traffic life" of 6 years at 1×10^6 per year. The computed life is 5 years, and the estimated life is 5 to 10 years. Another example of agreement is for a CRCP with a 10-inch slab and non-stabilized subbase in a wet-freeze region. CRCP-2 has lasted 10 years with traffic averaging 1.03×10^6 ESAL per year with maintenance-free performance, and is expected to last several more years as indicated by 10^+. The computed value is 10 years, and the project staff estimated life is 10 to 15 years.

2. There are some cells in which there is a disagreement in estimates shown. In general, the computed life is less than the estimate by the project staff, particularly for the thicker pavements.

3. The maximum maintenance-free life for conventional CRCP in each environmental region is estimated as follows for an 8 and 9-inch slab with a stabilized subbase under 1×10^6 18-kip ESAL/year.

Environment	Maintenance-free Life, Years 8 ins.	9 ins.
WF	< 5	10-15
W	5-10	10-20
DF	5-10	10-15
D	5-10	15-20

The 10-inch CRCP is not considered as conventional herein, even though it has been constructed on two freeways in Chicago, as it has not been used anywhere else in the United States. Still, an example is provided to illustrate the seemingly high potential of this type pavement.

One 10-inch CRCP carrying very heavy traffic is the Dan Ryan Expressway (I-90) in Chicago. The four truck lanes were reconstructed in 1971 and each of the truck lanes carries approximately 7100 trucks per day. Total ADT is 260,000 vehicles. The total 18-kip ESAL per year is estimated at 4×10^6 for each truck lane. The reconstructed truck lane section included a 10-inch CRCP and 4 inches of asphalt-treated subbase. The pavement has been in-service for four years and shows no sign of distress even though approximately more than 16×10^6 ESAL application have been applied. The pavement is in excellent condition structurally as shown in Figure 6.8.

6.4 FLEXIBLE PAVEMENTS

Flexible pavements defined as conventional are those typical of past and current design and construction practice with compositions which vary widely. Typical compositions are as follows: asphalt concrete surface of 3 to 6 inches, a non-stabilized or stabilized granular base of 8 to 12 inches, and one or more granular subbases usually

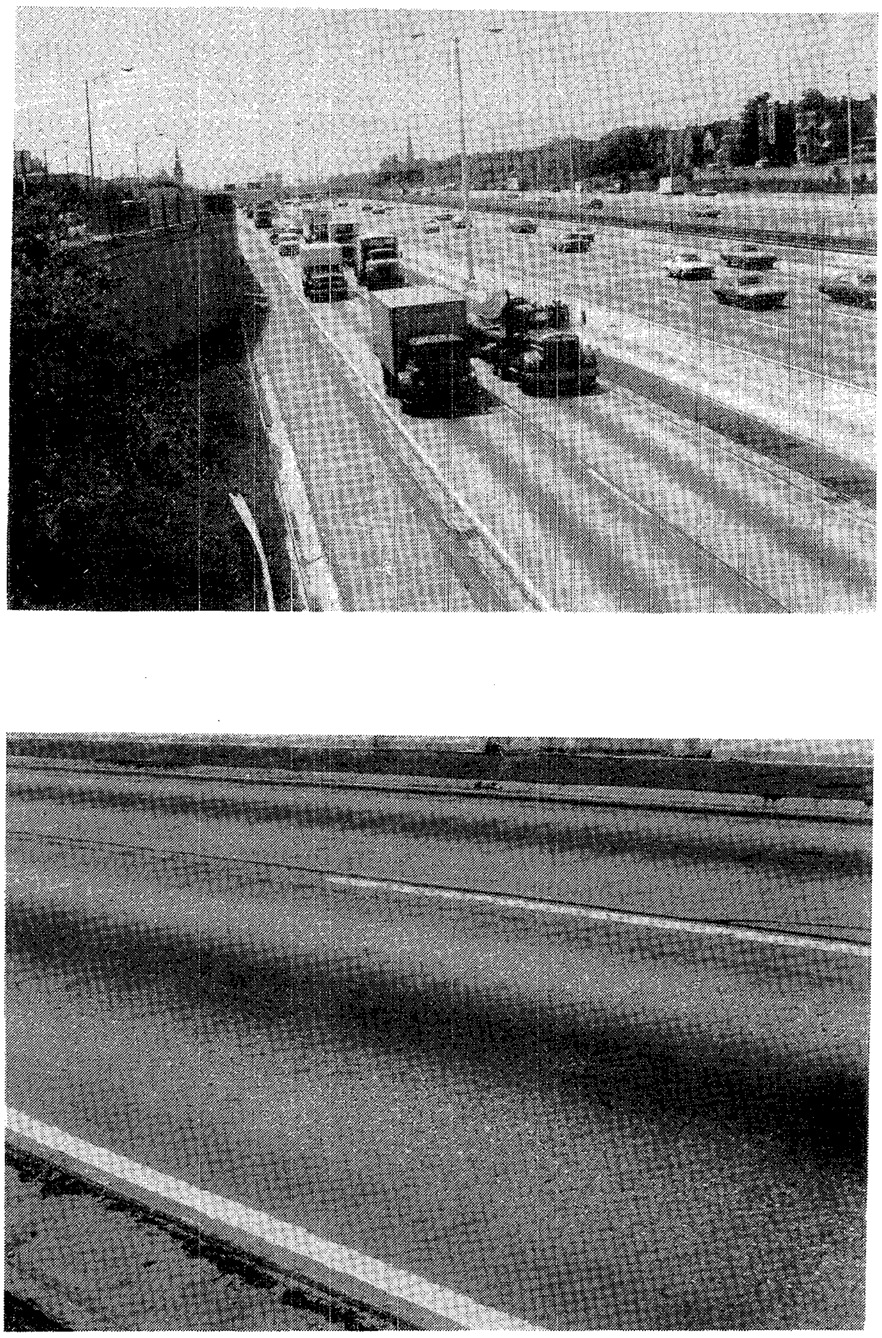

Figure 6.8. Maintenance-free CRCP (Dan Ryan Expressway, Chicago, Illinois), with a traffic of 4×10^6 18-kip ESAL/year (lane, 10 inches thick constructed 1971).

from 8 to 20 inches.

Field Performance

Results from the field survey of 19 flexible pavement projects had 9 with maintenance-free performance. These 9 pavements ranged in age from 8 to 23 years with an average traffic loading of 0.26 x 10^6 18-kip ESAL per year in the heaviest traveled lane. Traffic loadings ranged from 0.40 to 0.54 x 10^6 18-kip ESAL/year for flexible pavements having ADTs ranging from 30,000 to 138,500. It should be noted that this traffic loading is considerably less than for all other pavement types, which averaged from 0.5 to 1.1 x 10^6 18-kip ESAL per year.

The major types of distress which significantly affect the maintenance-free lives of conventional flexible pavements are alligator (or fatigue) cracking, transverse cracking, longitudinal cracking, and rutting. A summary of project data grouped by environmental region from the field survey is shown in Table 6.12 including the maintenance-free life, major types of distress requiring maintenance, and maintenance performed.

The following maximum maintenance-free lives are obtained from these data in each environmental region considering the actual traffic loadings:

Environment	Maintenance-Free Life, Years
WF	5-23+
W	7-23+
DF	9
D	10-10+

Since the flexible projects were not subjected to as heavy a traffic volume as the JCP, JRCP, CRCP, and COMP, these lives are not directly comparable.

Table 6.12. Maintenance-Free Life of Flexible Pavements

Environment	Flex Project No. - State		Annual Maint. - Free Life of the project in years	Average 18-kip ESAL/year x10^6	Distress that Required Maintenance	Maintenance Received
WET and FREEZE	1	NH	5-15+**	0.52	TC, LC	CF
	3	MN	5-10	0.15	TC, AC, LC	P, CF
	4	MN	5-14+	0.15	TC, LC	CF
	5	MN	5-14+	0.13	TC	CF
	6	MN	5-9+	0.65	TC	CF
	7	IL	5	0.39	AC	P
	8	IL	13	0.40		
	9	IL	13+	0.39		
	10	IL	8	0.47	TC, AC	P
	2	NJ	23	0.54	Rutting	P*
WET	11	WA	15+	0.12		
	12	WA	12+	0.03		
	18	GA	23+	0.17		
	19	GA	7	0.40	AC	
DRY and FREEZE	16	UT	9	0.34		
DRY	13	CA	10	0.46	AC	P, CF
	14	CA	10	0.25		
	15	CA	10+	0.41		
	17	AZ	11	0.43	AC	P, CF

T.C. = Transverse Cracking
A.C. = Alligator Cracking
L. C. = Longitudinal Cracking

P. = Patching
C.F. = Crack Filling

* Received minor amount of maintenance.

** This lower value is the maximum Zero-Maintenance life of the project due to transverse crack filling.

Factors Affecting Maintenance-Free Life

The major factors that affect the maintenance-free life of flexible pavements include traffic, environment, materials, construction, and variability. Most of these factors are discussed in detail in Chapter 3, and will only be briefly discussed in this section to summarize their effect on maintenance-free life.

1. Traffic: The effect of repeated loadings of heavy vehicles is particularly significant for the maintenance-free life of flexible pavements. A structural deterioration under repeated loads is, in most cases, the primary factor that limits the life of conventional flexible pavement. Structural distress due to the repeated traffic loadings takes the form of fatigue (alligator) cracking and/or permanent deformation of the pavement layers. The fatigue analysis presented in Chapter 3 for flexible pavements illustrates the significant effect of traffic loadings on pavement distress and especially fatigue cracking. Based upon both theory and the results of the field survey, the maintenance-free life of most conventional flexible pavements is limited to less than 15 years (probably 10 in most instances) when subjected to heavy traffic loading.

2. Environment: Environmental factors affect the maintenance-free life of flexible pavements by cyclic freeze-thaw, low temperature shrinkage, cyclic thermal fatigue, excess moisture, and swelling soils. In certain regions of the U. S., these environmental factors become the dominant factor controlling maintenance-free life of flexible pavements. For

example, in heavy freeze regions, the low temperature cracking of asphalt concrete surfaces is of particular significance in establishing the maintenance-free life. Another example is the effect of swelling soils in portions of the country which limits the maintenance-free life.

3. Materials: Several material properties affect the maintenance-free life of flexible pavements including asphalt aging (hardening and becoming brittle), asphalt stripping, degradation of aggregates due to abrasion, and durability of stabilized aggregates, due to freeze-thaw combined with the use of deicing salt, etc. In specific environmental regions and certain localized areas, these factors control the maintenance-free life of flexible pavements. The factor having the greatest overall influence is asphalt aging. This phenomenon may reduce the fatigue life of the asphalt mix and increase low temperature cracking in freeze regions. The magnitude of the effect of asphalt aging on maintenance-free life of flexible pavements varies depending on asphalt type, film thickness, voids, climate, etc., as discussed in Chapter 3. The disintegration of cement treated aggregate bases in freeze regions due to freeze-thaw and use of deicing chemicals is believed to seriously limit maintenance-free life in these regions.

4. Construction: The major errors or deficiencies which occur during construction and affect maintenance-free life of flexible pavements are built-in surface roughness, poor quality control, inadequate layer thickness, inadequate compaction, and poor construction of the lane paving joint. Any of these deficiencies can have significant effect

on and greatly reduce the maintenance-free life of a flexible pavement. For example, longitudinal cracking occurred at the paving lane joint on over one-half of the projects surveyed which, in itself, could reduce significantly maintenance-free life on an otherwise maintenance-free pavement.

5. Variability: There are many variations and uncertainties associated with load, materials, and climate which can cause localized distress requiring maintenance. These have been identified and analyzed by several researchers (31, 32, 33, 112). These variations include those occurring along a pavement (internal strength, resiliency, durability) and differences between parameter values assumed in design (i.e., load strength, etc.) and those attained "as constructed." These variations can significantly reduce maintenance-free life.

Expected Maintenance-Free Life

The same general approach previously used to determine the maintenance-free life of other pavement types is used for flexible pavements. The expected life of conventional flexible pavement is estimated by the following approaches:

1. Use of best available predictive models;
2. Actual project life based on data from the field survey; and
3. Estimate of project staff based on general performance found in the various climatic regions.

First, an analysis is made of flexible pavements to show that it is possible to analytically predict fatigue distress. The results of a fatigue analysis which was mainly described in Chapter 3 for 14 flexible pavements located across the U. S. is shown in Figure 6.9. The accumulated fatigue damage, D, as summarized in Table 3.21 is plotted versus the cracking index for each pavement. Cracking index here includes Class 1, 2, and 3 alligator or fatigue cracking measured in units of $ft^2/1000\ ft^2$. As fatigue damage becomes greater than approximately 0.6, the various projects exhibit a greater level of fatigue cracking. Another interesting aspect to this plot is that all projects having accumulated fatigue damage greater than 0.6 have either received maintenance for fatigue cracking or were in need of immediate maintenance at the time of the survey. It must be noted that some of the projects shown in Figure 6.9 had received maintenance due to other types of distress such as FLEX-4 which had severe thermal transverse cracking, but this project and others like it are indicated as maintenance-free here because there was no fatigue cracking. All projects, except one, having fatigue damage less than the 0.6 were maintenance-free for fatigue distress. These results emphasize the importance of traffic loading on the maintenance-free life of flexible pavements, and that this effect can be estimated roughly using a mechanistic approach.

The predictive model (Equation 4.14) derived from the results of the AASHO Road Test is used to estimate flexible pavement maintenance-free life using the serviceability-performance approach. A limiting serviceability value of 3.5 is used as recommended in Chapter 5. If this criteria is used along with Equation 4.14, the maintenance-free life may be roughly estimated for certain pavement designs including crushed stone base, asphalt treated base, and cement treated base located in different environmental regions.

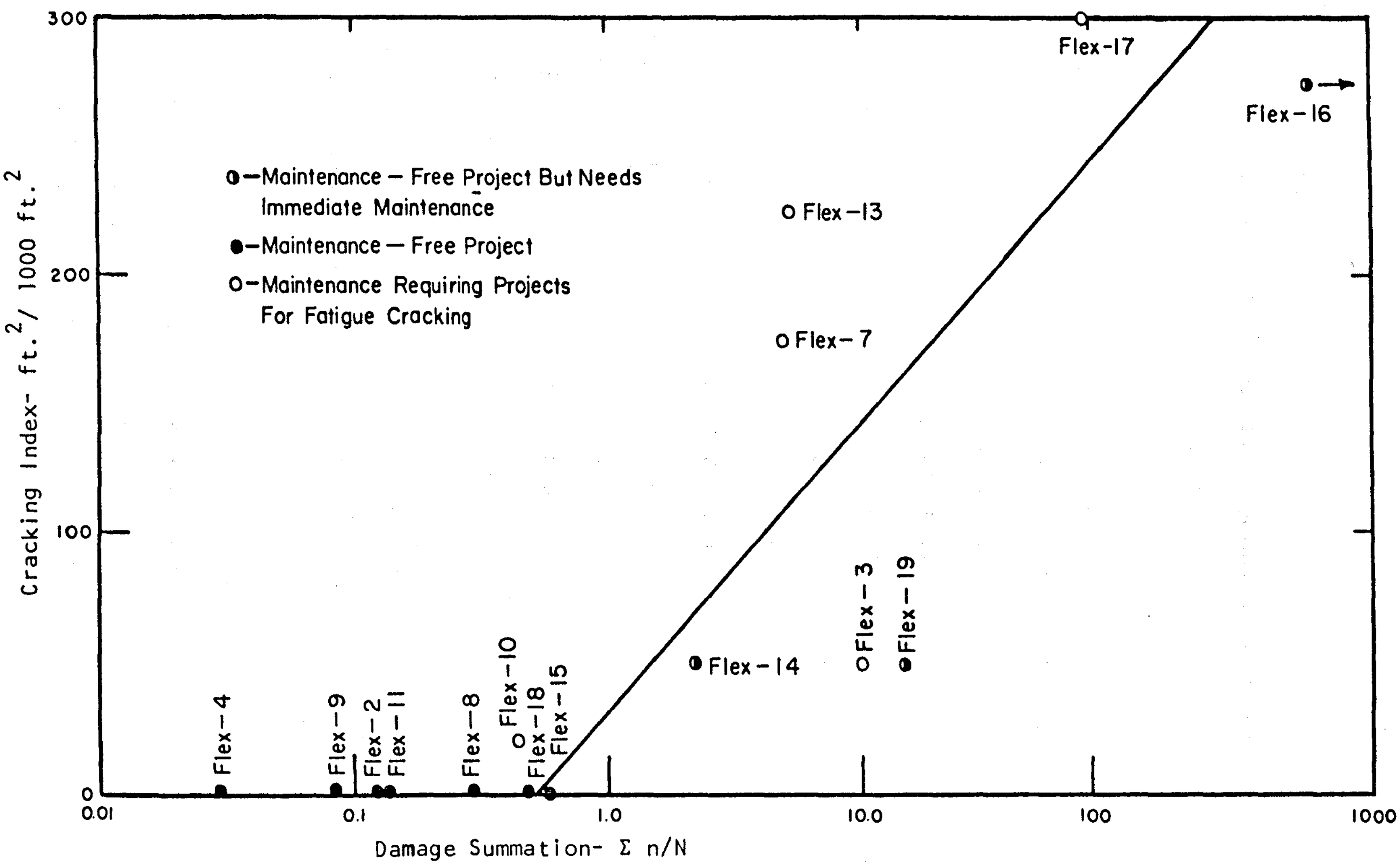

Figure 6.9. Accumulated Fatigue Damage versus Cracking Index for Flexible Pavements

A summary of estimations of the maintenance-free life of various designs is given in Table 6.13. These estimates are made for specific conventional design situations with the same design constants and assumptions as for JCP, JRCP, and CRCP, and the following additional values:

Traffic: heavy traffic at 0.7×10^6 18-kip flexible ESAL/year in design lane (this traffic level computed using flexible equivalencies is equivalent to 1.0×10^6 18-kip ESAL/year computed using rigid equivalencies, i.e. flexible x 1.5 ≈ rigid for the same traffic load distribution and volume)

Structural Section: 5 ins. asphalt concrete surface

10 ins. crushed stone, asphalt treated, or cement treated base

10 ins. sand-gravel subbase

Environment: The regional factors for the four climatic regions are as follows:

WF	1.0
W	0.7
DF	0.9
D	0.5

Each cell contains three estimates of the maintenance-free life of the pavement. The estimate given in the top group of each cell is the actual maintenance-free life in years for one or more projects included in the field study that generally fit the design criteria. The number of parentheses indicates "traffic life" in years at a rate of 0.7×10^6 18-kip ESAL/year for those projects where actual traffic is less than 0.7×10^6 per year. The center estimate is computed life using Equation 4.14 to

Table 6.13. Estimated Maximum Maintenance-Free Life of Flexible Pavements.*

Environment \ Base	CS	ATB	CTB
WET + FREEZE R = 1.0	(<5) FLEX-1 ** (<5) FLEX-7	(7+)FLEX-8,(7+)FLEX-9 (<5)FLEX-6,(18)FLEX-2	(<5) FLEX-10
	1 ***	14	4
	<5 ****	5-15	< 5
WET R = 0.7		(2+) FLEX-11 (6+) FLEX-18	(<4) FLEX-19
	3	20	6
	<5	10-15	5-10
DRY + FREEZE R = 0.9			(3) FLEX-16
	1	16	4
	<5	5-15	<10
DRY R = 0.5	(3+) FLEX-14 (<4) FLEX-17		(<8) FLEX-13 (5+) FLEX-15
	3	27	7
	5-10	10-15	5-10

* The layer-thicknesses are 5 inches of AC surface, 10 inches of base material (either CS, ATB, or CTB), and 10 inches of gravel-sand subbase.

** Actual project maintenance-free life, years (if in parentheses the value is traffic life in years at 1×10^6 18-kip ESAL/year).

*** Computed life to serviceability index of 3.5 from Equation 4.14, years.

**** Estimated maintenance-free life by project staff, years.

a terminal serviceability index of 3.5. The bottom estimate is the range of probable maintenance-free life determined by the project staff based upon the experience gained from the field surveys.

The following summarizes conclusions related to the life estimates given in Table 6.13:

1. The three sets of estimates in each cell are generally in agreement. For example, consider a flexible pavement with a cement treated base located in a dry/non-freeze region. Two projects (Flex 13 and 15) are located in this region with approximately the same structure, and provide estimates of actual life of < 8 and 5+ years, respectively. The computed life is 7 years, and the estimated life by the project staff is 5 to 10 years. The relatively short life estimated for the crushed stone and cement treated bases compares well to the performance of sections of similar composition at the AASHO Road Test. For example, AASHO Road Test Section which had 6 inches of AC surface, 9 inches of crushed stone base, and 16 inches gravel-sand subbase lasted only 2,900,000 18-kip ESAL to a serviceability of 3.5. Hence, its maintenance-free life would be approximately 4-5 years as a major freeway pavement which is comparable to the estimates given in Table 6.13.

2. The expected maximum maintenance-free life of conventional flexible pavements located in the four environmental regions are as follows for asphalt treated base which provided the best performance:

Environment	Expected Maintenance-Free Life, Years
WF	5-15
W	10-15
DF	5-15
D	10-15

The shorter life of 5 years as indicated for flexible pavements located in freeze regions is due to expected low temperature cracking of the AC surface.

6.5 COMPOSITE PAVEMENTS

Conventional composite pavements are difficult to define because so few have been constructed in the U. S., especially for high traffic volume pavements. Based upon projects included in the field survey, they are defined as having a 3 to 6 inch asphalt concrete surface, possibly a granular stress relief course up to 8 inches in thickness between the AC surface and PCC base, a plain portland cement concrete base 6 to 8 inches thick, and a granular subbase 6 inches thick.

Field Performance

Results from the field survey of seven composite pavements are included in Chapters 2, 3, 4, and 5 which provide useful information concerning the maintenance-free lives of composite pavements. Five of these seven showed maintenance-free performance. These 5 pavements ranged in age from 8 to 14 years with an average traffic loading of 0.93×10^6 18-kip ESAL/year in the heaviest traveled lane. Traffic loadings which were extremely heavy ranged from 0.78 to 2.00×10^6 18-kip ESAL per year for composite pavements having ADTs ranging from 50,000 to 102,000.

The major distress which usually ended maintenance-free life of the composite pavements was transverse crack spalling and deterioration. Other types of distress observed were rutting, edge cracking, and longitudinal cracking.

Various data obtained from the field survey are summarized in Table 6.14 including: maintenance-free life, distress that required maintenance, and maintenance type that each project received. The maximum maintenance-free lives for these projects range up to 14 years and more under heavy traffic for the best designs in both wet/freeze and wet/non-freeze regions. Similar or better performance is expected in dry/freeze and dry/non-freeze regions.

Factors Affecting Maintenance-Free Life

The major factors which affect the maintenance-free life of conventional composite pavements include traffic, environment, materials, construction, and variability. Many of these factors which affect the life of composite pavements are similar to those which affect flexible pavements, and hence only the aspects of these factors as they apply uniquely to composite pavements will be discussed in this section.

Critical stresses in composite pavement include (1) tensile shrinkage stress in the PCC base, (2) flexural stresses at the bottom of the PCC base, (3) tensile strains at the bottom of the AC surface, but only if a granular stress relief course is used between the AC surface and PCC base, (4) shear stress at the surface of the AC that tends to spall the cracks formed by reflection from the PCC base, and (5) confining stress in the AC surface causing permanent deformation or rutting. The most critical effect of environment and materials on the maintenance-free life is through initial shrinkage cracking of the PCC slab and its reflection through the AC surface. When such cracks reflect to the surface, traffic causes spalling and deterioration of the AC at the crack.

Table 6.14. Some characteristics of maintenance-free life of composite pavements

Project No.-State		Environmental Region	Average 18-kip ESAL X 10^6 per year	Distress Requiring Maintenance	Maintenance Performed	Maintenance-Free Life, years
1	MI	WET-FREEZE	0.86	---	---	9+
2	CAN	WET-FREEZE	0.76	TCS	CF, P	<5
3	CAN	WET-FREEZE	0.76	TCS	CF, P	<5
4	CAN	WET-FREEZE	2.00	---	---	14+
5	IL	WET-FREEZE	0.40	---	---	12
6	NJ	WET-FREEZE	0.77	---	---	11+
7	WA	WET	0.15	---	---	8+

This type distress caused "severe" damage on two pavements, and "major" to "moderate" damage on all projects exhibiting transverse cracking. This type of distress may limit the maintenance-free life of conventional composite pavement to less than 20 years. For example, the COMP-4 has a 6-inch layer of AC over an 8-inch PCC base, and has lasted 28 x 10^6 18-kip ESAL over a 14 year period, and is still in good condition, although some of the transverse cracks are showing moderate spalling.

The elimination of transverse reflection cracking has been achieved by placing a granular cushion course between the AC surface and PCC base, and using joints at 15 ft intervals in the PCC slab as in COMP-6. The most critical effect that traffic has on the maintenance-free life of this type of design is to cause fatigue or alligator cracking in the AC surface. If this distress occurs, the maintenance-free life would be severely limited. COMP-6 has received approximately 9 x 10^6 18-kip ESAL applications over 11 years, and no sign of fatigue is visible, even though calculations in Chapter 3 indicated possible fatigue distress.

Expected Maintenance-Free Life

The approach used previously to determine the maintenance-free life for other pavement types is used for composite pavements as follows:

1. Use of best available predictive models;
2. Actual project life based on data from the field survey; and
3. Estimate by the project staff based on general performance found in the different climatic regions.

A summary of estimations of the maintenance-free life of various designs is given in Table 6.15. These estimates are made for specific conventional design situations with the same design constants and assumptions as for the other pavement types, and with a traffic loading of 0.7×10^6 18-kip ESAL/year. The major parameters are AC surface thickness (3 and 6 inches), granular relief layer of 5 inches between the AC and PCC (yes or no), and the four environmental regions previously described.

Each cell contains three estimates, in years, of the maintenance-free life of the composite pavement described by the cell. The estimate given in the top position of each cell is the actual maintenance-free life in years for projects included in the field study that meet the general design criteria. The center estimate is computed life using Equation 4.14 to a terminal serviceability index of 3.5. The bottom estimate is the range of probable maintenance-free life determined by the project staff based upon the experience gained from the field surveys.

The following summarizes conclusions related to the maintenance-free life estimates given in Table 6.15.

1. General agreement exists between the three estimates for most "cells." For example, consider a composite pavement with a 3-inch AC surface, without a granular relief course, and located in a wet/freeze region. Three composite pavements (COMP-2, 3, and 5) showed a maintenance-free life of less than 5 to 7 years at a rate of traffic load application of approximately 0.7×10^6 18-kip ESAL/year. The computed estimate is 2 years, and the estimate by the project staff is less than 5 years.

Table 6.15. Maintenance-Free Life of Composite Pavements.*

AC Thickness / G. R. L. / Environment	3 inches		6 inches	
	No	Yes	No	Yes
WET + FREEZE	<5 COMP-2 <5 COMP-2 (7) COMP-2 3 5	(11+)COMP-6 6 5-10	9+ COMP-1** (40+) COMP-4 11*** 10-20****	24 10-20
WET	4 <5	9 5-10	(2+)COMP-7 16 10-20	34 10-20
DRY + FREEZE	3 <5	7 5-10	13 10-20	26 10-20
DRY	6 5-10	11 10-15	23 15-20	47 15-20

* Layer thicknesses are AC surface = 3 and 6 inches, granular relief course = 0 and 5 inches, PCC base = 8 inches, and granular subbase = 6 inches.

** Actual project maintenance-free life, years (if in parentheses the value is traffic life in years at 1×10^6 18-kip ESAL'year).

*** Computed life to serviceability index of 3.5 from Equation 4.14, years.

**** Estimated maintenance-free life by project staff, years.

G. R. L. = Granular stress relief layer.

2. A few cells show some disparity in estimates. For example, consider a composite pavement with a 6-inch AC surface, no granular relief course, and located in a wet/freeze region. COMP-1 shows a maintenance-free life of greater than 9 years, and COMP-4 shows a "traffic life" of 40 years (actual life is 14 years, but at average load applications of 2×10^6 18-kip ESAL/year). The computed life is only 11 years, and the estimate by the project staff ranges between 10 and 20 years.

3. The maximum expected maintenance-free life for conventional composite pavements with a 6-inch AC surface placed over an 8-inch PCC base is as follows:

Environment	Expected Maintenance-Free Life, Years
WF	10-20
W	15-20
DF	10-20
D	15-20

6.6 SUMMARY OF MAINTENANCE-FREE LIVES

A comprehensive analysis has been conducted to determine the maximum maintenance-free life that can be reasonably expected from conventional pavements under high traffic volumes, and located in various environments. The analyses and estimates contained herein relate specifically to conventional pavements, which are defined as typical past and current designs for JCP, JRCP, CRCP, FLEX, and COMP types that have been constructed on high traffic volume pavements in the U. S.

Three approaches were used to estimate the maximum maintenance-free life:

1. Predictive models;
2. Actual project life determined from projects included in the field study; and
3. Estimate life by the project staff based on overall experience gained through field visits.

Summaries of the expected ranges of maximum maintenance-free lives of the five pavement types for each environmental regions and also for various alternative designs of each type are presented in the tables of Chapter 6. An overall summary of maintenance-free life under certain specific conditions for all types is given in Table 6.16. The specific pavement sections used in this table are selected as the "conventional" design for each type that performed best according to the field survey. The specific section for each type is also indicated in Table 6.16. The maintenance-free life of any available pavement project included in the field survey that "fits" the cell is also included. The specific assumptions and conditions for which these estimates are made include:

1. Traffic: Average 18-kip ESAL is 1.0×10^6 per year in heaviest traveled land for JCP, JRCP, and CRCP pavements, and 0.7×10^6 per year for FLEX and COMP pavements.
2. Construction: Good quality with no significant errors or deficiencies that severely limit life.
3. Subgrade: Similar to clay soil of AASHO Road Test Site.
4. Material Durability: No significant "D" cracking of PCC or asphalt stripping of AC materials.

The following summarizes some of the results of this chapter relative to the conventional pavements described in Table 6.16.

Table 6.16. Summary of Maximum Maintenance-Free Life of Conventional Pavements Subjected to Heavy Traffic (1 x 10^6 18-kip rigid ESAL/year or 0.7 x 10^6 flexible ESAL/year) in Various Environmental Regions.

Environment	Estimated/Actual Life	JCP*	JRCP	CRCP	FLEX	COMP
WET-FREEZE	ACT	16** JCP-2	<12 JRCP-7 8+ JRCP-10 23+ JRCP-5	(8+) CRCP-4	(18) FLEX-2 (7+) FLEX-8 (7+) FLEX-9 (<5) FLEX-6	9+ COMP-1 (40+) COMP-4
	EST	10-15**(W/LTD)	5-15	10-15	5-15	10-20
WET	ACT	(17+) JCP-12			(6+) FLEX-18	
	EST	10-20 (W/LTD)	10-20	10-20	10-15	10-20
DRY-FREEZE	ACT	(4+) JCP-9				
	EST	10-15 (W/LTD)	10-15	10-15	5-15	10-20
DRY	ACT	(28+) JCP-10 (32+) JCP-7		(6+) CRCP-11		
	EST	15-20	15-20	15-20	10-15	15-20
Pavement Structure		PCC - 9 ins. Stab. Subbase- 6 ins. Joints-Random spaced ave.15' and skewed	PCC- 10 ins. Gran. Subbase- 6 ins. Joints-***	CRC - 9 ins. Stab. Subbase - 6 ins. Deformed Rebar	AC - 5 ins. ATB - 10 ins. Gran. Subbase- 10 ins.	AC - 6 ins. PCC - 8 ins. 15 ft. joints Gran. Subbase- 6 ins.

* Doweled load transfer devices used in freeze regions and wet regions only as indicated.
** All estimates are in years,
*** Joints - Contraction at 30-56 ft or less, or expansion for greater than 40 ft.

1. Wet/Freeze Regions: Composite pavement provides 10-20 years of maintenance-free life. JCP (with LTD) and CRCP provide 10 to 15 years maintenance-free life. Environmental factors in the wet/freeze region severely limit the lives of JRCP and FLEX pavements to 5 to 10 years.

2. Wet/Non-Freeze Regions: JCP, JRCP, CRCP, and COMP provide maintenance-free life from 10 to 20 years, and FLEX from 10 to 15 years.

3. Dry/Freeze Regions: COMP provides maintenance-free life of 10 to 20 years. JCP, JRCP, and CRCP provide 10 to 15 years due to environmental effects. FLEX is limited to 5-15 years due mainly to thermal cracking effects.

4. Dry/Non-Freeze Regions: JCP, JRCP, CRCP, and COMP provide from 15 to 20 years maintenance-free life. FLEX is limited to 10 to 15 years due to traffic load damage.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 GENERAL

This chapter briefly summarizes the significant conclusions reached in this study and makes recommendations to the sponsor, the Federal Highway Administration, U.S. Department of Transportation, as to future research work under this contract, and possible future contracts.

7.2 CONCLUSIONS

The conclusions reached in this study are based upon three major sources of information: (1) the numerous projects surveyed and analyzed during the field studies (these projects are many times representative of several other projects in the region), (2) information gained from interviews with numerous pavement engineers (design, construction, maintenance, and administration), and (3) review of previous road tests data (AASHO, Michigan, Minnesota, Illinois, etc.).

1. Given below for each of the pavement types are (1) the major types of distress, and (2) causes for distress, with maintenance performed:

a. JCP

(1) The major distress types are faulting at the joints (where no LTD's are used) and transverse cracking.

(2) Causes are any of the following: lack of adequate load transfer and heavy traffic loadings, inadequate slab thickness, inadequate drainage, and pumping.

(3) Major maintenance activities associated with these pavements are crack sealing, patching, and leveling of faulted joints by grinding or slab jacking.

b. JRCP

(1) The major distress types are joint deterioration (spalling and blowups), and transverse cracking with spalling and faulting of the crack if reinforcement ruptures.

(2) Causes for this distress are any of the following: heavy traffic loadings, inadequate slab thickness, curling of slabs, pumping of subbase, inadequate drainage, improper joint design, and corrosion of LTD's, probably augmented by heavy use of deicing salt.

(3) Major maintenance activities are crack sealing, slab patching, joint repair, and slab jacking.

c. CRCP

(1) Major distress types are localized failure, surface depression, crack spalling, yielding of the reinforcing steel (with subsequent spalling and faulting of the crack), and construction joint deterioration.

(2) Causes include any of the following: heavy traffic loadings, inadequate steel lap, poor joint construction, inadequate drainage, pumping of subbase, inadequate slab thickness for the specific subgrade support conditions, and corrosion of steel, especially in areas of heavy deicing salt applications.

(3) The major maintenance activity associated with CRCP is slab patching and repair of edge distress.

d. FLEX

(1) Major types of distress include fatigue or alligator cracking, transverse cracking, longitudinal cracking, and rutting.

(2) Causes for the distress include any of the following: heavy traffic loadings and inadequate pavement structure, low temperature shrinkage, poor lane joint construction, aging of asphalt cement, and disintegration of cement treated base from deicing salts and freeze/thaw.

(3) Major maintenance activities are crack sealing, pavement patching, and application of thin overlays.

e. COMP

(1) Major distress is transverse cracking and subsequent crack spalling and deterioration.

(2) Causes for distress are primarily reflective cracking from the PCC base with concomitant spalling associated with heavy traffic loadings.

(3) Major maintenance activities are crack sealing and patching.

2. Adequacy of commonly used design procedures: A detailed evaluation of several design procedures for all five pavement types was made which includes the AASHO Interim Guide, Portland Cement Association, Asphalt Institute, and several procedures used by the State DOT's. Each procedure was evaluated conceptually, the theoretical deficiencies and limitations determined. The procedures were also evaluated for actual design situations by "redesigning" several projects selected from the field study using the actual traffic data and actual pavement system. The methods showed many theoretical limitations and also failed to provide adequate designs for maintenance-free performance in most instances. The commonly used design procedures evaluated are incapable of designing maintenance-free pavements under heavy traffic for periods of 20 to 40 years. Many limitations and deficiencies of these procedures are documented, and the key factors which must be modified and/or added to the procedures to increase their potential for the design of maintenance-free pavements have been identified. The actual modifications of one or more of these procedures for zero-maintenance pavement design will be conducted during Phase II of this study.

3. Limiting criteria for design of zero-maintenance pavements: The limiting criteria used by maintenance personnel on high volume pavements in determining when maintenance should be applied are based partly upon user considerations (i.e., safety, confort, etc.), partly upon maintenance management policies, and partly on intuition. The two limiting criteria that provide direct evaluation of these factors and can also be used in designing zero-maintenance pavements are pavement serviceability and observable pavement distress. Comprehensive analyses of these factors and their effects were conducted, and the following limiting criteria selected:

a. Terminal serviceability - a value of 3.0 to 3.5 is recommended for all pavement types. The proportion of all projects showing maintenance-free performance at 3.5 level of serviceability was 75 percent. Also, the user costs on maintenance-free pavements increase dramatically when terminal serviceability drops below 3.0 to 3.5.

b. Distress severity and amount - recommended levels of severity for "safety" reasons are provided for the major distresses in flexible and rigid pavements. Recommended amounts for each distress are provided based on their detrimental effect in reducing the pavement serviceability level to a range of 3.0 to 3.5. These types of distress are the major types occurring on high traffic pavements, and if they can be prevented or limited, there is a high probability of obtaining a pavement with maintenance-free performance over longer periods of time.

4. Maximum maintenance-free lives of conventional pavements: A comprehensive analysis was made to determine the maximum maintenance-free life that can be expected from conventional pavements under high traffic volumes to serve in varying environments using current design technology. An overall summary of maintenance-free life under certain specific conditions for all five pavement types is given in Table 6.16. The following

summarizes maintenance-free life expected from conventional pavements included in this study, evaluated with current design methodology and criteria:

a. Wet/Freeze Region - Composite pavement provides 10 to 20 years of maintenance-free life. JCP (with LTD) and CRCP provide 10 to 15 years maintenance-free life. Environmental factors in the wet/freeze region severely limit the lives of JRCP and FLEX pavements to 5 to 10 years.

b. Wet/Non-Freeze Region - JCP, JRCP, CRCP, and COMP provide maintenance-free life from 10 to 20 years, and FLEX from 10 to 15 years.

c. Dry/Freeze Region - COMP provides maintenance-free life of 10 to 20 years. JCP, JRCP, and CRCP provide 10 to 15 years due to environmental effects. FLEX is limited to 5 to 15 years due mainly to thermal cracking effects.

d. Dry/Non-Freeze Region - JCP, JRCP, CRCP, and COMP provide from 15 to 20 years maintenance-free life. FLEX is limited to 10 to 15 years due to traffic load damage.

7.3 RECOMMENDATIONS

Five pavement types that have been constructed on heavily trafficked highways were evaluated. Examples of long-term maintenance-free performance up to 14 to 25 years were found for all of these types of pavements. Most of these types of pavements, with an optimized design, have the potential for zero-maintenance performance over significantly longer time periods. It is desirable to ultimately develop design procedures for maintenance-free pavements for each type indicated for the following reasons:

1. Most geographic regions of the United States have constructed only one or two types of pavements on their freeway systems, and hence have little or no experience with other types. If zero-maintenance design

procedures are developed for only one type of pavement, many regions that have used other types will be reluctant to construct that type. Even if the agency decided to construct the recommended type, the chances for design or construction errors are much greater because of lack of experience.

2. Based upon the results obtained thus far, each pavement type may be designed to give maintenance-free life for periods beyond 20 years, but in various regions certain types may provide better performance and be more economical than others, due to local conditions. Also, with changing energy priorities, the economics and resource priorities may also change precipitiously. <u>Data are available from the first phase of this project to develop maintenance-free design procedures for each type, but current funding constraints limit the development to one type.</u>

Therefore, based upon funding and time limitations of this study, it is recommended that zero-maintenance design procedures be developed for the type that has the greatest potential for providing an immediate design for maintenance-free life under heavy traffic, if properly designed and constructed. The decision as to the most promising type should be based on an overall evaluation of design and construction capability, past performance, and recommendations of the pavement engineers interviewed during the field survey. The most promising type will be selected and detailed design procedures developed during Phase II of this study.

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APPENDIX A

Example of Pavement Redesign

The following is a detailed "redesign" example of section, JRCP-7, I-80 in Illinois redesigned by the AASHO Interim Guide Design Procedure for Rigid Pavement Structures.

1. TRAFFIC ANALYSIS

   a. Design ADT of Pavement

The design ADT is the average ADT over the life of the pavement. The design ADT is determined from available yearly ADT's from 1962 to 1974 listed below:

ADT	Year
4700	1963
6500	1964
8800	1965
12800	1968
12100	1969
14700	1971

A graph of ADT vs time in years indicates a straight line trend. The average ADT over the entire period from 1962 through 1974 is computed to be 11,100.

   b. Percent Trucks (% T)

The average percent trucks (including pick-ups and panels) was determined similar to the mean ADT. A value of 32.4 percent is obtained.

c. Average axles/truck (A)

The average axles/truck, or A, is found by dividing the total number of axles (counting tandem axles as one) by the total number of trucks over a particular period. The following data was obtained from a recent W-4 form for a weigh station near the project.

$$A = \frac{\text{Total axles}}{\text{Total trucks}} = \frac{21782}{7721} = 2.82 \; \frac{\text{axles}}{\text{truck}}$$

d. Directional Distribution Factor (DD)

The traffic is assumed to be equally distributed in each direction. Therefore DD = 0.50.

e. Lane Distribution (LD)

The LD is the percentage of traffic in the heaviest traveled lane. Actual counts were made on several pavements and use was also made of results obtained from the Georgia Department of Transportation. The following equation was used to estimate lane distribution for JRCP-7 which is a four lane rural freeway.

$$\begin{aligned} \text{LDF } (\%) &= 96.39 - 0.0004 \text{ ADT}/2 \\ &= 94.17 \text{ percent}^* \end{aligned}$$

f. Design Life (L)

The pavement was under regular traffic from October 1962 through 1974. This section was also previously part of Loop 6 of the AASHO Road Test under traffic from 1958-1960 when 1,114,000 applications of a 48-kip tandem axle load was applied.

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*"Determination of the Lanal Distribution of Truck Traffic on Freeway Facilities, Georgia Department of Transportation, Research Project No. 7001, Final Report, 1971.

g. Total 18-Kip Equivalent Singe Axle Load Applications

The total 18-kip equivalent single axle load applications applied are computed from the following expression:

$$n = \left[\sum_{i=1}^{k} p_i \; e_i \right] (ADT_d) \; (A) \; (L) \; (T) \; (\overline{DD}) \; (\overline{LD})$$

where: n = total 18-kip ESAL

ADT_d = average ADT over design period

A = average axles/truck

L = design life, days

T = percent trucks of ADT

$\overline{DD}$ = directional distribution, fraction

$\overline{LD}$ = land distribution, fraction

p_i = percent axles in i^{th} load range

e_i = equivalency factor for i^{th} load range

k = number of load ranges

The $\Sigma p_i e_i$ is determined from the actual load distribution of the project and the equivalency factors. Several 8-hour truck weighings were made for this project from 1962 to 1974. An average load distribution over this time period is given in Table A.1.

$$n = (.3535)\ (11100)\ (2.8)\ (365)\ (13)\ (.324)\ (.5)\ (.9417)$$
$$= 7{,}953{,}059 \ 18^K \text{ SAL APPLICATIONS}$$

Because this test section, JRCP-7 was part of the original AASHO Road Test, LOOP 6, it was necessary to include the 1,114,000 applications of a 48-kip tandem axle to the total 18-kip eq SAL applications

Therefore:

$$n = 7{,}953{,}059 + 1{,}114{,}000(7.99)$$
$$= 7{,}953{,}059 + 8{,}900{,}860$$
$$= 16{,}853{,}919 \ 18^K \text{ SAL APPLICATIONS}$$

where: 7.99 = the equivalency factor of a 48-kip tandem axle

2. CONCRETE PROPERTIES

a. Modulus of Rupture

The flexural strength of the concrete slab of the AASHO Road Test at 28 days was 690 psi.

3. SOIL SUPPORT

a. k_g Value on Top of Subbase

The support under the concrete slab of the road test section, JRCP-7 consists of a 6" granular subbase placed on an A-6 embankment soil. From the AASHO Road Test Research Report #5, the average soil support values where given as follows:

k_e on top of subgrade = 86 psi

k_e on top of subbase = 100 psi

The gross, k_g, on top of the subbase was determined by the relationship:

$$k_e = 1.77\ k_g$$

Therefore: $k_g = 108/1.77 = 61.0$ psi

4. SERVICEABILITY

Serviceability values were given as follows:

P_i = initial serviceability = 4.5

P_t = terminal serviceability = 2.5

5. AASHO NOMOGRAPH

The design inputs used to obtain a slab thickness are as follows:

$P_i = 4.5$

$P_t = 2.5$

Total Eq. 18-K SAL Applications 16.85×10^6

M_r = Modulus of Rupture = 690 psi

f_t = working stress in concrete

= safety factor X M_r

= .75(690) = 517.5 psi

k_g ="gross k" = 61 psi

Using these data and the AASHO Rigid Pavement Nomograph the slab thickness is determined to be 10.8 ins.

Table A.1 LOAD DISTRIBUTION ON JRCP 7

LOADS	% Load Distribution	Equivalency factor e_i	$(P_i)(e_i)(10^2)$
Single Axles			
3^K	7.52	.00055	.004136
$3 - 7^K$	16.82	.006375	.1072275
$7 - 8^K$	8.01	.025	.20025
$8 - 12^K$	16.56	.0925	1.5318
$12 - 16^K$	4.92	.35625	1.75275
$16 - 18^K$	2.01	.77	1.5477
$18 - 20^K$	.87	1.26	1.0962
$20 - 22^K$	.20	1.995	.399
$22 - 24^K$	.08	2.975	.238
$24 - 26^K$	.01	4.1	.041
$26 - 30^K$	-	-	-
$30^K \rightarrow$	.01	8.54	.0854
Tandem Axles			
6^K	.37	.003	.0011
$6 - 12^K$	10.31	.012	.12372
$12 - 18^K$	5.36	.071	.38056
$18 - 24^K$	5.68	.269167	1.528868
$24 - 30^K$	9.92	.765	7.588
$30 - 32^K$	5.90	1.36	8.024
$32 - 34^K$	3.73	1.75	6.5275
$34 - 36^K$	1.24	2.235	2.7714
$36 - 40^K$	.45	2.825	1.27125
$40 - 42^K$	-	-	-
$42^K \rightarrow$	.03	4.3	.129
			35.35

Appendix B Data Used To Develop The Predictive Equation Of Serviceability Index In Chapter 5.

Table B.1.

Flexible Information n = 95

PSR	$\overline{RD}$ (in)	$\overline{RDV}$ in^2 X 100	AC $ft^2/1000ft^2$	TC $ft/1000ft^2$	PATCHING $ft^2/1000ft^2$
4.3	0.10	0.7	0	0	0
2.4	0.22	9.2	343	0	0
3.3	0.08	3.6	8	0	0
4.4	0.08	0.7	0	0	0
3.8	0.06	0.4	0	0	0
2.6	0.08	5.7	64	0	0
3.2	0.15	3.4	2	0	3
2.4	0.16	3.4	17	0	14
1.3	0.26	10.3	292	0	11
1.1	0.19	10.9	21	0	2
3.8	0.04	0.4	0	29	0
3.8	0.09	0.3	0	34	0
3.8	0.05	0.2	0	14	0
3.8	0.04	0.4	0	9	0
3.2	0.14	0.5	0	0	10
1.3	0.07	6.6	145	22	35
1.3	0.08	2.9	75	12	55
2.1	0.18	3.2	15	0	5
1.5	0.37	5.6	30	2	76
2.4	0.07	3.2	2	0	3
4.2	0.11	0.2	0	0	0
3.9	0.09	0.2	0	0	0
3.1	0.08	1.3	1	68	66
2.2	0.13	5.2	0	1	4
1.5	0.08	5.4	0	7	6
1.6	0.12	4.6	0	0	0
4.0	0.04	0.2	0	0	0
4.2	0.03	0.2	0	0	0
2.9	0.01	1.4	0	74	0
4.1	0.24	0.4	44	43	0
4.0	0.34	0.5	204	75	0
3.2	0.02	0.4	12	1	0
2.4	0.17	1.8	455	0	17
2.9	0.10	1.2	292	0	32
1.7	0.02	1.9	719	0	111
1.0	0.03	5.1	691	0	161
1.3	0.14	7.0	613	0	159
3.2	0.18	2.0	17	0	0
2.7	0.14	0.8	45	0	0

PSR	$\overline{RD}$ (in)	$\overline{RDV}$ in^2 X 100	AC $ft^2/1000ft^2$	TC $ft/1000ft^2$	PATCHING $ft^2/1000ft^2$
1.6	0.23	2.3	502	0	31
1.4	0.27	2.9	437	0	72
2.6	0.24	1.2	10	64	2
3.4	0.22	0.2	183	46	0
2.9	0.09	0.8	177	0	4
4.3	0.01	0.1	0	0	0
4.3	0	0	1	0	0
4.2	0.12	0.2	0	1	0
3.9	0.16	0.4	0	2	0
3.8	0.08	0.3	0	0	0
3.4	0.20	2.2	51	0	0
3.1	0.11	0.8	17	0	7
4.1	0.03	0.2	0	0	0
3.4	0.33	0.9	14	0	0
3.4	0.24	0.9	0	0	0
2.8	0.43	1.4	5	0	25
3.5	0.46	0.5	0	0	0
3.3	0.39	0.4	9	0	0
3.3	0.44	1.0	2	0	0
3.6	0.47	0.6	0	0	0
3.2	0.53	1.3	0	0	0
3.4	0.56	0.5	0	0	0
1.8	0.73	5.1	80	0	16
3.3	0.38	0.5	0	0	0
2.6	0.55	0.6	15	0	0
3.2	0.54	0.7	1	0	0
1.7	0.92	2.8	222	0	0
2.4	0.53	1.5	21	0	0
3.0	0.46	1.1	0	0	0
3.3	0.09	0.4	0	0	0
2.7	0.11	3.8	300	0	1
2.4	0.22	1.3	496	0	52
0.9	0.25	6.2	392	0	60
4.3	0.02	0.13	3	0	0
3.2	0.05	1.23	239	0	0
4.0	0.01	0.04	0	37	0
3.4	0.02	0.18	113	2	0
1.5	0.10	1.82	0	97	62
4.0	0.10	0.23	0	0	0

PSR	$\overline{RD}$ (in)	$\overline{RDV}$ in^2 X 100	AC $ft^2/1000ft^2$	TC $ft/1000ft^2$	PATCHING $ft^2/1000ft^2$
4.1	0.09	0.35	0	0	0
4.4	0.05	0.25	0	0	0
2.9	0.09	0.73	48	0	17
2.1	0.11	1.30	7	0	15
2.1	0.05	2.33	0	28	48
3.6	0.05	0.32	10	5	0
3.4	0.03	0.21	2	0	0
3.9	0	0.19	0	13	0
3.8	0.03	0.22	0	0	0
2.6	0.04	1.24	6	0	90
3.6	0.07	0.90	69	0	0
4.2	0.02	0.15	4	0	0
3.5	0.18	1.72	10	0	9
4.0	0.02	0.30	0	1	0
3.9	0.03	0.36	0	8	0
4.3	0.03	0.30	37	0	0
4.2	0.11	0.58	58	0	0

Table B. 2.

Rigid Information   n = 65

PSR	Faulting in/1000 ft	Cracking ft^2/1000 ft^2	Spalling ft^2/1000 ft^2	Patching ft^2/1000 ft^2
2.0	2	53	4	8
4.2	0	4	0	0
2.6	0	42	0	11
2.3	7	46	0	7
1.2	1	102	0	28
2.8	3	15	2	1
4.4	0	0	0	0
1.1	3	65	11	5
0.9	1	74	19	85
1.3	1	40	60	59
1.8	0	23	4	66
2.1	0	47	1	41
4.1	0	4	0	0
3.8	0	2	0	0
3.0	1	14	0	1
3.0	1	22	0	0
2.9	0	14	0	0
2.5	0	34	0	0
4.3	0	0	0	0
4.3	0	0	0	0
3.7	0	0	0	0
3.6	0	0	0	0
3.9	0	0	0	0
3.9	0	0	0	0
1.3	0	76	2	1
1.2	10	64	0	0
2.2	4	97	0	1
4.4	0	0	0	0
4.0	2	0	1	0
3.8	4	11	1	0
3.6	1	2	4	0
3.2	4	1	1	2
2.6	5	72	13	0
2.8	5	70	10	1
1.8	1	41	4	29
1.8	2	42	8	37
2.1	1	50	7	29
2.2	2	86	5	33
1.8	0	40	6	65
2.7	5	81	3	5
4.2	1	0	1	0
4.3	1	0	0	0
4.3	1	0	0	0

PSR	Faulting in/1000ft	Cracking $ft^2/1000ft^2$	Spalling $ft^2/1000ft^2$	Patching $ft^2/1000ft^2$
2.2	18	36	1	0
4.3	1	0	0	0
2.8	2	5	2	13
2.7	2	5	7	16
1.3	23	39	11	0
1.9	11	42	10	0
3.3	3	8	1	0
2.5	10	33	3	0
3.2	3	31	3	0
3.4	2	22	1	0
3.5	2	10	1	0
3.9	1	0	2	0
1.7	24	33	10	0
1.7	19	17	15	0
3.4	4	3	1	0
3.4	5	0	1	0
3.4	3	0	1	0
3.4	2	0	2	0
3.5	2	9	0	0
3.4	3	0	2	0
3.6	3	2	0	0
3.2	17	0	0	0

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