PB91-100560

Performance of Jointed Concrete Pavements

Volume V Appendix B - Data Collection and Analysis Procedures

Publication No. FHWA-RD-89-140

March 1990

NJ Route 130 10-in. JCRP 12-in. gravel base 36 Years 35 Million ESAL's PSR - 3.8

U.S. Department of Transportation
Federal Highway Administration

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

FOREWORD

This report is one volume of a three-volume set of interim reports evaluating the performance of 95 experimental or other in-service pavements in the United States or Canada. Extensive design, construction, traffic and performance data were collected. Design features evaluated are: slab thickness, base type, joint spacing, reinforcement, joint orientation, load transfer, dowel bar coatings, longitudinal joint design, joint sealant, tied shoulders and subdrainage. Volume I summarizes the performance data. Volume IV contains a detailed summary of the performance of the individual sections and Volume V describes the data collection and analysis procedures and includes an annotated bibliography on various aspects of portland cement concrete pavement performance, design and analysis.

Volumes II and VI will be distributed separately in early 1990 and will describe the evaluation and modification of various design and analysis models. Volume III (Summary of Research Findings) and the Technical Summary will be given widespread distribution also in early 1990.

This report will be of interest to those involved in the design, construction and maintenance of jointed concrete pavements.

Sufficient copies of this report are being distributed by FHWA memorandum to provide one copy to each FHWA Region and Division and two copies to each State highway agency. Direct distribution is being made to the Division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), U. S. Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161. A small charge will be imposed for each copy ordered from NTIS.

Thomas J. Pasko Jr

Thomas J. Pasko, Jr., P.E.
Director, Office of Engineering and
Highway Operations Research and Development

NOTICE

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3.
FHWA-RD-89-140		PB91-100560

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code
PERFORMANCE OF JOINTED CONCRETE PAVEMENTS Volume V - Appendix B Data Collection and Analysis Procedures	5. March 1990 6.
7. Author(s) K. D. Smith, D. G. Peshkin, M. I. Darter A. L. Mueller, S. H. Carpenter	8. Performing Organization Report No.
9. Performing Organization Name and Address ERES Consultants, Inc. 1401 Regency Drive East Savoy, Illinois 61874	10. Work Unit No. 3C1A2012 11. Contract or Grant No. DTFH61-86-C-00079
12. Sponsoring Agency Name and Address Office of Engrg. and Highway Operations R & D Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296	13. Type of Report and Period Covered Interim Report October 1986 - June 1989 14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contract Manager (COTR):
Roger M. Larson, HNR-20

16. Abstract

A major national field and analytical study has been conducted into the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the country. Field data collected and analyzed included distress, drainage, roughness, present serviceability rating (PSR), deflections, destructive testing (coring and boring), and weigh-in-motion (WIM) on selected sites. This information was compiled into a comprehensive microcomputer database. Projects were evaluated on an individual basis and then compared at a national level to identify performance trends. The performance data was used to evaluate and modify several concrete pavement design procedures and analysis models.

This volume provides documentation of the data collection and analysis procedures used in the study. The field data collection activities, including deflection testing and coring/boring, are discussed, as is the weigh-in-motion (WIM) data collection procedures. Traffic computations and backcalculation methodologies are presented, along with the basis for the drainage is followed by an annotated bibliography of pertinent publications.

FHWA No.	Vol. No.	Short Title
FHWA-RD-89-136	I	Evaluation of Concrete Pavement Performance and Design Features
FHWA-RD-89-137	II	Evaluation and Modification of Concrete Pavement Design and Analysis Models
FHWA-RD-89-138	III	Summary of Research Findings
FHWA-RD-89-139	IV	Appendix A Project Summary Reports and Summary Tables
FHWA-RD-89-141	VI	Appendix C Synthesis of Concrete Pavement Design Methods and Analysis Models Appendix D Summary of Analysis Data for the Evaluation of Predictive Models

17. Key Words	18. Distribution Statement
Concrete, concrete pavement, pavement performance, pavement evaluation, pavement design, nondestructive testing, slab thickness, base type, joint spacing, reinforcement, subdrainage	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	159	

Form DOT F 1700.7 (8-69) Reproduction of form and completed page authorized.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

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

MASS

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

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

LENGTH

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

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2

VOLUME

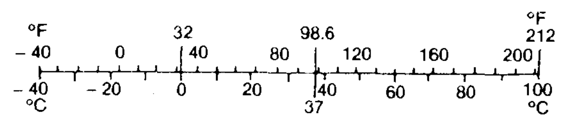
Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

MASS

Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

TABLE OF CONTENTS

VOLUME I EVALUATION OF CONCRETE PAVEMENT PERFORMANCE AND DESIGN FEATURES

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(continued)
VOLUME IV APPENDIX A - PROJECT SUMMARY REPORTS AND SUMMARY TABLES

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)
VOLUME V APPENDIX B - DATA COLLECTION AND ANALYSIS PROCEDURES

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

LIST OF TABLES
VOLUME I EVALUATION OF CONCRETE PAVEMENT PERFORMANCE AND DESIGN FEATURES

LIST OF TABLES
(Continued)

LIST OF TABLES
(Continued)

LIST OF TABLES
(Continued)

VOLUME IV PROJECT SUMMARY REPORTS AND SUMMARY TABLES

LIST OF TABLES
(Continued)

LIST OF TABLES
(Continued)

LIST OF TABLES
(Continued)

LIST OF TABLES
(continued)

VOLUME V DATA COLLECTION AND ANALYSIS PROCEDURES

LIST OF FIGURES

VOLUME I EVALUATION OF CONCRETE PAVEMENT PERFORMANCE AND DESIGN FEATURES

LIST OF FIGURES
(continued)

VOLUME IV PROJECT SUMMARY REPORTS AND SUMMARY TABLES

VOLUME V DATA COLLECTION AND ANALYSIS PROCEDURES

LIST OF FIGURES
(Continued)

CHAPTER 1 FIELD DATA COLLECTION PROCEDURES

1. INTRODUCTION

At the very heart of the success of this project is the extensive field data collection effort. Four trips of 10 days to 6 weeks duration were scheduled. The field surveys started in May 1987 with a trip to Minnesota. That was followed by trips to the east (Michigan, Ontario, Northern Ohio, Pennsylvania, New Jersey, and New York), the southeast (Southern Ohio, North Carolina, and Florida), and the west and southwest (Arizona and California). The field surveys for Phase I were completed in November. Field surveys were also conducted at proximate Phase II sections located in Pennsylvania, New Jersey, New York, Connecticut, Maine, and Louisiana.

Integral to the success of the field surveys and the reliability of the data obtained is the degree of consistency or repeatability achieved in the performance of the surveys, at locations from Maine to California and Minnesota to Florida. Several steps were taken to achieve a high level of reliability in the data collection process. The Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) Data Collection Guide was followed to ensure the identification and collection of all key data elements.(1) In the field, pavement distresses were identified and quantified according to reference 2. This distress identification manual provides a uniform basis for collecting distress data from survey to survey, and its use also ensures that the collected data will be consistent with data collected for the LTPP studies.

The composition of the survey crews also helped to ensure consistency. The crews typically consisted of two or three members, one a senior project engineer. Another member of the survey crew was a technician who was present on <u>all</u> of the field survey trips and participated in the survey of each section to ensure consistency in data collection. In addition, the same equipment was used in the field evaluation of every section, thereby eliminating equipment variability as a potential source of inconsistency.

A brief description of the data collection activities has been previously discussed in chapter 2 of volume I. In this chapter, the field data collection procedures are described in detail.

2. CONDITION SURVEY

The condition survey included several levels of data collection. Some distresses were actually measured while others were mapped. There were distress conditions whose presence was noted on the survey and there were also conditions that were noted. The actual data collection efforts were guided by the survey sheets shown in figures 1, 2, and 3.

Measurements

Extensive measurements were made on each pavement section. To begin with, an attempt was made to accurately identify the location of the project section so that it could be found in the future. The starting milepost, station, or distance from the nearest fixed object (such as an overpass) were measured. The length of the section was also marked off with a measuring

FIELD SURVEY: GENERAL INFORMATION

STATE CODE __ __
PROJECT ID __/__ __ __/__
ERES ID __ __ __ __-__ __

DATE OF FIELD SURVEY (MONTH/DAY/YR) __ __/__ __/__ __
SURVEYORS' INITIALS ___/___/___

TEST SECTION LOCATION:

START POINT MILEMARK __ __ __.__ __
END POINT MILEMARK __ __ __.__ __

START POINT STATION NUMBER __ __ __ __+__ __.__ __
END POINT STATION NUMBER __ __ __ __+__ __.__ __

LENGTH OF SECTION (FEET) __ __ __

IF NO MP OR STN, DISTANCE FROM NEAREST STRUCTURE/
INTERCHANGE/CROSSROAD (FEET) __ __ __ __

TYPE/NAME OF STRUCTURE/INTERCHANGE/CROSSROAD ________________

NUMBER OF THROUGH LANES IN DIRECTION OF SURVEY ___

OUTSIDE SHOULDER WIDTH (FEET) __ __.__

INSIDE SHOULDER WIDTH (FEET) __ __.__

SHOULDER SURFACE TYPE
OUTSIDE SHOULDER ___
INSIDE SHOULDER ___

TURF...............1	CONCRETE............4
GRANULAR...........2	SURFACE TREATMENT...5
ASPHALT CONCRETE...3	OTHER__________6

AVERAGE CONTRACTION JOINT SPACING (FEET) __ __ __.__
RANDOM JOINT SPACING (IF APPLICABLE) __ __-__ __-__ __-__ __
TRANSVERSE JOINT SKEWNESS Y / N
FT/LANE ___

ROUGHNESS AND SERVICEABILITY:

		LANE NUMBER*		
		1	2	3
ROUGHNESS INDEX	(TRIAL 1)	__ __ __	__ __ __	__ __ __
	(TRIAL 2)	__ __ __	__ __ __	__ __ __
ROUGHNESS MEASUREMENT SPEED (MPH)		__ __	__ __	__ __
PRESENT SERVICEABILITY RATING (MEAN)		__.__	__.__	__.__

*LANE 1 IS OUTER LANE, LANE 2 IS NEXT TO LANE 1, ETC.

Figure 1. General field survey sheet.

FIELD SURVEY: DRAINAGE INFORMATION

STATE CODE __ __
PROJECT ID. __/__ __ __/__
ERES ID __ __ __ __-__ __

	STATION	SLOPE INNER	SLOPE OUTER
LONGITUDINAL SLOPE (NEAREST 1/16") (3 MEASUREMENTS, EQUALLY SPACED ALONG PROJECT)	__ __ __ __+__ __	___/___	___/___
	__ __ __ __+__ __	___/___	___/___
	__ __ __ __+__ __	___/___	___/___
TRANSVERSE SLOPE (NEAREST 1/16") (3 MEASUREMENTS, EQUALLY SPACED ALONG PROJECT)	__ __ __ __+__ __	___/___	___/___
	__ __ __ __+__ __	___/___	___/___
	__ __ __ __+__ __	___/___	___/___
SHOULDER SLOPE (NEAREST 1/16") (3 MEASUREMENTS, EQUALLY SPACED ALONG PROJECT)	__ __ __ __+__ __	___/___	___/___
	__ __ __ __+__ __	___/___	___/___
	__ __ __ __+__ __	___/___	___/___

CUT OR FILL DEPTH (GROUND LEVEL TO PAVEMENT SURFACE ELEVATION) ____

FILL > 40 FT......................1
FILL 16 - 40 FT...................2
FILL 6 - 15 FT....................3
AT GRADE (5 FT FILL TO 5 FT CUT)...4
CUT 6 - 15 FT.....................5
CUT 16 - 40 FT....................6
CUT > 40 FT.......................7

DEPTH OF DITCH LINE (FROM PAVEMENT SURFACE, FEET) __ __

LANE/SHOULDER JOINT INTEGRITY:

	OUTER SHOULDER	INNER SHOULDER
SEALANT DAMAGE	N L M H	N L M H
BLOWHOLES	N L M H	N L M H

TYPE OF SUBSURFACE DRAINAGE SYSTEM PRESENT (VISUAL) ____

NONE.........................1
LONGITUDINAL DRAINS..........2
TRANSVERSE DRAINS............3
OTHER________________________7

INDICATORS OF POOR DRAINAGE:

CATTAILS OR WILLOWS GROWING IN DITCH	Y / N
DRAINAGE OUTLETS CLOGGED	Y / N
DRAINAGE OUTLETS BELOW DITCHLINE	Y / N
NON-CONTINUOUS CROSS SECTION, CROWN TO DRAINAGE DITCH	Y / N
PUMPING	L M H
OTHER____________________________________	Y / N

Figure 2. Drainage field survey sheet.

TO BE SKETCHED	NOTE ON SKETCH	
ALL CRACKING	LIN FT	L M H
LONGITUDINAL JOINT SPALLING		L M H
LONGITUDINAL FAULTING	INCHES	
CRACK FAULTING	INCHES	
SCALING/MAP CRACKING	AREA	L M H
PATCHES/REPLACED SLABS	DIMENSIONS	
IMPROPER JOINT CONSTRUCTION CRACKS	LIN FT	
MISSAWED JOINTS (BONDED PCOL)	LIN FT	
BLOWUPS	BU	
"D" CRACKING	D	L M H
REACTIVE AGGREGATE	RA	
ALL SHOULDER DISTRESSES		

FIELD DATA COLLECTION FORM

STATE __ __
PROJECT ID __/__ __ __/__
ERES ID __ __ __ __-__ __
PAGE # __ __ OF __ __

TRANSVERSE JOINT TYPE CODE
CONTRACTION JOINT.........1
CONSTRUCTION JOINT........2
PATCH APPROACH JOINT......3
PATCH LEAVE JOINT.........4
PRESSURE RELIEF JOINT.....5

	INNER LANE				MIDDLE LANE				OUTER LANE			
	STATION	STATION	STATION	STATION	STATION	STATION	STATION	STATION	STATION	STATION	STATION	STATION
TRANSVERSE JOINT TYPE (USE CODE)												
SPALLING	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
JOINT SEAL DAMAGE	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
LONGITUDINAL JOINT SEAL DAMAGE	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
PATCH/SLAB REPLACEMENT DETERIORATION	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
SLAB DETERIORATION ADJACENT TO PATCH	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
LANE TO SHOULDER DROPOFF (INCHES)	__.__ __								__.__ __			
LANE/SHOULDER SEPARATION (INCHES)	__.__ __								__.__ __			
FAULTING (INCHES)	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __
JOINT WIDTH (INCHES, OUTER LANE ONLY)	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __	__.__ __

Figure 3. Field data collection form.

wheel. The location of features of interest, distresses, and other information were kept track of by stationing each section beginning at STA 0+00. The exact length is important for several calculations, including the normalization of distress quantities to a per mile basis and the calculation of the roughness index. During the marking off of the length of the section, the joint spacing of the mainline pavement (and the shoulder, if applicable) was verified. At the beginning of the survey the paved width of the inner and outer shoulders was also measured.

The faulting of each transverse joint was measured with a faultmeter. This device has a dial gauge which permits values to be recorded to the nearest 0.01 in (0.25 mm). Readings were taken in the outer wheelpath (approximately 18 in [457 mm] in from the edge of the pavement) of both the inner and outer lane. Where traffic volumes were high or where there were more than two lanes in the direction of the survey, no faulting was measured in the second lane; this also often prevented the measurement of faulting in the inner lane. Finally, faulting was also measured on transverse and longitudinal cracks where it occurred.

At regular intervals (determined by the joint spacing), such as every third, fourth, or fifth joint, the width of the joint was measured with a 6-in (152 mm) stainless steel rule. The measurements were recorded to an accuracy of 0.05 in (1.3 mm). At the same interval, the dropoff from the mainline pavement to the shoulder was measured with the faultmeter and the separation between the mainline pavement and the shoulder was measured with the small hand rule. The lane-shoulder separation was recorded as zero if the sealant was tightly adhered to both the pavement and the shoulder.

Transverse slopes of all lanes and the inner and outer shoulders, as well as longitudinal slopes of the pavement lanes, were measured with a 4-ft (1.2 m) level and bubble slope indicator. These readings were taken three times, usually at the beginning, middle, and end of the section. The average was reported in the summary tables and used in all drainage calculations.

Mapped Distresses

Many of the pavement distresses were mapped on the distress survey sheets. First, the transverse joints were drawn on the sheets. They were spaced to scale; however, skewed joints were only noted as skewed and not drawn skewed. Transverse and longitudinal cracks were sketched in, approximately to scale, and their severity was noted. The length of the cracks was scaled off of the survey sheets. Areas of transverse and longitudinal joint spalling were also drawn on the survey sheets and their severity noted alongside on the sketch. The lengths of the spalled area were noted on the survey sheets, as it was difficult to draw these to scale.

All patched areas were drawn to scale on the survey sheets. The approximate shape of the patch was sketched freehand and an attempt was made to identify the patch material. Areas of grinding and grooving were marked on the pavement. The location of both old core holes and the cores taken during this study were drawn on the survey sheet at the section's station. Fixed objects located within the section's boundaries were noted to further aid in locating the section. These included signposts or lightposts, bridges, ramps, and drainage structures.

If concrete shoulders existed on a project, all cracking, spalling, and corner breaks were mapped on the distress sheets. The locations of the joints in the shoulder were also noted in relation to the joints in the mainline pavement. For asphalt-concrete shoulders, the presence of all distress (transverse and longitudinal cracking, alligator cracking, shoving, weathering and raveling) was noted and a general assessment of the shoulder condition was made.

Evaluated Conditions

At each transverse joint, several conditions were evaluated and rated. The overall extent of joint deterioration was noted, and formed the basis for a rating of NONE, LOW, MODERATE, or HIGH. The joint sealant damage at each transverse joint was also evaluated. Ratings of joint seal damage ranged from NONE to HIGH. If there was no joint sealant it received a damage rating of HIGH. The same rating scheme was used for the longitudinal joint sealant. The condition of the longitudinal joint sealant was evaluated continuously along the project, from transverse joint to joint.

The condition of each full width patch was evaluated in the same manner as a transverse joint. The surveys identified spalling at the patch, the condition of the patch, and the condition of the sealant between the patch and the pavement.

Noted Conditions

The surface condition of the pavement was most commonly noted on a project level. The manifestations of materials problems, such as "D" cracking and reactive aggregate, were recorded for the entire section if all areas were similarly affected. Localized surface distresses, such as popouts and areas of crazing and scaling, were also identified.

3. PHOTO SURVEY

The record of the condition survey was backed up by the creation of a 35 mm photographic record of each section. This photo survey consisted of several "standard" photographs and a number of unique photographs. The general approach was the same for each section, however. The first set of slides was shot from the beginning of the section facing toward the end of the section. They showed a progression from the outer drainage ditch to the outer shoulder, the mainline pavement, the inner shoulder, and the inner drainage ditch. This set of photographs serves as an overview of the section. Subsequent photos were made of typical transverse joints, typical slabs, typical lane-shoulder and longitudinal joints, and visible drainage features. The remainder of the photographs were used to record the most notable distresses of the section. For a section in fairly good condition, it was not uncommon to obtain from 12 to 18 photos. However, badly deteriorated sections usually required from 24 to 36 photographs to adequately record the overall condition of the section. The last photograph in each section was taken from the end of the project along the centerline of the pavement and facing back toward the start of the section. An example of the photographic log is shown in figure 4.

ERES PROJECT ID #s:
__ __ __ __-__ __
__ __ __ __-__ __
__ __ __ __-__ __

PROJECT PHOTO RECORD

DATE __ __/__ __/__ __
PAGE # __ __ OF __ __

PHOTO #	DESCRIPTION OF PHOTO	STATION
1		__ __+__ __
2		__ __+__ __
3		__ __+__ __
4		__ __+__ __
5		__ __+__ __
6		__ __+__ __
7		__ __+__ __
8		__ __+__ __
9		__ __+__ __
10		__ __+__ __
11		__ __+__ __
12		__ __+__ __
13		__ __+__ __
14		__ __+__ __
15		__ __+__ __
16		__ __+__ __
17		__ __+__ __
18		__ __+__ __
19		__ __+__ __
20		__ __+__ __
21		__ __+__ __
22		__ __+__ __
23		__ __+__ __
24		__ __+__ __
25		__ __+__ __
26		__ __+__ __
27		__ __+__ __
28		__ __+__ __
29		__ __+__ __
30		__ __+__ __
31		__ __+__ __
32		__ __+__ __
33		__ __+__ __
34		__ __+__ __
35		__ __+__ __
36		__ __+__ __

NOTE: 1ST PHOTO OF SECTION IS OF TITLE SHEET.
2ND PHOTO OF SECTION IS ALONG LENGTH OF PROJECT.
3RD PHOTO OF SECTION IS OF SHOULDER AND LANE/SHOULDER JOINT, ALONG LENGTH OF PROJECT.
4TH PHOTO IS OF REPRESENTATIVE DRAINAGE CONDITIONS ALONG PROJECT.

Figure 4. Project photographic record.

The photographic record serves several purposes. It tells the story of the pavement's condition in a form that is fairly easy to understand. It also helps to visualize the overall setting of the pavement for someone who was not present at the field survey. The field survey can also be an extremely useful tool in the reevaluation of the condition survey, as it helps to answer questions about the extent and severity of various distresses without returning to the field.

4. DRAINAGE SURVEY

The third component of the field data collection process is the drainage survey. Information was collected at each section which, in conjunction with materials data, could be used to perform a rational drainage analysis of each section. The drainage evaluations are described in greater detail in chapter 4 of this volume. The survey itself is described here.

Initially, the average depth from the pavement slab (at the pavements' edge) to the bottom of the drainage ditch was estimated. The location of the section in an area of cut or fill was also made, and the depth of the cut or fill was estimated. If there were any visible signs of the drainage outlets, these were noted on the survey sheets.

The condition of the sealant between the mainline pavement and the shoulder is an important indicator of the condition of the sections' drainage. The damage to the sealant was rated for the section between both the inner and outer shoulders. A rating of NONE showed that the sealant was in excellent condition, while a rating of HIGH meant that the sealant was either in poor condition or nonexistent. Asphalt shoulders were examined for the presence of blowholes. These are semicircular areas of the shoulder where the asphalt pavement has been removed through the powerful expulsion of fines and water trapped underneath the pavement slab.

The drainage survey included an inventory of indicators of poor drainage. Where present, the transverse drain outlets were examined. Were the drains clogged or clear and freeflowing? Were they silted over or partially submerged, or well clear of the ground? The condition of the drainage ditch is also an indication of the overall drainage condition of the section. The ditch was evaluated for the presence of cattails or willows. These grow where there is an adequate supply of water which does not flow freely.

Another sign of poor drainage is the presence of a noncontinuous cross section from the pavement crown to the bottom of the ditch. Under certain conditions, water and sediment or debris may drain along the project before they reach the bottom of the ditch. When this happens, it will often cause a discontinuity in the pavement surface profile, as a buildup of sediment or debris forms at the edge of the drainage flow.

The preceding are mainly indicators of poor drainage on the pavement's surface. The primary indicator of poor subsurface drainage is pumping. The most common sign of pumping is the staining of the shoulder by the fine sediment ejected between the pavement and the shoulder. This phenomenon is usually observed at the transverse joints of the pavement. Another sign of pumping is the presence of blowholes, which are discussed above. Pumping severity was rated on a project level according to its extent.

5. FIELD TESTING

Several types of field testing were conducted on the pavement sections. These included deflection testing with a Falling Weight Deflectometer (FWD), coring and boring, and the collection of roughness and Present Serviceability Rating (PSR) data.

Falling Weight Deflectometer (FWD)

FWD testing was performed with a Dynatest Model 8000. The deflection testing pattern used for each section was determined by the type of shoulder included on the section. The different testing patterns are shown in figures 5 through 7.

The FWD testing was performed while the temperature of the pavement was below 80 °F (27 °C), to avoid the influence of slab expansion on the measurement of load transfer and void detection. The temperature of the pavement was monitored by a thermometer probe placed in conducting media poured in a hole drilled in the pavement. The FWD equipment monitors the ambient temperature and records it along with the raw deflection data on both computer tape and a hardcopy output. An example of the raw FWD data output is shown in figure 8.

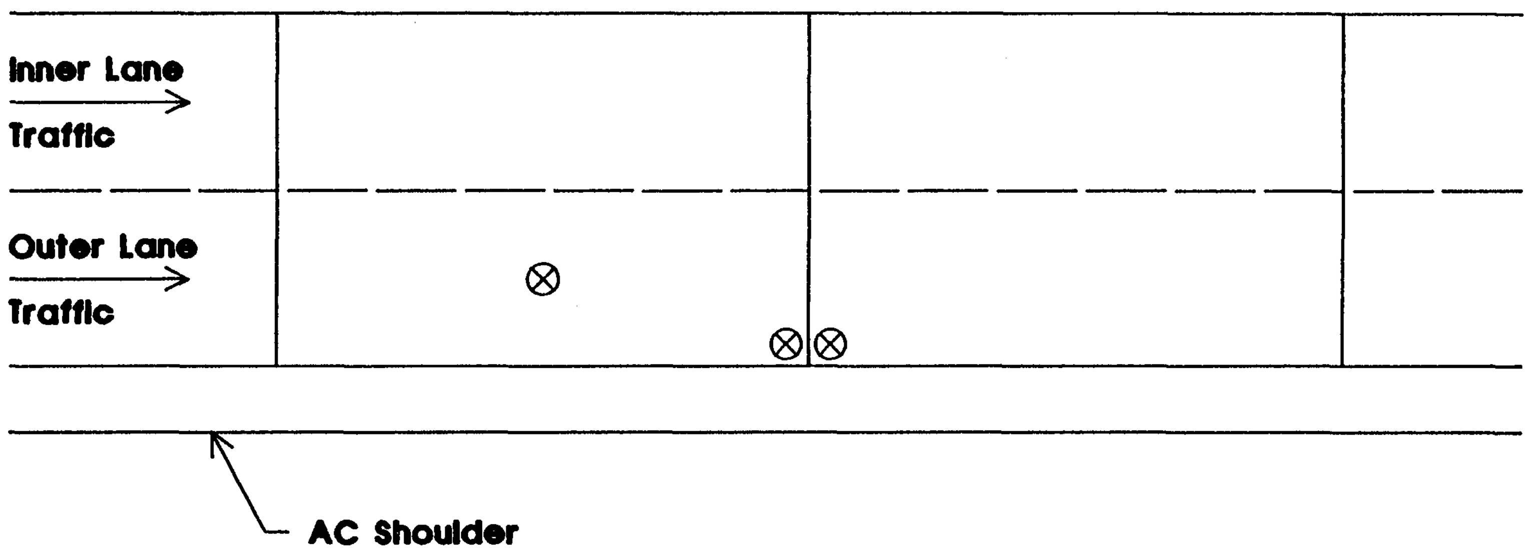
The deflection data from the FWD testing is used in a number of analyses. The center slab deflections are used to backcalculate the modulus of elasticity (E) of the slab and the dynamic modulus of subgrade reaction on top of the base (k). The center slab deflections are also used to calculate the correction factor for the curvature of the slab under testing.

It should be emphasized that the k-value determined from the deflection testing is the dynamic value and not the static value. The static k-value is traditionally the value used in pavement design; it is typically about one-half of the dynamic value. Thus, for the work done in volumes I and II requiring static k-values, the dynamic k-values were divided by two.

Load transfer at the transverse joints was measured with the FWD by following the testing pattern at the joint corners. The ratio of the deflection of the unloaded side to the deflection of the loaded side was multiplied by the correction factor mentioned above to obtain the percent load transfer. The joint corner deflections were also used to detect voids under joint corners. The procedures followed are described in reference 3. The load transfer across the joint between the mainline pavement and portland cement concrete (PCC) shoulders was measured in a similar manner.

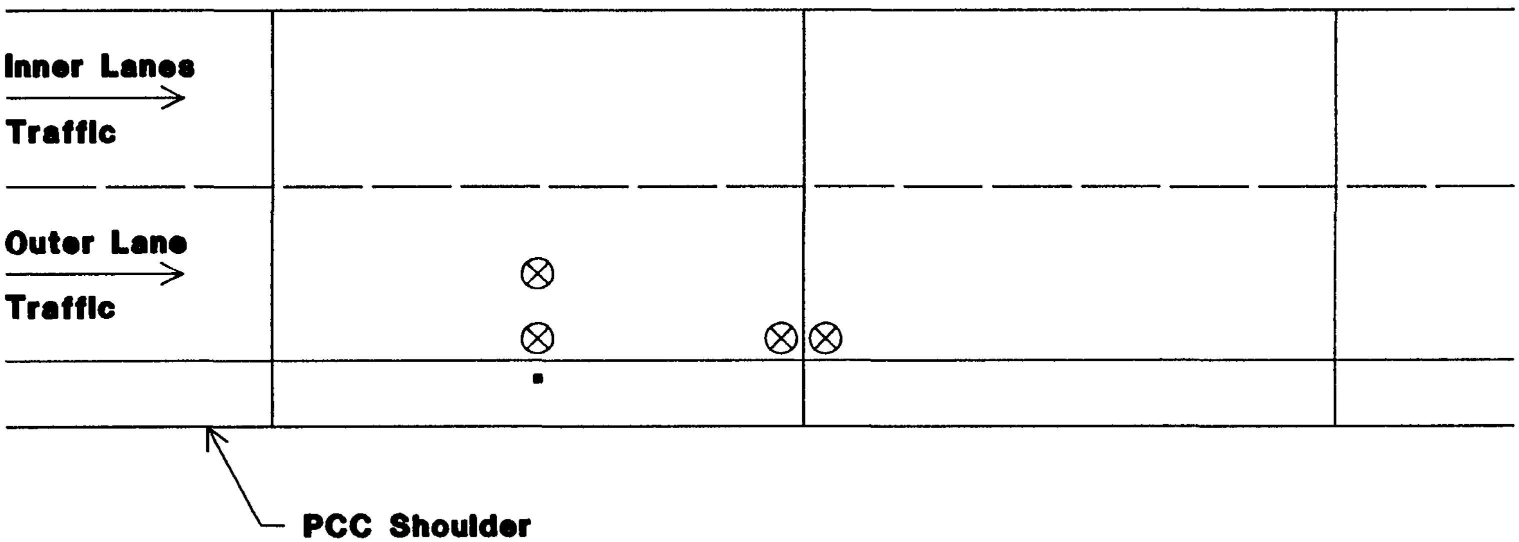
Coring and boring

Almost all of the pavements were cored for the retrieval of a concrete sample. Coring was performed with a portable drill, equipped with a 6-in (152 mm) diameter bit. Center slab cores were retrieved from almost all of the sections in the United States. These were properly prepared and tested in split tensile according to AASHTO T-198 (ASTM C 496). Stabilized base samples of sufficient dimensions were also retrieved and tested similarly.



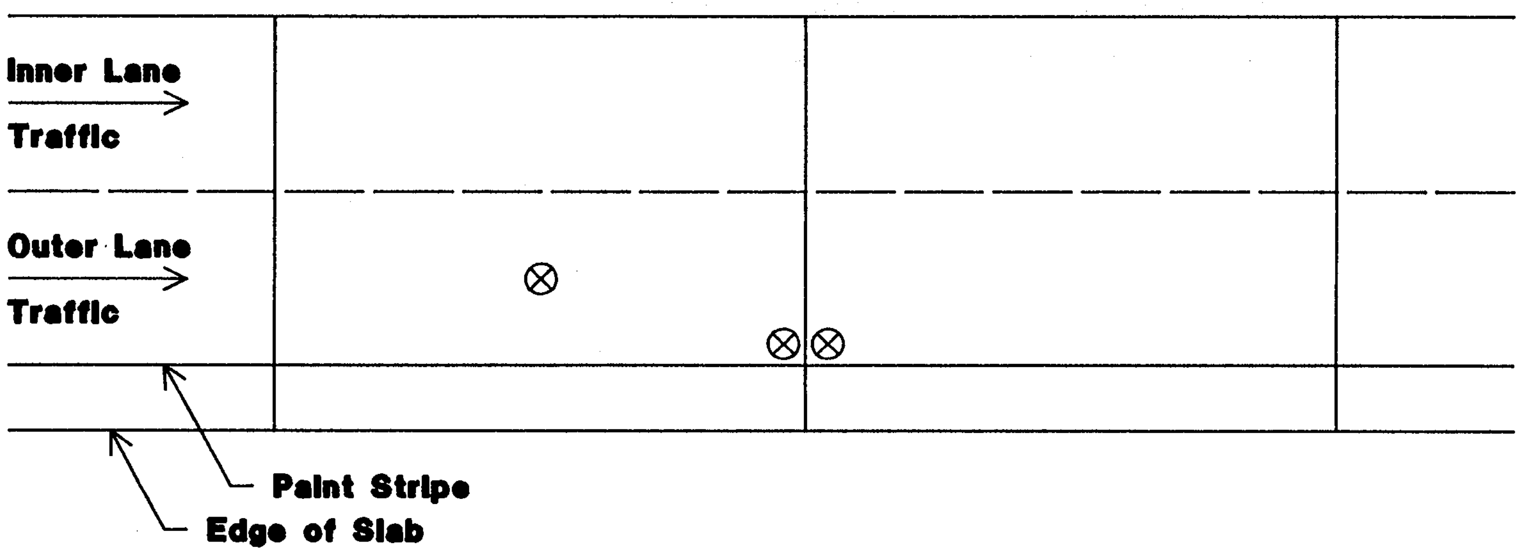
Required: 10 slab centers and 20 corners spread out over length of test section.

Figure 5. Layout for FWD testing of projects with AC shoulders.



Required: 10 slab centers and 5 slab edges, including load transfer across shoulder joint. Also need 10 slab corners (20 drops) spread out over length of test section.

Figure 6. Layout for FWD testing of projects with PCC shoulders.



Required: 10 slab centers and 20 corners at edge of paint stripe, not edge of slab. Testing spread out over length of test section.

Figure 7. Layout for FWD testing of projects with widened lanes.

Raw data from HP file : mi1-1a.hpd

Station - 36CEN Pvt - 28 Temperature - 81°F

Load	D0	D1	D2	D3	D4	D5	D6
8045	2.8	2.5	2.2	1.9	1.5	1.3	2.6
8141	2.8	2.6	2.2	1.9	1.5	1.3	2.6
12545	4.6	4.2	3.7	3.1	2.5	2.1	4.2
17220	6.5	5.8	5.2	4.3	3.6	3.0	5.8

Station - 72TJT Pvt - 28 Temperature - 81°F

Load	D0	D1	D2	D3	D4	D5	D6
7759	6.6	5.7	4.5	3.4	2.5	2.0	6.1
7759	6.4	5.6	4.3	3.3	2.4	1.9	5.9
12036	10.6	9.1	7.2	5.5	4.2	3.3	9.6
17124	14.6	12.7	9.8	7.5	5.7	4.4	13.1

Station - 72TBT Pvt - 28 Temperature - 81°F

Load	D0	D1	D2	D3	D4	D5	D6
7823	6.1	5.2	4.3	3.3	2.6	2.0	5.6
7648	6.0	5.1	4.3	3.3	2.6	2.0	5.6
12036	9.9	8.3	6.9	5.4	4.2	3.3	8.9
17013	13.9	11.8	9.7	7.6	6.0	4.7	12.4

Station - 108CEN Pvt - 28 Temperature - 81°F

Load	D0	D1	D2	D3	D4	D5	D6
7823	2.7	2.6	2.2	1.9	1.6	1.3	2.6
7966	2.8	2.5	2.3	1.9	1.6	1.4	2.6
12513	4.6	4.3	3.8	3.3	2.8	2.3	4.2
17013	6.5	6.0	5.2	4.5	3.7	3.1	6.1

Station - 143TJT Pvt - 31 Temperature - 84°F

Load	D0	D1	D2	D3	D4	D5	D6
7823	5.7	5.0	4.2	3.4	2.7	2.1	4.9
7934	5.5	4.8	4.0	3.2	2.6	2.0	4.6
12116	9.4	8.1	6.8	5.4	4.3	3.5	7.8
16759	12.7	11.0	9.1	7.3	5.8	4.6	10.6

Figure 8. Raw data file from the Dynatest Model 8000.

The split tensile results obtained from the testing of the cores provided an estimate of the concrete modulus of rupture through the following relationship:

$$M_R = 1.02*f_t + 210 \qquad (1)$$

where:

M_R = concrete modulus of rupture, psi

f_t = split tensile strength, psi

It has been shown that split tensile testing results on 6-in (152 mm) diameter cores provide better correlations with the concrete modulus of rupture than split tensile results on 4-in (102 mm) diameter cores.(4)

Cores were also retrieved from the transverse joints of most of the older sections. These were visually examined and given a subjective rating of their condition. Of special interest were any signs of deterioration along the underside of the joint core. The joint cores were also examined for microcracking of the aggregate, which would indicate a materials durability problem.

Base, subbase, and subgrade materials were retrieved from beneath the slab cores. The particle size distribution of granular materials was determined according to standard test methods. The liquid limit and plasticity index were also determined. This information was used to estimate a classification of the granular material according to procedures described in AASHTO M 145.

Roughness/PSR

Pavement surface roughness data was collected on all of the sections. A 1985 Buick Le Sabre was fitted with a rear axle-mounted Mays Roughness Meter. This was run over each lane of each section twice and the results were averaged. A sample of the output is shown in figure 9. Calculation of the Roughness Index in units of inches/mile is a matter of knowing the length of the section and measuring the length of the accumulated roughness. The test was run at a constant speed of 50 mi/h (80 km/h). During the first pass the passengers in the car gave the section a present serviceability rating (PSR). These ratings were averaged for the section.

Traffic Control

Traffic control for the testing was provided by maintenance forces of the participating States.

6. WEIGH-IN-MOTION DATA COLLECTION

Weigh-in-motion (WIM) traffic information was also collected on selected sites. This work provided traffic volumes by lane and by direction, and also axle load distribution information. Details of his data collection activity are described in chapter 2 of this volume.

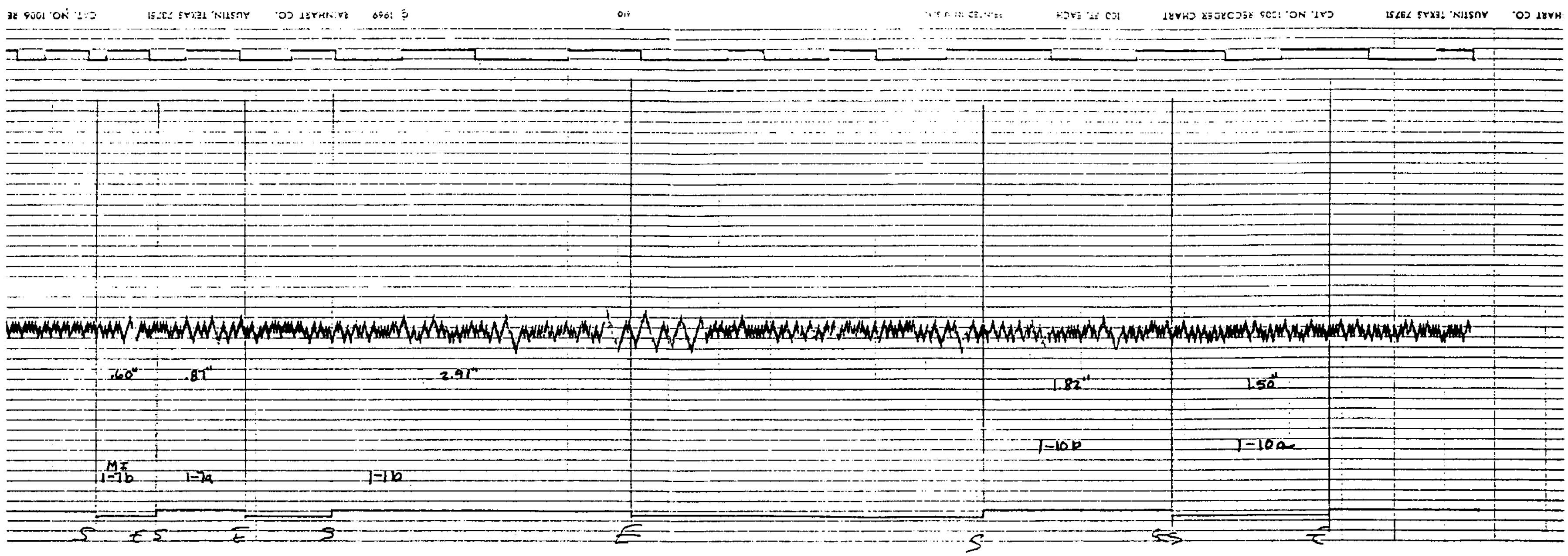


Figure 9. Sample raw output from Mays Roughness Meter.

7. SUGGESTIONS FOR FUTURE DATA COLLECTION AND TESTING

Every effort was made to make the data collection and testing procedures outlined in this chapter as complete and comprehensive as possible. However, as is often the case, several uncollected data items and unperformed tests were later identified which would have contributed to the performance evaluation of the projects. These items include:

- Actual field measurements of the transverse and longitudinal joint depths.
- Mid-slab edge deflections with the FWD.
- Measurements of the volume of the cores before and after subjecting them to heating and cooling to determine the thermal coefficient of expansion.
- Distinction between deformed and smooth welded wire reinforcement.
- Site-specific temperature data for the months of the concrete paving and curing.

CHAPTER 2 WEIGH-IN-MOTION DATA COLLECTION PROCEDURES

1. INTRODUCTION

As part of this study, truck weight and vehicle classification data was collected for selected sections using weigh-in-motion (WIM) equipment. The location of the 12 sites that were monitored with the automated equipment are shown in table 1.

The weight data was obtained from the Streeter Richardson portable weigh-in-motion (WIM) system using a capacitive mat as its weight transducer. The WIM transducer was placed in one wheelpath of the outside lane in the direction of travel specified by the sponsor. Temporary inductive loops were placed both upstream and downstream of the capacitive speeds. The data was collected for 48 continuous hours under the monitoring of a data collection technician.

In addition, vehicle classification data was acquired using automated vehicle classification (AVC) equipment in conjunction with pneumatic tube axle sensors for 48 continuous hours. Two axle sensors were used with the AVC equipment to provide data from which to calculate vehicle speeds and axle spacings which are the basis for the vehicle classification criteria defined by the Federal Highway Administration (FHWA). The FHWA vehicle classification categories listed in the Traffic Monitoring Guide were used in the study.(5) These are tabulated in table 2.

2. SITE SETUP AND CALIBRATION

In every case, the setup of the site for data collection was preceded by coordination with the local maintenance office so that they could provide work zone traffic control. The WIM sensor deployment was accomplished by marking the proper location for each sensor and then fixing it to the pavement surface. For the temporary inductive loops, this involved removing the backing material from each preformed loop and applying the loop to the pavement surface. The capacitive mat was installed by laying it on an adhesive pad that had been cut to its dimensions. Nails were used to hold the mat securely to the pavement surface.

Once the WIM system sensors were installed, they were connected to the electronic data collection system located in a van parked on the roadside. A single unit truck provided by the local maintenance yard was then used in the calibration process. The gross weight of the calibration vehicle weight was measured at a static scale. The calibration vehicle was then driven through the WIM sensor array several times at the posted speed limit. The mean of the gross weights for the calibration vehicle obtained with the WIM system was computed. The WIM system calibration parameter was then adjusted to produce WIM readings which, on average, correspond to the measured static reading for the gross weight of that vehicle. Whenever possible, vehicles with known static gross vehicle weights were compared with their gross vehicle weights obtained using the WIM system.

Table 1. WIM project locations.

1. Michigan 1 - U.S. 10, Clare
2. Michigan 4 - Interstate 69, Charlotte
3. Michigan 5 - Interstate 94, Paw Paw
4. New York 1 - Route 23, Catskill
5. New York 2 - Interstate 88, Otego
6. Ohio 1 - U.S. 23, Chillicothe
7. Minnesota 1 - Interstate 94, Rothsay
8. Minnesota 2 - Interstate 90, Albert Lea
9. Minnesota 3 - Interstate 90, Austin
10. Minnesota 4 - Trunk Highway 15, New Ulm
11. Pennsylvania 1 - Routes 66/422, Kittanning
12. New Jersey 3 - Interstate 676, Camden

Table 2. FHWA vehicle classification types.(5)

FHWA Classification Type	Description
1	Motorcycles (optional)
2	Passenger Cars
3	Other Two-Axle, Four-Tire Single Unit Vehicles
4	Buses
5	Two-Axle, Six-Tire Single Unit Trucks
6	Three-Axle Single Unit Trucks
7	Four or More Axle Single Unit Trucks
8	Four or Less Axle Single Trailer Trucks
9	Five-Axle Single Trailer Trucks
10	Six or More Axle Single Trailer Trucks
11	Five or Less Axle Multi-Trailer Trucks
12	Six-Axle Multi-Trailer Trucks
13	Seven or More Axle Multi-Trailer Trucks

The pair of pneumatic tubes for vehicle classification was placed in each lane perpendicular to traffic with a separation of 20 ft (6.1 m). The sensors were then connected to the pneumatic tube connectors on the AVC unit. The AVC unit was then programmed to collect data for 48 continuous hours. Hourly summaries of the total number of each type of vehicle in each lane were stored. The performance of the AVC units were verified periodically by the WIM system operator.

The field procedures involved in the collection of WIM data are summarized in table 3.

3. ACCURACY OF WIM DATA

Although no study was carried out to assess the accuracy of the WIM data acquired in this study, statements can be made about the nature of the data collected using the Streeter Richardson portable WIM system. In other research, it has been shown that the systematic error (i.e., the mean of the observed percent differences between static weighting and WIM results) is less than 4 percent for gross vehicle weights. It has also been shown that the random error (i.e., the standard deviation of the observed percent differences between static weighing and WIM results) is generally less than 6 percent for gross vehicle weights.

The application of the truck weight data to computing 18-kip (80 kN) Equivalent Single-Axle Loads (ESAL) applications is straightforward but requires an understanding of the phenomena that are being measured by the WIM equipment. That is, the WIM transducer accurately indicates the force applied to it. If the weight applied to a static scale is also statically applied to the WIM transducer then the WIM system can be calibrated to yield a very accurate (within 0.1%) reading. However, excitations to the truck suspension systems due to the road profile as well as other perturbations prevent the force actually applied to the WIM transducer from being the same as that that would be applied to the static scale. Since accuracy is determined from paired comparison of static and WIM weighing results, it is not possible to obtain very low accuracy results unless the approach profile is very smooth and the trucks are in excellent condition. These conditions are rarely present on U.S. highways.

The issue of whether WIM equipment or other data collection means should be used to determine the number of ESAL's on a specific highway section is an important one. The only significant obstacle to doing so is the fact that the ESAL values used in design procedures are based on static weight values. It will be necessary to develop pavement design procedures based on dynamic loading before WIM data can be applied directly to the process. Meanwhile, there exists no better data to be used. In fact, the FHWA has notified the States that it will discontinue acceptance of data not collected using WIM systems. The reason for this is that data collected using static scales does not include those vehicles that are loaded in excess of legal limits since these vehicles will make every effort to avoid the scales. WIM weighing is clearly preferable to static weighing for obtaining a true sample of the weights of the truck population.

Table 3. Field procedures for collection of WIM data.

I. PRELIMINARY ELEMENTS IN DATA COLLECTION ACTIVITIES

A. Contacted Traffic Control (TC). Minimum advance notice of three days typical.

B. Inspected all equipment to ensure in proper working condition.

C. Met TC at their office and proceed to the site together.

II. THE SETUP PROCEDURE

A. TC would block lane 1.

B. Laid temporary loops.

C. Placed mat 1 ft (305 mm) from the entrance loop.

1. For PCC pavements, the Hilte gun was used to secure the mat.
2. For AC pavements, nails were used to secure the mat.
3. Connected the WIM System and checked for communication problems, such as proper cable hookup or oscillator malfunction.

D. Laid the tubes for lane 1.

1. For PCC pavements, the Hilte gun was used along with concrete nails and bituthene.
2. For AC pavements, nails were used exclusively.
3. Each lane had two tubes 20 ft (6.1 m) apart connected to a Streeter 241 WIM device.
4. Only one lane had a WIM System but all had tubes for classification.
5. The 241's were placed opposite of each other 10 ft (3.0 m) from the shoulders.

E. Highways with a middle lane on either side.

1. Tapeswitches were used with the same setup as the tubes.
2. The tapeswitches were installed using bituthene.

F. All of the systems were checked carefully before TC was released.

G. The WIM system was then calibrated using a van with measured axle spacings and weight.

1. The van would be driven over the system 10 times at 55 mi/h (88 km/h).
2. The system was calibrated each time the van passed by using a handheld calculator to calculate the percentage differences.
3. The speed, length, and the weight were set according to specifications.

Table 3. Field procedures for collection of WIM data (cont'd).

4. The front axle weights of 3-S2's were then closely monitored for at least an hour. The front axles of 3-S2's typically weigh 9000-11,000 pounds.
5. The spacings of the tandems were also monitored for the first hour.
6. If the system appeared to be measuring either too high or too low, the calibration would be checked in the same manner as the initial calibration.

III. **MAINTENANCE AND OPERATION**

A. Vests worn at all times.

B. Checked the 241's every 30 minutes.

1. The batteries had to have a 5.5 volt charge or a charged 241 would be installed in its place.
2. Each 241 had a built-in computer which allows classifications to be checked while the system was in operation.
3. Tubes were reset if loosened from the highway.
4. Broken tubes were replaced when support was available.

C. WIM loop and mat malfunctions.

1. Loops would hang up when the leads were too long and when the connections were directly exposed to water.
2. Vibrations of the road loosened the oscillator cover and in turn the oscillator.
3. All cable connections were required to be kept dry.

D. Every 4 hours, crew members not on duty checked the station.

IV. **SYSTEM REMOVAL**

A. One person stored the data from the WIM and the 241 systems.

1. WIM data was stored by day each on a separate floppy and the 241 data was stored on cartridges according to lane.

B. Other crew members removed the mat and tubes.

C. The loops were left but the leads were cut.

D. All of the materials were removed from the highway and the trash was discarded before TC was released.

E. Loaded up equipment and traveled to the next location.

V. **POST DATA COLLECTION ACTIVITIES**

A. All cartridge data was placed on floppy at the hotel using TrafcompIII.

Given the decision of the FHWA to accept only WIM data in the future, this data will be used to prepare the W-4 tables produced as a result of the Truck Weight Study. It is important to note that the system of computer programs used by the FHWA to process the truck weight data has been adapted to IBM PC-compatible microcomputers and is now available. The WIM data obtained at any location can be input to this battery of programs to produce the W-4 tables for that location.

CHAPTER 3 TRAFFIC ANALYSIS

1. INTRODUCTION

In order to evaluate the overall performance of a pavement, estimates of the number of traffic loadings sustained by a pavement over its performance period must be known. The gauge used to measure the amount of traffic loading has traditionally been the number of 18-kip (80 kN) single-axle load applications. However, not all trucks are 18-kip (80 kN) single-axles, so conversion factors are necessary to convert mixed traffic with various axles and axle loadings to the standard axle and axle loading. These factors, developed at the AASHO Road Test, allow for the calculation of the number of 18-kip (80 kN) equivalent single-axle load (ESAL) applications. This concept is extensively used in pavement design and in evaluating pavement performance.

There are several procedures and algorithms available for computing the number of 18-kip (80 kN) ESAL applications from given traffic parameters. A rigorous evaluation would involve the contribution from each individual truck classification. In this way, the relative contributions of each truck classification would be very clear and a more accurate number would result. However, for this project a more general and rapid calculation procedure was used for the estimation of traffic loading due to time and budget constraints The methodology of this calculation procedure is presented in this chapter.

2. INPUTS FOR PROCEDURE

There are six inputs required for the calculation procedure. To estimate the cumulative ESAL applications, the calculation procedure requires historical data for each input (i.e., a separate input for each year that traffic loadings are being calculated). Often, this information may not be available and estimates or interpolations must be made.

Average Daily Traffic, ADT

The traffic volume on a given roadway for a given year is represented by the two-way Average Daily Traffic (ADT). This input is readily available from the States as the collection and processing of traffic volume data is well-established.

Percentage of Heavy Trucks, TKS

The majority of ESAL loadings are produced by trucks. Automobiles are ignored in the calculation as their contribution is negligible. Thus, the percentage of heavy trucks which significantly contribute to the number of ESAL applications must be known.

The percentage of heavy trucks is readily available from State highway agencies. For the purposes of this study, all heavy trucks were considered to be those in FHWA vehicle classifications 5 through 13 (see reference 5 for more information on FHWA vehicle classifications). It is important that the percentage excludes panels and pickups as these vehicle classes do not contribute significantly to the number of ESAL applications.

Directional Distribution, DD

Occasionally, a roadway may have more traffic traveling in one direction than the other. The purpose of the directional distribution is to account for traffic volume differences by direction.

State traffic studies may be available which indicate a directional difference in traffic. However, in most instances, it can be assumed that there is no difference in directional distribution. Thus, unless there are studies to show otherwise, a directional distribution of 0.5 should be assumed.

Lane Distribution, LD

The lane distribution is a factor which accounts for the percentage of heavy trucks in each lane. The calculation procedure evaluates only one traffic lane at a time, so that the percentage of heavy trucks in each traffic lane must be known.

Lane distribution information may be available from traffic monitoring data. However, this information is often not available and may have to be estimated. An empirical regression equation is available that estimates the lane distribution factor based on the ADT and the number of lanes.(6)

If the roadway is a two-lane highway (i.e., one lane in each direction), then a lane distribution factor of 1 should be used.

Average Truck Factor, TF

The average truck factor is simply the average number of ESAL applications per truck. This factor is estimated from historical W-4 tables. A computer program was furnished by FHWA which generated W-4 tables by year for each State in the study.(7) The W-4 tables were further generated by functional classification for each project in the State.

Using the conversion factors developed at the AASHTO Road Test, the W-4 tables provide information on the number of 18-kip (80 kN) ESAL's by truck for each truck classification. The average truck factor is then calculated as a weighted average of the number of ESAL's for each individual truck classification.

However, it should be noted that the information in W-4 tables is based on static weighing methods. It has been shown in several studies that static truck weighing operations nearly always provide truck weight data in which the overloaded vehicles are significantly underrepresented.(8) Thus, it is necessary to adjust the average truck factor to reflect the probable truck factor. To accomplish this, information was used from the Weigh-in-Motion (WIM) data to adjust the truck factors. Using the WIM data, an average truck factor was determined. An adjustment factor was determined by dividing the 1987 WIM average truck factor by the 1987 W-4 average truck factor. This adjustment factor was then applied to all of the historical W-4 truck factors to account for the overloaded trucks not represented in the W-4 tables.

3. CALCULATION PROCEDURE

The calculation procedure used in this study is simple and straightforward. The equation used in estimating the number of ESAL applications is given below. This equation calculates the number of ESAL applications accumulated in 1 year for one lane of a given roadway.

$$ESAL_{ij} = ADT_j * TKS_j * DD_j * LD_{ij} * TF_j * 365 \qquad (2)$$

where:

$ESAL_{ij}$ = Number of 18-kip ESAL applications sustained by lane i in year j

ADT_j = Two-way Average Daily Traffic for year j

TKS_j = Percent Heavy Trucks for year j

DD_j = Directional Distribution of traffic for year j

LD_{ij} = Lane Distribution of trucks in lane i for year j

TF_j = Average Truck Factor for year j

This calculation provides an estimate of the number of ESAL's sustained by lane i of the pavement in year j. To obtain the cumulative ESAL applications for lane i, this calculation is performed for every year and then added together. This procedure can easily be established in a worksheet file so that the calculation procedure can be automated. This will also enable the user to investigate the relative impacts of the various inputs on the calculated ESAL's.

Weigh-in-motion (WIM) data was available for 12 sites. Data collected included traffic counts (by lane and by direction) and axle load weights. For each site for which this data was collected, it was used to adjust the various inputs in the ESAL calculation equation. In addition, average truck factors were adjusted using the WIM data to provide for a more realistic assessment of the actual truck traffic loadings.

4. SUMMARY

The calculation procedure presented herein is simple and straightforward. It provides a means to quickly estimate the cumulative ESAL sustained by a pavement. However, it should be noted that the calculation assumes an average truck factor which represents an average ESAL loading per truck. In some cases, this simplifying assumption may not be appropriate. Where possible, the truck factors should be broken down by vehicle classification for more accurate results; this will require very detailed truck classification data for each year of calculation.

CHAPTER 4 DRAINAGE ANALYSIS AND DETERMINATION OF DRAINAGE COEFFICIENTS

1. INTRODUCTION

Drainage has been shown to have a large impact on the performance of concrete pavements. It is an accepted fact that excess moisture, if not quickly removed from the pavement system, can produce premature failure in an otherwise well-designed pavement. A drainage analysis is generally recommended for both new pavement design and rehabilitation design to establish the overall drainability of the pavement section. In this way, drainage problems can be identified and addressed <u>before</u> the construction or rehabilitation of the pavement. Detailed information on drainage analysis procedures and their importance on concrete pavement design are discussed in references 9, 10, and 11.

In acknowledgment of the importance of moisture effects on pavement performance, one addition that was made to the 1986 AASHTO Design Procedure was the inclusion of a drainage coefficient.(12) The drainage coefficient itself was designed to provide an adjustment to the structural design of a pavement to account for the impact of drainage on the life of the pavement. This would assist the engineer in analyzing existing pavements to better explain why premature deterioration could be found in some pavements but not in others. A pavement with a high drainage coefficient would be expected to sustain higher traffic loadings than a similar pavement with a low drainage coefficient.

The drainage coefficient merely reduces or increases the total traffic that the pavement can carry before reaching a terminal serviceability level. In the development phase of the drainage coefficient for rigid pavements, it was applied to the modulus of rupture of the concrete to produce different thicknesses representing a reduced ability to carry traffic. This was further related to modulus of subgrade reaction to illustrate a corresponding level of subgrade support that would produce the same loss in life for the pavement. However, these values of subgrade support are unattainable in the field even for the worst and best moisture conditions that may be found. This complicates the assignment of a drainage coefficient by a rational procedure considering material properties that truly reflects the performance of the inservice pavement.

To develop a true meaning of the drainage coefficients, it is necessary to examine their basis for development. The AASHO materials, the designs, and the traffic levels predicted from the design equations were assigned the arbitrary value of "good." All adjustments for improved behavior as compared to the AASHO predicted lives are based on material combinations that are either better or worse than the AASHO road test. Thus, if better draining materials are used in a pavement, the drainage coefficient should be greater than one. If materials are used which are worse than the AASHO materials, the drainage coefficient should be less than one. If a value is selected that is equal to one, it means the pavement designed using these values can be expected to perform as well pavements designed in the past using the old AASHO design procedure.

The selection of these coefficients can have a pronounced effect on the final pavement thicknesses designed using the 1986 procedure. An important factor such as this should have a set of well-defined rules to guide their selection. However, while the revisions did provide criteria to use in selecting a drainage coefficient, they did not provide a rational means for developing the criteria based on the pavement properties. The steps described in this section will present a rational procedure to determine a combined "whole pavement" drainage coefficient which represents the impact of drainage on the potential life of the pavement being analyzed. The procedure generally follows the guidelines of the AASHTO guide, but it includes the subgrade effect on drainage, the effect of the cross section of the pavement, and material type placement within the cross section.

2. AASHTO DRAINAGE CRITERIA

The selection of a drainage coefficient in the 1986 AASHTO design procedure requires only two items to be selected. These items are:

1. The time the pavement is exposed to moisture levels approaching saturation.

2. The time of drainage for the aggregate layers in the pavement.

These two parameters recognize the two important areas of moisture influence on pavement performance. First, there must be sufficient water present from the environment to promote moisture related problems in the pavement. Secondly, the materials in the pavement may or may not be prone to moisture problems, even if moisture is present. While these are very important considerations, they by no means represent the whole impact of moisture on pavement performance, and additional considerations must be added to ensure the entire pavement structure is considered in the evaluation of the potential for moisture damage to develop. The roadbed soils are only indirectly considered, and the impact of the pavement cross section and materials selected are not considered even though they have repeatedly been shown to influence performance of the pavements in the presence of moisture.

Roadbed Soils

The drainage characteristics of the roadbed soil are not considered in the drainage coefficient selection. The "Effective Roadbed Soil Modulus" developed from monthly or seasonal estimates of the variability in modulus of the roadbed soil is supposed to take care of the reduction in pavement support when the roadbed soil is saturated at different times throughout the year. This reliance on a seasonal structural relationship ignores the drainage effect the roadbed soil has during each season when water may enter the pavement system. It is important to connect the influence of the roadbed soil on moisture behavior to the selection procedures for the drainage coefficients currently in the AASHTO procedure.

Cross Section

There is no consideration of different material arrangements in the current selection criteria. When stabilized materials are used there is no selection of a drainage coefficient for that pavement, even though the material may have a large impact on the moisture related performance of the pavement and may actually reduce the benefits of positive draining layers on the long-term performance of the pavement. "Bathtub" sections are an example of material combinations which trap moisture in direct contact with the upper structural layers, principally portland cement concrete. Bathtub sections are very susceptible to moisture damage over the long term. Positive drainage considerations are required in these sections to reduce the time moisture is trapped in the upper portion of the structure. A poor draining cross section need not have the traditional bathtub construction, however. An example of this is the use of a dense, granular base material which traps moisture in contact with the pavement for sufficiently long periods.

The use of stabilized materials can promote moisture damage, particularly in rigid pavements where the materials trap surface infiltration in the upper portion of the pavement. This promotes faulting and pumping in the pavement, which can be evaluated with the Loss of Support factor, but which is more correctly a moisture related problem of these materials, and should be included in the drainage analysis. Permeable base layers placed below an impermeable stabilized material will not assist in removing surface infiltration, and this moisture can accelerate damage through pumping or faulting in the portland cement concrete slabs. In this manner, the benefit of the permeable base layer is minimized, and should be suitably discounted.

The use of permeable base layers directly beneath the slab has become more widespread. Such layers, either stabilized or nonstabilized, are open-graded and contain very few fines. They are expected to quickly remove water from the pavement structure, thus decreasing the amount of time that the pavement structure is in a saturated state. In this way, the presence of the permeable base layer will prevent or reduce the amount of moisture-associated distress.

In the design of pavement sections with permeable base layers, it is imperative that a filter material be placed beneath the permeable base. This layer will prevent the intrusion of fines into the permeable base layer so that it will remain free-draining. However, for a filter layer to function properly, its aggregate gradation must meet certain criteria relative to the gradation of the permeable base and the subgrade. Such criteria are given in references 13 and 14.

3. MAD INDEX

A rational procedure to evaluate material properties of the pavement layers as they relate to moisture damage, and the climatic inputs was initially developed in 1980 as part of an FHWA sponsored research project which evaluated climatic effects on drainage and shoulder performance.(15) The <u>M</u>oisture <u>A</u>ccelerated <u>D</u>istress Index (MAD Index) is a combination of climate and material properties that use numerical procedures to evaluate the potential for moisture problems to develop in a particular pavement given its construction. This procedure is ideally suited to developing drainage coefficients which are based on the material's impact on pavement performance in a given climatic area. The MAD Index procedure relies on two major calculation schemes, the drainability of a base material, and the moisture availability in the general climatic area, and a third value to include the influence of the subgrade soil.

Base Drainability

This factor is a calculation of the time for a base course material to drain to a certain level of saturation, which indicates a drainability quality level of the material. The saturation level of a material has been shown to relate directly to performance of the material under repetitive loadings.(16) The longer a material remains at high levels of saturation, the more likely it is to develop increased damage from traffic. The faster a material drains, the less damage will occur.

Climatic Moisture

The second calculation determines the moisture availability from climatic variables using monthly temperature and rainfall data. This is not a calculation scheme to determine inflow in to a pavement. It is a general guide to accurately determine the moisture availability in a soil over a year. This can be influenced by local conditions such as high water tables, springs, seepage, and so on, all of which must be known to determine water inflow to design drainage systems.

Subgrade

The influence of the subgrade on moisture damage is included in the MAD Index procedure by recognizing the natural drainage characteristics of the existing soil upon which the pavement is constructed. This Natural Drainage Index (NDI) can be obtained from County Soil Maps, which are available for most counties in the United States. Additionally, many agencies have extensive soil mapping programs which have developed an extensive catalog of soil properties. All of these should be consulted in the evaluation. Estimating procedures can be used when maps are not available. Cut and fill sections can be analyzed when the borrow area is known and the soil properties from that area can be determined. Cut sections generally have the properties of the deeper soil horizons, which are obtained from the soil maps as the parent material.

MAD Index Categories

The MAD Index was established as a selection procedure for pavement performance as related to moisture, in a general manner, and there were no selection procedures developed to compare specific pavements in a quantifiable manner. This section presents procedures to quantify pavement differences arising from moisture problems. They are compared to the AASHTO criteria to show how they can be used to develop the drainage coefficients.

<u>Base Drainage</u>

The base drainability was originally set into three categories depending on how long the granular material would require to drain to 85 percent saturation if initially saturated (following a rainfall). The longer this time, the longer the material would hold moisture and be susceptible to traffic damage. The levels were:

1. < 5 hours - <u>a</u>cceptable - a
2. 5 - 10 hours - <u>m</u>arginal - m
3. >10 hours - <u>u</u>nacceptable - u

The 5-hour limit is generally regarded as the critical time for a material exposed to Interstate levels of traffic that drains well, and, more importantly, does not retain moisture after the rain stops. Pavements with lower levels of traffic can have longer drainage times without sustaining the same amount of damage.

Subgrade Drainage

The natural drainage index (NDI) was established by soil scientists to represent the drainage characteristics of the existing soil structure, and the impact moisture had on the formation of the soil. Hole, in his study of soil types related to pavement performance, assigned numerical values to each category.(17) The seven categories of drainability and the associated NDI proposed by Hole are:(17)

1.	Excessively Drained	-10
2.	Somewhat Excessively Drained	-2
3.	Well-Drained	1
4.	Moderately Well-Drained	2.5
5.	Somewhat Poorly Drained (Imperfect)	4
6.	Poorly Drained	5.5
7.	Very Poorly Drained (Local Conditions)	7 - 9
8.	Bog Soil	10

In the MAD Index scheme, NDI values were grouped into the following categories (15) as relates to their drainage and pavement support capacity:

1.	-10 to -2	Excellent	- k
2.	-2 to 2.5	Average	- j
3.	2.5 to 10	Poor	- i

Climatic Moisture Availability

In the MAD Index procedure, the availability of moisture is established with the Thornthwaite Moisture Index (TMI).(18) This is a climatic classification parameter obtained from average monthly temperatures and rainfall data. It represents the yearly average condition of moisture in the general area of the pavement, and not specifically the moisture in a pavement. As such, it provides an indicator for potential moisture availability, a parameter useful in design or general analysis for moisture-related problems. The TMI is the balance of rainfall and Potential Evapotranspiration (PET). PET is the total amount of moisture that would be evaporated if it were present. Over a year, the PET indicates the general excess or lack of moisture in an area.

The MAD scheme resulted in a climatic map shown in figure 10, which breaks the United States into nine climatic zones based on temperature, moisture availability, and their influence on performance of the pavement. These same considerations, although not numerically quantifiable, were used to develop the six AASHTO climatic zones also shown in figure 10. Climatic zones are important in evaluating the potential for moisture damage to occur, as temperature interacts with moisture differently depending on which environmental zone the pavement is in. The FHWA zones represent the following influences on pavement performance:

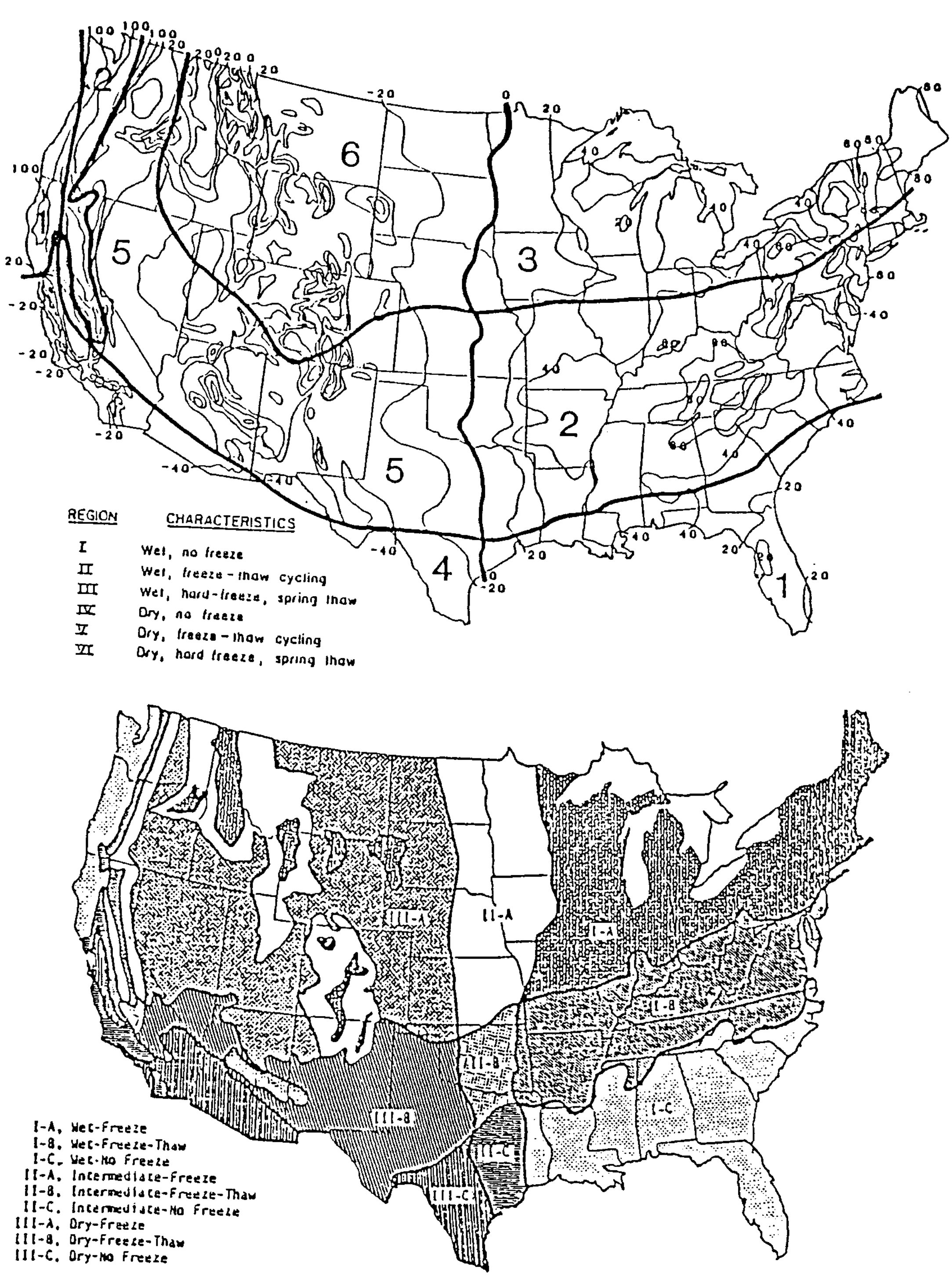


Figure 10. Climatic zones for AASHTO and FHWA procedures.

1. Zone I — Low temperatures during the winter with the subgrade being frozen to an appreciable extent, and frost heave and spring thaw problems developing in susceptible soils.

2. Zone II — Freeze-thaw area with cyclical temperature extremes. The subgrade occasionally freezes, and frost heave problems and corresponding spring thaw problems are lessened.

3. Zone III — Area of little low temperature activity in the winter; high temperature problems in summer months may present a problem.

4. Zone A — This is a zone of surplus moisture over a year's period. The subgrade will be at high levels of saturation for the greatest amount of time in these areas.

5. Zone B — This is a zone of statistical uncertainty in performance. In any 1 year the moisture conditions may be dry or wet, and the performance will vary accordingly in these areas.

6. Zone C — This area is a zone of moisture deficit on the average. While there may be periods during the year when moisture surplus is found, it is far outweighed by the period when there is a deficit.

This map was coupled with the base and subgrade drainability classification to result in the MAD classification scheme.(15) These procedures form the basis for the procedure presented in this report to select suitable drainage coefficients for a pavement.

4. DEVELOPING A DRAINAGE COEFFICIENT

Time of Saturation

The Thornthwaite Moisture Index calculation procedure uses monthly average temperature and rainfall data along with the latitude of the pavement to calculate the Potential Evapotranspiration (PET). Evapotranspiration is the moisture removed from a soil by evaporation and by plant transpiration. It is called potential since it represents the amount that would be removed if there were an inexhaustible source of water. In cold periods the value is low, while in hot periods there would be a great deal of evaporation, except that it is likely that all of the soil moisture would eventually be removed and evaporation would stop.

The resulting data is a table comparing rainfall and PET. The balance of these two indicates whether there will be water entering the soil as rainfall, or leaving by evaporation. As water enters the soil, the soil will store moisture. Soils have a finite capacity to store moisture, and while variable, it can be approximated at about 4 in (102 mm) of water. When the soil has this much water in its structure any more water will runoff and the soil is said to have a surplus of moisture available. As water stored in the soil evaporates, and PET exceeds rainfall, the storage is removed from the soil. When all the moisture in the storage has been removed, the soil is in a deficit stage, and it will be continually dry.

There are nine possible moisture conditions which can occur throughout the United States. For each of these nine conditions, the pavement is expected to be saturated for a different amount of time. The amount of time that a pavement is saturated is a required input for calculation of the AASHTO drainage coefficient. The nine moisture conditions are:

1. Frozen period. When the average monthly temperature is below freezing, no contribution to saturation can be made, regardless of moisture conditions.

2. Surplus period following the winter. The soil will be continually saturated during this period especially in Zone A. In Zone A all months will contribute to the saturation time.

3. Surplus period following the winter in Zone B. Here the spring thaw phenomenon is not as critical. There may be periods where the soil is not totally saturated, and these should correspond to dry days in the "rain and dry day" sequences. In Zone B, include the first month and one-third of remaining months in the saturation time.

4. Surplus following a recharge which does not follow a frozen period. Here one-third of the months in the surplus period should contribute to the saturation time.

5. Utilization period following a surplus. During this period, evaporation exceeds rainfall, and the storage moisture is being depleted. The soil is going from a saturated to a dry condition. The initial month may have a portion during which it is considered saturated. Include one-third of all months which have a storage exceeding 3 in (76 mm), representing wet months with rain.

6. Utilization after a recharge which did not lead to a surplus. None of the time in this moisture state contributes to saturation time.

7. Recharge leading to a surplus. The final months may contribute to saturation. Include one-third of all months which have storage values above 3 in (76 mm).

8. Recharge leading to no surplus. This period will contribute little. Include none of the time during this period toward saturation time.

9. Deficit. During this time, there is no water available at all, and if there were to be rainfall, it would evaporate before entering the soil. Include none of this period in the saturation time.

A plot of the monthly rainfall and the monthly PET calculated by the Thornthwaite procedure provides a calculation scheme to provide a numerical, repeatable procedure to show the time a pavement is exposed to moisture levels approaching saturation. An example is shown in figure 11 for Clare, Michigan.

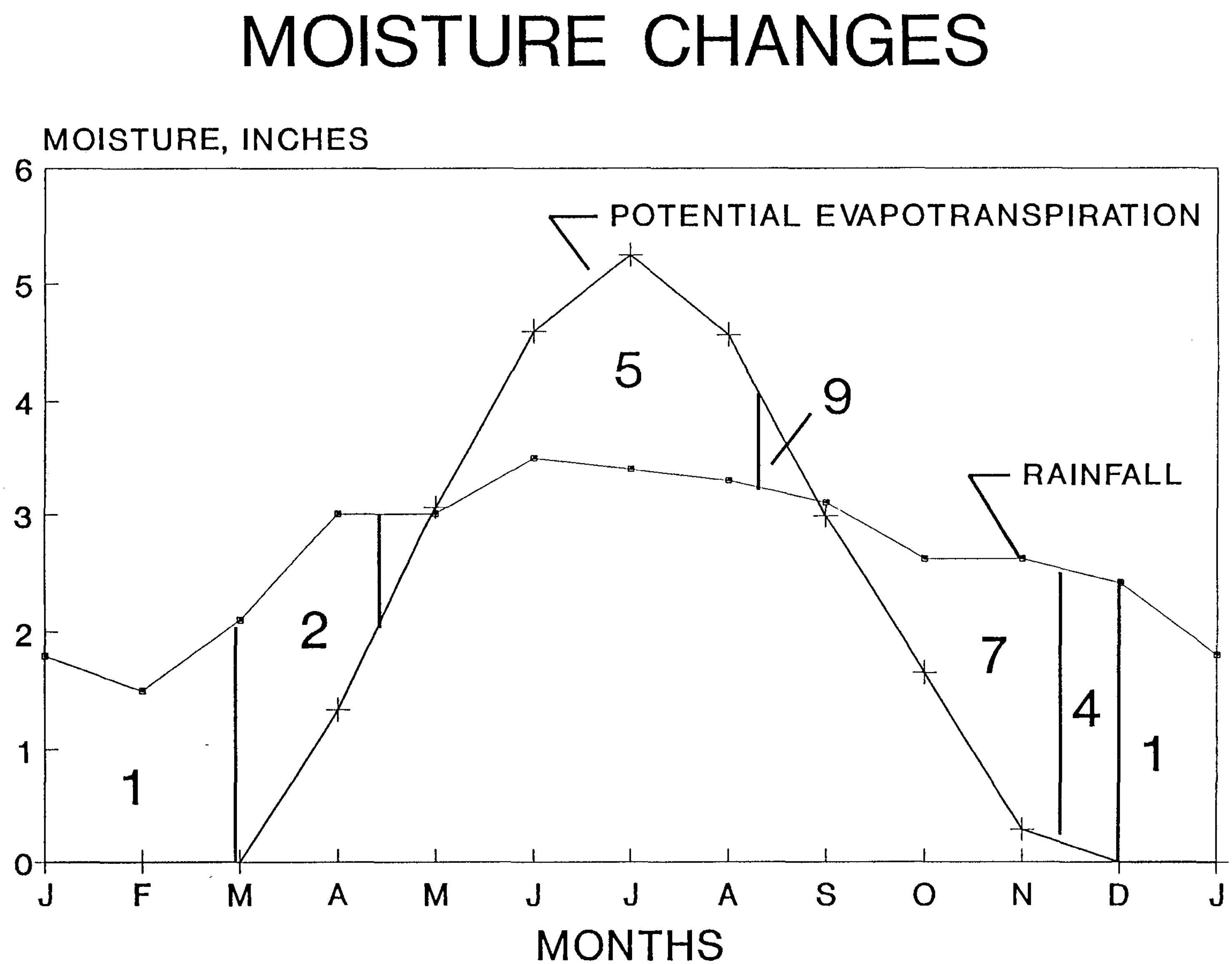


Figure 11. Plot of monthly rainfall and potential evapotranspiration for Clare, Michigan.

Base Drainability

The initial calculation scheme for base drainage times used in the MAD Index procedure was set up using the Casagrande-Shannon procedure, extended by including soil properties to convert time to drain water to time to reach specific saturation levels. It is the saturation level in a granular material that relates to performance under loadings, not just the physical moisture content. This calculation procedure assumes the subgrade to be impermeable and not to assist in draining the base. It also does not allow analysis of the situation where a granular base is placed over a permeable subbase. The permeable subbase (or subgrade) will assist the base by shortening the drainage time, but the combination cannot be predicted using the original procedure.

In their work examining infiltration effects, Liu and Lytton derived expressions for the drainage characteristics of a base placed over a permeable subgrade.(19) This procedure should be used to analyze a base over a more permeable subbase to determine the impact of a permeable material on base drainage times.

The data elements required to calculate drainage times include the following material properties and cross section data:

- Gradation
  - Percent Fines (-#200)
  - D_{10} Effective Grain Size.
- Dry Density, as Compacted (not laboratory maximum).
- Specific Gravity of Solids.
- Base Thickness.
- Longitudinal Slope.
- Transverse Slope.
- Length of Drainage Path.

These values can be used in the statistical relationships presented in the FHWA Highway Subdrainage Manual to obtain estimates of permeability and effective yield, which are in turn used to calculate a table of saturation levels and time in hours.(13). This is shown in table 4.

The criteria for judging the drainability of a granular base as defined by the three categories developed for the MAD Index do not precisely fit with the AASHTO categories of base drainage, which are:

1.	Excellent	2 hours
2.	Good	1 day
3.	Fair	1 week
4.	Poor	1 month
5.	Very Poor	Water Will Not Drain

These time requirements are based on the time it takes a material to lose whatever water it will drain. It does not recognize that the material will retain moisture within its soil structure, and this value may be high, often above 85 percent saturation. The following times have been determined for approximate comparison with the recommended AASHTO criteria, while maintaining the relationship in the MAD Index procedure:

Table 4. Chart to calculate saturation time curve.

Pavement Section ______

Percent Fines ______ Type of Fines ______

D_{10} (mm) ______ Density, γ_{dry} (pcf) ______

G_s ______ k (ft/day) ______

H (ft) ______ D (ft) ______ gt (ft/ft) ______

gl (ft/ft) ______ $Le = D\sqrt{(gl/gt)^2+1} =$ ______

$ge = \sqrt{gl^2+gt^2} =$ ______ $S = H/[(Le)(ge)] =$ ______

A (g/cc) $= \gamma_d/62.5 =$ ______ $V_t = 1.0$ cc

W_s (g) = A = ______ $X = (Ne\ D^2) / (H\ k)$

V_s (cc) $= W_s/G_s =$ ______

V_v (cc) $= 1.0 - V_s =$ ______ $= Ne_{max}$

Estimated Water Loss from Figure 10 = C (%) = ______

Specific Yield = Ne = V_v x (C/100) = ______ x ______/100 = ______

(1) U	(2) T	(t days) (2) x X	(t hours) t x 24	(3) Ne x U	(4) Vv - (3)	(5) (4)/B	S=(5)x100
.1							
.2							
.3							
.4							
.5							
.6							
.7							
.8							
.9							

1. < 5 hours, Exceptional - e
2. 5 to $\leq$ 30 hours, Acceptable - a
3. 30 to $\leq$ 100 hours, Marginal - m
4. 100 to $\leq$ 200 hours, Poor - p
5. > 200 hours, Unacceptable - u

The category of Poor is a special breakdown of the marginal drainage time. When it is calculated, it requires applying the lowest drainage category arrived at for the various combinations. The category of Exceptional requires applying the next highest drainage factor, when a final range is arrived at using the other pavement variables. This will be explained later.

Subgrade Drainability

County soil maps do a good job in classifying the soils found under a pavement in terms of their soil properties and drainability. Additional properties may be available as part of the DOT's soil mapping programs. All data sources should be investigated. It would take an exhaustive amount of testing to develop the data that is available in existing soil maps. Key items available include slope, permeability, hydrological group, and natural drainage class for NDI. All of these factors can be considered in deciding exactly what NDI rating the soil will be given.

Because the pavement will extend over numerous soil types, it is necessary to obtain a composite rating of the subgrade soil drainability over which the pavement passes. This can be constructed by summing the percentage of occurrence of each material multiplied by its selected NDI value. The sum of values provides a composite NDI value for the length of pavement being analyzed which can be directly compared to another section which may pass over different soil combinations.

Combining Base and Subgrade

It is the combination of base and subgrade that really determines the drainability of the entire pavement section being analyzed, and not just the base. The combination scheme shown in figure 12 represents a preliminary method to combine base and subgrade to arrive at the AASHTO drainage quality recommendations. The procedure to combine all these parameters is included in this figure.

Cross Section

The final item which must be considered is the cross section of the pavement itself. The two important items in the cross section are:

1. The materials,
   - stabilized impermeable,
   - unbound, and
   - permeable.
2. The arrangement of these materials in layers.

The following recommendations are made for adjusting the drainage coefficient obtained from the procedure outlined above.

		Base Drainabilities		
		A Acceptable	M Marginal	U Unacceptable
Subgrade Drainability	G Good k	EXC	G	F to P
	F Fair j	G	F	P to VP
	P Poor i	F to P	P to VP	VP

Quality of Drainage Criteria

EXC - Excellent
G - Good
F - Fair
P - Poor
VP - Very Poor

Figure 12. Procedure to combine drainabilities of base and subgrade.

1. If edge drains are present in the cross section, the coefficient can go to the highest value in the range obtained by the procedure here, or to 1.0. If a permeable base layer has been designed into the pavement, the drainage coefficient can go to the highest value in the range, and probably should be raised to 1.0 as it should remove all surface infiltration. These adjustments should be made only if the drainage provided by these materials is such that it drains the pavement, and is not cutoff from the pavement by an impermeable layer. The presence of a permeable base layer will improve the overall performance of the pavement to the highest value in the range specified. However, the impact of the permeable layer on the drainage coefficient will not be as great if the layer beneath the permeable base does not meet the recommended filter criteria provided in reference 13.

2. If the cross section is such that the pavement is completely contained in impermeable layers, and surface infiltration cannot exit the pavement, the coefficient goes to the low end of the range obtained, or even to a value of Poor, 0.8.

3. Because stabilized layers in a rigid pavement have not provided adequate relief from moisture related distresses such as pumping and faulting, they should be combined into the drainage coefficient, and not separated into the LOS factor. If the pavement has a cement-treated granular layer that cannot be analyzed with the drainage time scheme, that layer will be assigned the drainage coefficient of the lowest value in the category selected from the above procedure. The remaining factors of other materials in the pavement will raise or lower that value. An impermeable asphalt layer can be rated no higher then the midpoint of the lower range selected from the above procedures. Lean concrete base materials will be rated at the midpoint of the ranges selected above.

4. If the pavement has been constructed on a fill, the fill will generally not be a poor soil. If the borrow area can be located, and the soil type determined from the soil map, the NDI for that soil can be used. If the location of the fill material cannot be determined, the assumption can be made using average soil classification values for the length of the project.

5. DRAINAGE COEFFICIENTS FOR SECTIONS

The assignment of coefficients through the arbitrary, but sound, procedure outlined here must be tempered with normalized comparisons of performance of pavements constructed with varying quality of materials in different climatic regions. Only when the lives of these different pavements are compared, and the impact of the differing qualities of materials extracted, can the assignment procedure be fine-tuned to produce accurate coefficients. At present, excellent draining materials can have their coefficients reduced if the subgrade is poor or if the level of saturation time is great. The current procedure provides a preliminary estimate of the drainage coefficient, although it may be conservative. If it is conservative, it should remain so until the data is more thoroughly and completely analyzed.

This section contains a description of the selection of drainage coefficients (C_d) for each pavement in the database, following the procedures outlined described above.

Arizona 1

All of the sections of pavement studied on Route 360 in Tempe were built on well-drained subgrade soils that have an NDI of 1, putting them in the average (j) category. These sections are in an area that has the potential for being saturated 0 percent of the time. Section 1-7 was built in a 16- to 40-ft (4.9 to 12.2 m) cut; all the other sections were built in a 6- to 15-ft (1.8 to 4.6 m) cut. Section 1-7 was the only section that was built with edge drains.

These sections have only a marginal potential for holding water. Because the pavement has a 0 percent saturation time and the subgrade is well drained, these should help in taking moisture away from the pavement. The opportunity for a "bathtub" condition to exist is reduced.

AZ 1-1 was constructed with an impermeable CTB and a thin aggregate subbase. The CTB material lowers the coefficient. The subbase has a excellent drainage time of approximately 3 hours to reach 85 percent pavement saturation level. This gives it a rating of acceptable (a). This quality of drainage of this section was rated in the FAIR (1.15) to POOR (1.00) range, with the POOR value being selected due to the CTB. AZ 1-2, AZ 1-4, and AZ 1-5 were all constructed as slabs on grade, with no base or subbase. These sections were all rated FAIR (1.15 - 1.10).

AZ 1-6 and AZ 1-7 were both constructed with an impermeable LCB and no subbase. Both sections were in the FAIR (1.15) to GOOD (1.20) range with 1-7 at the higher end of the range because of its edge drains.

SECTION	C_d
AZ 1-1	1.00
AZ 1-2	1.10
AZ 1-4	1.10
AZ 1-5	1.10
AZ 1-6	1.10
AZ 1-7	1.15

Arizona 2

Section AZ 2 was constructed with an impermeable LCB placed directly on the subgrade. The subgrade is well-drained material with an NDI of 1, placing it in the average (j) category. The cross section of this pavement shows that this pavement has a slight possibility to perform as a "bathtub" section. However, since the pavement has the potential to be saturated 0 percent of the time and the subgrade is well drained, the possibility of a "bathtub" action is greatly reduced. This section is FAIR (1.15) to POOR (1.00), and is rated low because of the LCB.

SECTION	C_d
AZ 2	1.05

California 1

The five pavement sections studied near Tracy were all constructed on well drained soil with an NDI of 1. This places them in the average (j) category. These sections are in an area that has the potential for the pavement to be saturated approximately 0 percent of the time. The well-drained subgrade should help in removing any excess moisture from the pavement. No additional drainage features were added to any of the sections.

Sections 1-1, 1-3, 1-5, and 1-9 are all at grade with 5- to 7-ft (1.5 to 2.1 m) ditch lines. Section 1-7 is partly at grade and partly on a fill of 16- to 40-ft (4.9 to 12.2 m) with a 10-ft (3.0 m) ditch. CA 1-1, CA 1-3, CA 1-5, and CA 1-9 were all constructed with an impermeable CTB and two aggregate subbases 12-in (305 mm) thick. CA 1-7 was constructed with an impermeable LCB and also had two 12-in (305 mm) thick aggregate subbases.

The upper subbase for CA 1-1 was rated acceptable to marginal (a-m) with an 85 percent saturation time of approximately 27 hours. The lower subbase was rated acceptable (a), with a drainage time of less than 1 hour. The upper subbase for CA 1-3 was rated as marginal (m), with a drainage time of about 61 hours. The lower subbase was rated acceptable to marginal (a-m), with a drainage time of approximately 12 hours. The upper subbase for CA 1-5 was rated as acceptable to marginal (a-m) and had a drainage time of roughly 12 hours. The lower subbase was rated acceptable (a) with a drainage time of approximately 2 hours. The upper and lower subbases for CA 1-7 were both rated acceptable (a), with drainage times of 2 and 3 hours. The upper and lower subbases for CA 1-9 were both rated acceptable (a) and had drainage times of approximately 2 hours each. All of these sections were rated in the FAIR (1.15) to GOOD (1.20) range, with the CTB material reducing the rating to the low end of the GOOD (1.15) scale or the high end of POOR (1.10).

SECTION	C_d
CA 1-1	1.10
CA 1-3	1.00
CA 1-5	1.10
CA 1-7	1.15
CA 1-9	1.10

California 2

Both sections of CA 2 were constructed on half well-drained soils and half modified or altered soils. The NDI of 1 places these soils in the average (j) category. Even though these soils were altered, the underlying soils are well-drained and well-drained soils were the principal source of borrow materials. These sections are in an area that has the potential for the pavement to be saturated approximately 11 percent of the time. CA 2-2 was constructed in a cut area greater than 40-ft (12.2 m), with a 1-ft (0.3 m) ditch line. CA 2-3 was constructed on a fill of greater than 40-ft (12.2 m), with the ditch line at the bottom of the embankment.

CA 2-2 was constructed with an AC interlayer over a permeable CTB over a 3-in (76 mm) aggregate subbase. The aggregate subbase had an acceptable (a) drainage time of less than 1 hour to reach approximately 85 percent saturation level. CA 2-3 was constructed with a CTB over a 6-in (152 mm) aggregate subbase. The aggregate subbase had an acceptable (a) drainage time of approximately 3 hours. CA 2-2 contains edge drains but CA 2-3 does not.

These sections have a small potential for being "bathtub" sections on the basis of their 11 percent saturation time, the well-drained subgrade, and the edge drains in section 2-2, which has an impermeable asphalt layer over the permeable layer of CTB. The aggregate subbases of both sections have excellent drainage times, placing them in the FAIR (0.9) to GOOD (1.10) range. Section 2-2 has better drainage potential than section 2-3 because of the permeable CTB and the edge drains.

SECTION	C_d
CA 2-2	1.10
CA 2-3	0.90

California 3

All three sections of CA 3 were constructed on well-drained subgrade material. The NDI is 1 for these soils, placing them in the average (j) category. All of the sections were constructed with a CTB, an aggregate subbase, and no additional drainage features. These sections are in an area that has a potential for the pavement to be saturated 17 percent of the time. Section 3-1 is built on a 6- to 15-ft (1.8 to 4.6 m) fill, section 3-2 is built in a 16- to 40-ft (4.9 to 12.2 m) cut, and section 3-5 is on a fill of greater than 40 ft (12.2 m).

CA 3-1 has a 5.5-in (140 mm) aggregate subbase with an acceptable (a) drainage time of about 11 hours to reach an 85 percent saturation level. CA 3-5 has a 3.5-in (89 mm) aggregate subbase that has an acceptable (a) drainage time of 9 hours. CA 3-2 has an 18-in (457 mm) aggregate subbase with an acceptable (a) drainage time of less than 1 hour. These sections all rate in the FAIR (1.0) to POOR (0.8) range due to the CTB.

Even though the sections were constructed with impermeable CTB and no additional drainage features, the good drainage times for the subbases and the well drained subgrade should help in draining away excess moisture from beneath the pavement, and thus rate at the upper level of POOR to lower level of FAIR.

SECTION	C_d
CA 3-1	0.90
CA 3-2	1.00
CA 3-5	0.90

California 6

CA 6 was constructed on well-drained to somewhat excessively drained subgrade soils. These soils have an NDI of -1, placing them in the average (j) category. CA 6 has a 4.2-in (107 mm) LCB and a 1.8-in (46 mm) AC subbase placed on 33.0 in (825 mm) of very well-drained borrow material. This section was constructed with edge drains. Half of this section is at grade and the other half is on a fill of greater than 40 ft (12.2 m). This section is in an area that has the potential for the pavement to be saturated approximately 11 percent of the time.

While this section includes an impermeable base course, the well-drained subgrade and the presence of edge drains should help remove excess moisture from the pavement. This section is rated in the POOR (0.9) to FAIR (1.00) range which, because of the positive drainage, moves to lower end of the FAIR scale.

SECTION	C_d
CA 6	1.00

California 7

This section of pavement was constructed on somewhat poorly drained subgrade soils. These soils have an NDI of 4, which is in the marginal to poor (i) category. This section is in an area that has the potential for the pavement to be saturated approximately 11 percent of the time. It was constructed at grade, with a 5-ft (1.5 m) ditch line.

CA 7 was constructed with an impermeable CTB, a 5.4-in (137 mm) lime-treated subbase, and edge drains. The subbase has an unacceptable (u) drainage time of 107 hours to reach the 85 percent saturation level. Taking into account the design of the section and the fact that it was built on a somewhat poorly drained subgrade, this design has the potential to be a "bathtub" section. This section has a quality of drainage in the VERY POOR (0.70) to FAIR (1.00) range. The edge drains in the section should help somewhat in removing excess moisture, producing an average coefficient for this section of the two extremes.

SECTION	C_d
CA 7	0.85

California 8

This section was constructed with a 5.4-in (137 mm) ATB over two aggregate subbases. Edge drains are present. The subgrade of this section is well-drained material that has an NDI of 1, which places it in the average (j) category. CA 8 was built on a fill of 6- to 15-ft (1.5 to 4.6 m) with a 15-ft (4.6 m) ditch line. This section has the potential for pavement saturation 13 percent of the time.

The upper subbase is 2-in (51 mm) thick and has an unacceptable (u) drainage time of approximately 230 hours. The lower subbase is 6-in (152 m) thick and has a marginal (m) drainage time of approximately 34 hours, based on the time it takes the subbase to drain to an 85 percent saturation level. This section is rated in the VERY POOR (0.70) to FAIR (1.00) range. The edge drains and well-drained subgrade should help in removing pavement moisture.

SECTION	C_d
CA 8	0.95

Florida 2

FL 2 was constructed with a sand base placed over a moderately well-drained subgrade. The subgrade has an NDI of 2.50. This gives it a rating in the average (j) category. The section is at grade, with a 5-ft (1.5 m) ditch line. It is in an area that has the potential for the pavement to be saturated approximately 11 percent of the time. No additional drainage features were constructed with this section.

The sand base had an unacceptable (u) drainage time of approximately 571 hours. This gives FL 2 a rating in the VERY POOR (0.70) to POOR (0.90) range. Although the base has poor drainability, the moderately well-drained subgrade may help some in draining excess pavement moisture.

SECTION	C_d
FL 2	0.80

Florida 3

FL 3 was constructed with an impermeable LCB and placed directly on a poorly to very poorly draining subgrade. The subgrade has an NDI of 6.05, or a rating in the poor (i) category. FL 3 was constructed at grade, with a 5-ft (1.5 m) ditch line. This section has the potential for the pavement to be saturated approximately 22 percent of the time.

The design of this pavement and the poorly to very poorly draining subgrade material makes FL 3 a possible "bathtub" section. There is a layer of filter fabric placed in the shoulder which could help with subdrainage. This section is rated in the VERY POOR (0.70) to POOR (0.90) range.

SECTION	C_d
FL 3	0.75

Michigan 1

The Michigan sections are in an area that has the potential for the pavement to be saturated approximately 20 percent of the time. The sections were constructed on a variety of subgrade soils. Mi 1-1a, 1-7a, and 1-7b soils range from very poorly drained to somewhat excessively drained. The NDI of 2.12 puts these soils in the average (j) category. Mi 1-1b, 1-4a, 1-10a, and 1-10b soils are somewhat excessively drained, with an NDI of -2.

This places them in the excellent (k) category. MI 1-25 soils ranged from poorly drained to excessively drained, producing an NDI of 0.88, placing them in the average (j) category.

MI 1-1a and 1-7b were constructed at grade, with a 1-ft (0.3 m) ditch line. MI 1-1b and 1-7a were constructed in a cut of 6 to 15 ft (1.8 to 4.6 m), with a 3- to 4-ft (0.9 to 1.2 m) ditch line. MI 1-10a and 1-10b were constructed on a 16- to 40-ft (4.9 to 12.2 m) fill with a 4- to 5-ft (1.2 to 1.5 m) ditch line. MI 1-25 was constructed on a fill of greater than 40-ft (12.2 m), with a ditch line at the bottom of the fill.

French drains are present in 1-1a, 1-4a, 1-7a, and 1-10a. There are no subdrainage features in the other sections. These sections have the potential to be "bathtub" sections. The French drains, however, will reduce that potential in these sections. The drainability of the subgrade in all of the sections should help to drain moisture away from the pavement.

MI 1-1a, 1-1b, 1-7a, and 1-7b were constructed with a 4- to 6-in (102 to 152 mm) aggregate base and a 10 in (254 mm) sand subbase. Section 1-1a had a base drainage time of approximately 550 hours. Section 1-1b had a base drainage time of approximately 2900 hours. Section 1-7b had a base drainage time of approximately 1300 hours. All of the aggregate bases are rated as unacceptable (u).

MI 1-4a was constructed with a thin PATB. Sections 1-10a, 1-10b, and 1-25 were constructed with a dense-graded ATB. These sections were also constructed with a 10-in sand subbase. Sections 1-1b, 1-4a, 1-7a, 1-7b, and 1-10a had subbase drainage times ranging from approximately 10 to 21 hours. These subbases were all rated in the marginal to acceptable (m-a) range. Sections 1-10b had a subbase drainage time of approximately 40 hours and is rated as marginal (m). Section 1-1a had a subbase drainage time of approximately 107 hours and is rated as unacceptable to marginal (u-m).

Considering all of the above, MI 1-1a, 1-7a, and 1-7b have a quality of drainage in the VERY POOR (0.70 - 0.80) category, but the french drains in sections 1-1a, and 1-7a will improve them one rank, section 1-1a to FAIR, and section 1-7a to the low end of FAIR. MI 1-1b and 1-4a are in the POOR (0.80) to FAIR (1.00) range, with 1-4a moving up to GOOD due to the edge drains. MI 1-25 is in the FAIR (0.90) to POOR (0.80), which must be lowered to VERY POOR (0.80 - 0.70) because of the impermeable ATB, and MI 1-10a and 1-10b rate in the GOOD (1.00) to FAIR (0.90) range, which must be lowered to POOR (0.90 - 0.80) because of the ATB.

SECTION	C_d
MI 1-1a	1.00
MI 1-1b	0.90
MI 1-4a	1.10
MI 1-7a	0.90
MI 1-7b	0.80
MI 1-10a	0.85
MI 1-10b	0.85
MI 1-25	0.70

Michigan 3

MI 3 was constructed on very poorly drained to well-drained subgrade soils. The NDI of 2.4 places them in the average (j) category. This section has a 4.3-in (109 mm) permeable aggregate base with a 3.0-in (76 mm) aggregate subbase. Edge drains are present. The section is at grade, with a 5-ft (1.5 m) ditch line. MI 3 is in an area that has the potential for the pavement to be saturated approximately 22 percent of the time.

The permeable aggregate base had an acceptable (a) drainage time of approximately 2 hours to reach the 85 percent saturation level. The aggregate subbase also had an acceptable (a) drainage time of approximately 5 hours. This section is rated as GOOD (1.00 - 1.10). The good drainability of the base, subbase, and subgrade will make this section rate high in the GOOD (1.10) range. The permeable base, in conjunction with the edge drains, will move this into the low EXCELLENT (1.10 - 1.15) range.

SECTION	C_d
MI 3	1.12

Michigan 4

The cross sections of MI 4-1 and MI 4-2 are the same, with a thin aggregate base over a thick aggregate subbase and no edge drains. Both sections are on a fill of 6- to 15-ft (1.8 to 2.1 m), with a 5-ft (1.5 m) ditch line. These sections are in an area that has the potential for the pavement to be saturated about 22 percent of the time.

MI 4-1 has very poorly drained to well drained subgrade soils with an NDI of 2.44, placing them in the average (j) category. The aggregate base had an unacceptable drainage time of 1245 hours. The aggregate subbase had a marginal drainage time of approximately 30 hours. This section has a quality of drainage in the VERY POOR (0.70) to FAIR (1.00) range. MI 4-2 also has very poorly drained to well-drained subgrade soils, but they have an NDI of 3.38, which is in the marginal (i) category. This section has a marginal to unacceptable (m-u) base with a drainage time of approximately 161 hours. The subbase had a drainage time of approximately 9 hours and was rated marginal (m). This section is rated in the VERY POOR (0.70) to FAIR (1.00) range.

Even though neither section is a typical "bathtub" section, the poor drainability of the bases and subgrade along with no additional drainage features present could contribute to very poor drainability of the sections.

SECTION	C_d
MI 4-1	0.75
MI 4-2	0.70

Michigan 5

MI 5 was constructed with a thin permeable aggregate base over a very thick aggregate subbase. Edge drains are present in this section. MI 5 is at grade and has a 2-ft (0.6 m) ditch line. This section is in an area that has the potential for the pavement to be saturated approximately 33 percent of the time. The subgrade is well-drained material with an NDI of 1, which is an average (j) rating. The base and subbase both have excellent drainage times, less than 3 hours to reach the 85 percent saturation level. This section's quality of drainage rates in the GOOD (1.00) range, with the edge drain moving the drainage to the low EXCELLENT (1.10) range.

Even though the saturation time for the pavement is approximately 33 percent, the edge drains, combined with the acceptable drainage times of the base, subbase, and the subgrade, make this pavement section very drainable.

SECTION	C_d
MI 5	1.05

Minnesota 1

All of the MN 1 sections were constructed on well-drained subgrade materials with an NDI of 1, which is in the average (j) category. The pavement cross sections for all 12 sections were similar. The base types were either aggregate, CTB, or ATB. None of the sections were constructed with a subbase and there were no additional drainage features. These sections are in an area that has the potential for the pavement to be saturated approximately 10 percent of the time.

MN 1-1 was constructed at grade with a 4-ft (1.2 m) ditch line. It has a 6-in (152 mm) aggregate base that has a drainage time of approximately 16 hours. MN 1-2 was constructed on a fill of 6 to 15 ft (1.8 to 4.6 m), with a 4-ft (1.2 m) ditch line. It has a 6-in (152 mm) aggregate base that has a drainage time of approximately 8 hours. MN 1-3 was constructed on a fill of 6 to 15 ft (1.8 to 4.6 m), with a 3-ft (0.9 m) ditch line. It has a 6-in (152 mm) aggregate base with a drainage time of approximately 12 hours.

MN 1-4 was constructed on a fill of 16 to 40 ft (4.9 to 12.2 m), with a 6-ft (1.8 m) ditch line. It has a 6-in (132 mm) aggregate base with a drainage time of approximately 11 hours. All of the sections with aggregate bases were considered acceptable to marginal (a-m) and rated in the FAIR to GOOD range.

MN 1-5 was constructed on a fill of 6 to 15 ft (1.8 to 4.6 m) with a 7-ft (2.1 m) ditch line. MN 1-6 is at grade and has a 4-ft (1.2 m) ditch line. MN 1-7 is in a cut of 6 to 15 ft (1.8 to 4.6 m), with a 4-ft (1.2 m) ditch line. MN 1-8 is at grade and has a 6-ft (1.8 m) ditch line. All of these sections were constructed with an impermeable 5-in (127 mm) ATB and rated high FAIR (0.90), which is lowered to POOR (0.85) because of the dense asphalt base. MN 1-9, MN 1-11, and MN 1-12 were constructed in a cut of 6 to 15 ft (1.8 to 4.6 m) with 3- to 4-ft (0.9 to 1.2 m) ditch lines. MN 1-10 is at grade, with a 4-ft (1.2 m) ditch line. These sections were constructed with an impermeable 5-in (127 mm) CTB and rated at the low end of FAIR (0.90), which is lowered to the low end of POOR (0.80) due to the CTB.

Even with no additional subdrainage features added, the well-drained subgrade and the fact that the aggregate placed in the shoulder construction was "daylighted" should help in draining away excess moisture from the pavement.

SECTION	C_d
MN 1-1	1.05
MN 1-2	1.05
MN 1-3	1.05
MN 1-4	1.05
MN 1-5	0.85
MN 1-6	0.85
MN 1-7	0.85
MN 1-8	0.85
MN 1-9	0.80
MN 1-10	0.80
MN 1-11	0.80
MN 1-12	0.80

Minnesota 2

The MN 2 sections were constructed with a 5- to 6-in (127 to 152 mm) aggregate base, no subbase, and with no additional subdrainage features. MN 2-1 and MN 2-2 were constructed on poorly drained subgrade soils that have an NDI of 5.5, placing them in the poor (i) category. MN 2-3 and MN 2-4 were constructed on well-drained subgrade soils that have an NDI of 1, which places them in the average (j) category. All of the sections are at grade, with ditch lines ranging from 6 to 15 ft (1.8 to 4.6 m). These sections are in an area that has the potential for the pavement to be saturated approximately 19 percent of the time.

MN 2-1 had a base drainage time to the 85 percent saturation level of approximately 12 hours. This rates the base as acceptable to marginal (a-m). This section is in the VERY POOR (0.70) to FAIR (1.00) range. MN 2-2 had a base drainage time of approximately 222 hours. This base material has an unacceptable (u) rating and the quality of drainage of this section is considered as VERY POOR (0.80 - 0.70).

MN 2-3 had a base drainage time of approximately 300 hours and was rated unacceptable (u). This section is rated in the VERY POOR (0.70) to POOR (0.90) range. MN 2-4 had a base drainage time of approximately 26 hours and was rated marginal (m). This section is rated FAIR (1.00 - 0.90).

The aggregate in the shoulders was "daylighted", which will help in removing excess moisture from the pavement. The well-drained subgrade in MN 2-3 and MN 2-4 should also help somewhat with the subdrainage.

SECTION	C_d
MN 2-1	0.85
MN 2-2	0.75
MN 2-3	0.80
MN 2-4	0.95

Minnesota 3

MN 3 was constructed on well-drained subgrade soils that have an NDI of 1, placing them in the average (j) category. The section is at grade, with a 6-ft (1.8 m) ditch line. There were no additional drainage features. This section is in an area that has the potential for the pavement to be saturated approximately 22 percent of the time.

MN 3 has a 4-in (102 mm) aggregate base with a 10-in (254 mm) aggregate subbase. There was no core taken at this section, so the 85 percent saturation level drainabilities were assumed, based on information taken at other sections in that area. The base was estimated to be unacceptable (u) and the subbase was estimated to be marginal (m). Using these assumptions, this section is rated in the VERY POOR (0.70 - 0.80) to FAIR (1.00 - 0.90) range.

SECTION	C_d
MN 3	0.80

Minnesota 4

MN 4 was constructed on poorly drained subgrade materials with an NDI of 5.5. This subgrade is placed in the poor (i) category. It is in a fill of 6 to 15 ft (1.8 to 4.6 m) with a 10-ft (3.0 m) ditch line. This section is in an area that has the potential for the pavement to be saturated approximately 19 percent of the time.

MN 4 has a 5-in (127 mm) base that took approximately 44 hours to drain to an 85 percent saturation level. This rates the base as marginal to unacceptable (m-u). There is no subbase in this section. Edge drains are present. This section is rated in the VERY POOR (0.70) to POOR (0.90) range. The poorly drained subgrade probably will not help in drainage, but the edge drains and "daylighted" aggregate in the shoulders should help in taking away excess moisture from the pavement. The presence of edge drains will improve the drainage to the FAIR range (1.0 - 0.9).

SECTION	C_d
MN 4	0.90

Minnesota 5

MN 5 was constructed with a 3-in (76 mm) aggregate base and a 3-in (76 mm) aggregate subbase placed on well-drained subgrade materials. The subgrade has an NDI of 1, which is the average (j) category. There are no edge drains in this section. MN 5 is at grade and has a 10-ft (3.0 m) ditch line. It is in an area that has the potential for the pavement to be saturated approximately 16 percent of the time.

The base material had a drainage time of approximately 13 hours to reach the 85 percent saturation level. The base is rated as acceptable to marginal (a-m). The subbase had a drainage time of approximately 60 hours, rating it as marginal to unacceptable (m-u). The section is considered to be in the VERY POOR (0.70) to GOOD (1.00) range. The well-drained subgrade and the "daylighted" aggregate in the shoulders should help in draining away excess moisture from the pavement, and produces the higher coefficient for this section.

SECTION	C_d
MN 5	0.95

Minnesota 6

MN 6 was constructed with a 4-in (102 mm) PATB and a 4-in (102 mm) aggregate subbase placed on poorly drained subgrade materials. The subgrade has an NDI of 5.5 which is in the poor (i) category. This section has a subdrainage layer and longitudinal drains. It is at grade, and has a 10-ft (3.0 m) ditch line. MN 6 is in an area that has the potential to be saturated approximately 25 percent of the time. The subbase material had a drainage time of approximately 43 hours to reach the 85 percent saturation level. This rates the subbase material as marginal to unacceptable (m-u).

The poorly drained subgrade may not help much in draining away excess moisture from the pavement. The PATB, edge drains, and the "daylighted" aggregate in the shoulders should help considerably in alleviating any subgrade drainage problems. This section's quality of drainage is in the VERY POOR (0.70) to FAIR (1.00) range, with the presence of the PATB moving it up into the GOOD (1.10) range.

SECTION	C_d
MN 6	1.05

New Jersey 2

NJ 2 was constructed on moderately well-drained to well-drained subgrade soils. The NDI of 2.47 places these soils in the average (j) category. The section has a 12-in (305 mm) aggregate base placed directly on the subgrade, and no additional subdrainage features. It is at grade and has no ditch line. This section is in an area that has the potential for the pavement to be saturated approximately 22 percent of the time.

The aggregate base had a drainage time of approximately 6 hours to reach the 85 percent saturation level. This drainage time is considered acceptable (a). The quality of the base drainage and the subgrade soils should help in draining away excess moisture from the section. This section is rated GOOD (1.10 - 1.00).

SECTION	C_d
NJ 2	1.05

New Jersey 3

The sections for NJ 3 were constructed in an urbanized area. The original subgrade soils have all been disturbed and are unclassified. These sections were built on a fill of 16 to 40 ft (4.9 to 12.2 m). This fill material consists of a combination of slag, steel, and concrete. The top 3 ft (0.9 m) are a well-drained, clean, sandy gravel. The NDI of 1 is in the average (j) category. These sections were constructed with edge drains. The sections are in an area that has the potential for the pavement to be saturated approximately 22 percent of the time.

NJ 3-1 has a thin, permeable aggregate base and a thin aggregate subbase. There was not a large enough sample of the permeable base retrieved to determine drainability to the 85 percent saturation level. Since this base is considered permeable, it was assumed to be acceptable (a). The subbase has a marginal to acceptable (m-a) drainage time of approximately 12 hours. This section is rated in the FAIR (0.90) to GOOD (1.10) range, with the PATB providing a high GOOD (1.10) rating.

NJ 3-2 has a PATB and an acceptable to marginal (a-m) subbase drainage time of approximately 21 hours. This section is also rated in the FAIR to GOOD range, with the presence of the PATB moving it up into the high end of the GOOD (1.10) range.

SECTION	C_d
NJ 3-1	1.10
NJ 3-2	1.10

New York 1

The NY 1 sections are in an area that has the potential for the pavement to be saturated approximately 22 percent of the time. These sections do not have any additional drainage features. Sections 1-1, 1-3, and 1-4 are all on a fill of 16 to 40 ft (4.9 to 12.2 m), with 4- to 8-ft (1.2 to 2.4 m) ditch lines. Section 1-6 is at grade with a 6-ft (1.8 m) ditch line. Sections 1-8a and 1-8b are in a cut of 16 to 40 ft (4.9 to 12.2 m), with 8-ft (4.2 m) ditch lines.

NY 1-1, NY 1-3, NY 1-8a, and NY 1-8b were all constructed with an impermeable 3-in (76 mm) ATB and an 8-in (204 mm) aggregate subbase. NY 1-1 has moderately well-drained to somewhat excessively drained subgrade soils that have an NDI of 1 and are in the average (j) category. The aggregate subbase has an acceptable (a) drainage time of approximately 6 hours to reach the 85 percent saturation level. This section is rated in the FAIR (0.90) to GOOD (1.10) range, with the impermeable base lowering the rating to the high end of the POOR (0.90) range.

NY 1-3, NY 1-8a, and NY 1-8b were constructed on well-drained soils. They have an NDI of 1 and are in the average (j) category. The aggregate subbase for NY 1-3 had an acceptable (a) drainage time of approximately 3 hours. This section is rated in the FAIR to GOOD range, with the impermeable base lowering the rating to the high end of the POOR (0.90) rating. The aggregate subbase for NY 1-8a had a marginal (m) drainage time of approximately 18 hours. This section is also rated in the FAIR to GOOD range, with a final high POOR (0.90) rating. The aggregate subbase for NY 1-8b had an acceptable (a) drainage time of approximately 2 hours. This section is rated at the high end of GOOD, with a final low GOOD rating because of the ATB. NY 1-4 and NY 1-6 were both constructed with 4-in (102 mm) aggregate bases and 8-in (204 mm) aggregate subbases. The aggregate base for NY 1-4 had an unacceptable (u) drainage time of approximately 298 hours. The subbase drainage time of approximately 201 hours was also unacceptable (u). The subgrade soils for this section were well drained and in the average (j) category with an NDI of 1. This section is rated in the VERY POOR (0.70) to POOR (0.90) range.

There was not enough of the retrieved sample to determine the drainage time for NY 1-6. It is assumed to have the same type of aggregate used in NY 1-4, which gives it an unacceptable (u) drainage time. The subbase drainage time was also unacceptable (u), with a drainage time of approximately 208 hours. The subgrade soils for this section were somewhat excessively drained to moderately well-drained and had an NDI of 1. This places these soils in the average (j) category. This section is rated in the VERY POOR to POOR range.

SECTION	C_d
NY 1-1	0.90
NY 1-3	0.90
NY 1-4	0.80
NY 1-6	0.75
NY 1-8a	0.90
NY 1-8b	1.00

New York 2

NY 2-3 was constructed on excessively drained subgrade soils with a NDI of -10. This places these soils in the excellent (k) category. NY 2-9, NY 2-11, and NY 2-15 were all constructed on well-drained subgrade soils with an NDI of 1, placing these soils in the average (j) category. No additional subdrainage features were added to any of the sections. NY 2 is in an area that has the potential for the pavement to be saturated approximately 31 percent of the time.

Section 2-3 is in a cut of 16 to 40 ft (4.9 to 12.2 m), with an 8-ft (2.4 m) ditch line. Section 2-9 is also on a fill of 16 to 40 ft (4.9 to 12.2 m), with a 15-ft (4.6 m) ditch line. Sections 2-11 and 2-15 are at grade, with 10- to 12-ft (3.0 to 3.7 m) ditch lines. NY 2-3 and NY 2-15 were both constructed with aggregate bases and aggregate subbases. Not enough of a base sample was retrieved from NY 2-3, so base drainabilities from NY 2-9 and NY 2-15 were used to assume a marginal to unacceptable (m-u) drainage time for this section. The aggregate subbase also had a marginal to unacceptable (m-u) drainage time of approximately 61 hours. This section is rated in the POOR (0.90) to GOOD (1.10) range.

NY 2-15 had a marginal to unacceptable (m-u) aggregate base drainage time of approximately 75 hours. The subbase had a marginal (m) drainage time of approximately 37 hours. This section is rated in the VERY POOR (0.70) to FAIR (1.00) range. NY 2-9 and NY 2-11 were constructed with aggregate bases and no subbases. NY 2-9 had a marginal to unacceptable (m-u) base drainage time of approximately 73 hours. This section is rated in the VERY POOR to FAIR range.

For section NY 2-11, the aggregate base drainability was also assumed to be marginal to unacceptable (m-u) because of insufficient sample size. This section is in the VERY POOR to FAIR range.

SECTION	C_d
NY 2-3	1.00
NY 2-9	0.85
NY 2-11	0.80
NY 2-15	0.85

North Carolina 1

The eight pavement sections studied in this area of North Carolina were all constructed with no subbases and no additional subdrainage features. They were constructed over a wide variety of natural subgrade soils. The sections are all at grade and have 5-ft (1.5 m) ditch lines. These sections are in an area that has the potential for the pavement to be saturated approximately 22 percent of the time.

NC 1-1 and NC 1-4 were both constructed on poorly drained subgrade soils. They have an NDI of 5.5, which puts them in the poor (i) category. Both sections have thin aggregate bases that have unacceptable to marginal (u-m) drainage times to reach an 85 percent pavement saturation level. NC 1-1 was approximately 189 hours and NC 1-4 was approximately 169 hours. These sections were both rated in the VERY POOR (0.70) to POOR (0.80) range. NC 1-2 and NC 1-3 have thick, soil-cement bases. NC 1-2 was constructed on poorly drained to moderately well-drained subgrade soils. These soils have an NDI of 4.24, putting them in the poor (i) category. NC 1-3 was constructed on well-drained to poorly drained subgrade soils. These soils have an NDI of 2.12, putting them in the average (j) category. The quality of drainage rating for NC 1-2, and 1-3 is FAIR, with the CTB lowering that value to POOR (0.80-0.90).

NC 1-5 has an impermeable CTB and was constructed on poorly drained to moderately well-drained subgrade soils. These soils have an NDI of 5.05, which is in the poor (i) category. This section rated in the POOR to FAIR, with the CTB lowering it to a VERY POOR (0.75) range. NC 1-6 has an impermeable ATB and was constructed on well-drained to moderately well-drained subgrade soils. These soils have an NDI of 1.26, which puts them in the average (j) category. This section has a FAIR (1.00) rating, which the ATB lowers to POOR (0.90).

NC 1-7 and NC 1-8 both have thin aggregate bases with unacceptable to marginal (u-m) drainage times to reach an 85 percent saturation level. The drainage time for NC 1-7 was approximately 109 hours and approximately 96 hours for NC 1-8. NC 1-7 was constructed on well-drained subgrade soils with an NDI of 1, which is in the average (j) category. This well-drained subgrade should help with subdrainage. NC 1-8 was constructed on well-drained to poorly drained subgrade soils with an NDI of 2.18, which is in the average (j) category. Both sections rate in the VERY POOR (0.70) to FAIR (1.00) range, with section 1-8 being slightly worse for drainage.

SECTION	C_d
NC 1-1	0.75
NC 1-2	0.80
NC 1-3	0.98
NC 1-4	0.75
NC 1-5	0.75
NC 1-6	0.90
NC 1-7	0.90
NC 1-8	0.80

North Carolina 2

NC 2 was constructed with an impermeable LCB, no subbase, and retrofit edge drains. This section is on well-drained subgrade soils which have an NDI of 1, putting it in the average (j) category. It is in an area that has the potential for the pavement to be saturated about 20 percent of the time. Because of the cross section design, this section has the potential for being a "bathtub" section. With the well-drained subgrade and the edge drains, this section is rated in the FAIR (1.00 - 0.90).

SECTION	C_d
NC 2	0.95

Ohio 1

Ohio 1 had seven pavement sections that were all constructed over well-drained subgrade soils with an NDI of 1. The sections were constructed on a fill of greater than 40 ft (12.2 m), with ditch lines at the bottom of the fill. These sections are in an area that has a potential for the pavement to be saturated approximately 19 percent of the time. None of these sections was constructed with a subbase or any other subdrainage features.

OH 1-1 has a thick aggregate base with a drainage time to an 85 percent saturation level of approximately 27 hours. This gives the base a rating of marginal to acceptable (m-a). This section is in the FAIR (0.90) to GOOD (1.10) range, with a slight reduction because the poor base is thick. OH 1-10 is also in the FAIR (0.90) to GOOD (1.10) range. It has a thick aggregate base similar to section 1-1, but it has a drainage time of approximately 12 hours and a rating of acceptable to marginal (a-m), giving a rating of GOOD (1.10 - 1.00). OH 1-9 rates in the POOR (0.80) to FAIR (1.00) range. It has a thick aggregate base with a drainage time of approximately 60 hours, which places it in the category of unacceptable to marginal (u-m).

OH 1-6 rates is in the VERY POOR (0.80) to POOR (0.90) range. It has a thick aggregate base with a drainage time of approximately 175 hours which makes it unacceptable (u). OH 1-7 has a GOOD (1.00 - 1.10) rating. It has a thick aggregate base with a drainage time of less than 1 hour, which rates it as excellent (a). OH 1-3 and OH 1-4 were both constructed with impermeable ATB's. The quality of drainage of both these sections was FAIR (1.00 - 0.90) with the impermeable ATB lowering this to a rating of POOR (0.90).

Even though no additional subdrainage features were added to these sections, the well-drained soils on which they were constructed should help in keeping moisture drained from the pavement.

SECTION	C_d
OH 1-1	0.95
OH 1-3	0.90
OH 1-4	0.90
OH 1-6	0.85
OH 1-7	1.05
OH 1-9	0.90
OH 1-10	1.05

Ohio 2

Both of the sections for OH 2 were constructed by placing the pavement directly on the subgrade. No additional drainage features were added to either of the sections. These sections are in an area that has a potential for the pavement to be saturated approximately 14 percent of the time.

OH 2-33a was constructed on a somewhat poorly drained subgrade. This subgrade has an NDI of 4, which is in the poor (i) category. This section has a potential to be a "bathtub" section because of the cross section design and the somewhat poorly drained subgrade. Taking into consideration all of the above mentioned factors, this section is rated as a good POOR (0.90). Oh 2-33b was constructed on a somewhat poorly to moderately well-drained subgrade. This subgrade has an NDI of 3.04 and is in the marginal (i) category. Because this subgrade is going to be somewhat better drained than OH 2-33a, it does not have as much potential for being a "bathtub" section. Again, taking into account all of the above mentioned factors, this section is rated in the POOR (0.90) to FAIR (1.00) range.

SECTION	C_d
OH 2-33a	0.90
OH 2-33b	0.95

Ontario 1

In Ruthven, Ontario there were a total of four different pavement sections. These sections are all at grade with 3- to 8-ft (0.9 to 4.2) ditch lines and edge drains. The ONT 1 sections are in an area that has the potential for the pavement to be saturated approximately 28 percent of the time.

ONT 1-1 was constructed as a slab on grade. The subgrade soils for this section range from poorly drained to well-drained. The NDI of 4.08 places these soils in the poor (i) category. Since the poor draining subgrade probably will not help with drainage, this section is a possible "bathtub" section and is rated as VERY POOR (0.80) to POOR (0.90). Edge drains in the section should help with draining away excess moisture. and improve the rating to FAIR (0.95).

ONT 1-2, ONT 1-3, and ONT 1-4 were all constructed on moderately well-drained subgrade soils. These soils have an NDI of 2.5, which is in the average (j) category. ONT 1-2 has a PATB with no subbase. This section is rated FAIR (1.00 - 0.90), and since this section has a permeable base, edge drains, and a moderately well-drained subgrade, its rating can be increased to GOOD (1.00). ONT 1-3 and ONT 1-4 both have impermeable LCB's and no subbase. These sections could possibly be bathtub sections. Since there are edge drains and a moderately well-drained subgrade, this potential is somewhat lessened. These sections are rated FAIR (0.95), which is decreased to POOR (0.80) for the LCB, but increased to FAIR (1.00) because of the edge drains and subgrade.

SECTION	C_d
ONT 1-1	0.95
ONT 1-2	1.00
ONT 1-3	1.00
ONT 1-4	1.00

Ontario 2

This section was constructed with a 6-in (152 mm) impermeable CTB placed directly on the subgrade. The original subgrade soil for this area have all been disturbed due to urbanization, but the fill material used is poorly drained. These soils have an NDI of 5.5, placing them in the poor (i) category. These sections are in an area that has the potential for the pavement to be saturated approximately 28 percent of the time. This section is rated in the very poor-fair range.

SECTION	C_d
ONT 2	0.80

Pennsylvania 1

In Pennsylvania, there were five different cross sections constructed. All five sections were built with edge drains. These are all in an area that has the potential for the pavement to be saturated approximately 31 percent of the time.

Sections 1-1 and 1-4 were constructed over well-drained subgrade soils. These soils have an NDI of 1, which puts them in the average (j) category. These sections range from being in a cut of greater than 40 ft (12.2 m) to a fill of greater than 40 ft (12.2 m). The depth to the ditch lines is approximately 5 ft (1.5 m). PA 1-1 was constructed with a thick, impermeable CTB and a thick aggregate subbase. The subbase was rated unacceptable (u) because of its drainage time. This time was calculated to be approximately 290 hours. This section is in the POOR (0.80) to FAIR (0.90) range, with the CTB lowering the rating to POOR (0.80), but the edge drain raising it to FAIR (0.90). PA 1-4 was constructed with a thick, permeable aggregate base over an aggregate subbase. The base was rated excellent (a) with a drainage time of less than 1 hour. The subbase was rated as unacceptable (u) with a drainage time of approximately 130 hours. These factors give a drainage quality ranging from POOR (0.80) to GOOD (1.00), with the permeable base providing the GOOD (1.00) to EXCELLENT (1.10) rating of 1.05.

Sections 1-2 and 1-5 were constructed over moderately well-drained subgrade soils. These soils have an NDI of 2.5, which is in the average (j) category. These sections are in cuts from 16 ft (4.9 m) to greater than 40 ft (12.2 m), with 5-ft (1.5 m) ditch lines. PA 1-2 was constructed with a PATB over a thick aggregate subbase. The subbase was rated excellent (a) because of its drainage time of less than 1 hour. The permeable base and the good drainage time of the subbase give this section a rating of GOOD (1.00) to EXCELLENT (1.10). PA 1-5 was constructed with a very thick aggregate base placed directly on the subgrade. The drainage time of approximately 60 hours was rated marginal to unacceptable (m-u). These factors put this section in the POOR (0.80) to FAIR (0.90) range, with the edge drain raising the rating to GOOD (1.00). PA 1-3 was constructed over a well-drained to moderately well-drained subgrade with an NDI of 2.24 in the average (j) category. This section is in a cut of more than 40 ft (12.2 m) and has a 4-ft (1.2 m) ditch line. Section 1-3 has a permeable aggregate base over an aggregate subbase. Both the base and subbase have drainage times of less than 1 hour. These factors give this section a rating of GOOD (1.00) to EXCELLENT (1.10).

In these sections, the addition of edge drains and the well- to moderately well-drained subgrade soils will help considerably in draining off moisture from the pavement.

SECTION	C_d
PA 1-1	0.90
PA 1-2	1.05
PA 1-3	1.05
PA 1-4	1.00
PA 1-5	1.00

CHAPTER 5 BACKCALCULATION METHODOLOGY

1. INTRODUCTION

This chapter describes the methodology involved in the backcalculation of modulus values for the projects included in the study. Backcalculation procedures were used on the deflection data obtained during the field testing. The deflection testing was performed with a Falling Weight Deflectometer (FWD) on all but three sections: Ontario 1, Ontario 2, and Ohio 2. Deflection testing was not performed on these three projects due to logistical and financial reasons.

To determine the in-situ portland cement concrete (PCC) and foundation support moduli, an interpretation scheme for FWD deflection data based on a recently developed closed-form backcalculation procedure was used.(20) It is based on a theoretically rigorous approach utilizing the principles of dimensional analysis as well as the concept of the deflection basin area, first proposed by Hoffman and Thompson.(21) The backcalculation method has been automated by Ioannides through a computer program entitled ILLI-BACK.(20) The computer program and the theoretical methodology upon which it is based is used to determine in-situ strength parameters for pavement systems. This program/methodology greatly simplifies the effort required in interpreting nondestructive testing (NDT) data. In addition to yielding the required backcalculated parameters, this approach also provides an evaluation of the degree to which the behavior of the in-situ pavement system follows theory. The approach models the pavement system as an elastic medium-thick plate resting on a dense liquid (DL) or an elastic solid (ES) foundation. For the purposes of this study, the dense liquid subgrade model was used for backcalculation analyses.

2. FUNDAMENTAL CONCEPTS

The backcalculation scheme used in this study employs two fundamental and theoretical concepts.

1. A unique relationship exists between the deflection basin area, AREA, and the radius of relative stiffness, l, of the pavement-subgrade system.(20)

2. Deflections in rigid pavements, expressed in a dimensionless form, are solely a function of the load size ratio, (a/l), where a is the radius of the applied load.(22)

All three of these quantities (AREA, l, and a) are expressed in inches. The area of the deflection basin is calculated as follows:

$$AREA = 6[1 + 2(D_1/D_0) + 2(D_2/D_0) + (D_3/D_0)] \quad (3)$$

where D_0, D_1, D_2 and D_3 are the FWD deflections recorded at 0, 12, 24, and 36 in (0, 305, 610, and 915 mm) from the center of the loading plate, respectively.

The definition of the radius of relative stiffness of the pavement-subgrade system is a function of the type of foundation assumed. For a dense liquid (DL) foundation:

$$l = l_k = [Eh^3/\{12(1 - u^2)k\}]^{1/4} \qquad (4)$$

For an elastic solid (ES) foundation:

$$l = l_e = [Eh^3(1 - u_s)/\{6(1 - u)E_s\}]^{1/3} \qquad (5)$$

Where:

E = slab Young's modulus
E_s = soil Young's modulus
h = slab thickness
u = concrete Poisson's Ratio
u_s = soil Poisson's Ratio
k = modulus of subgrade reaction

Application of dimensional analysis indicates that a unique relationship between AREA and l exists and is valid for any chosen plate size and sensor arrangement. Figure 13 shows the AREA vs. l curves for four different loading and support conditions. In this figure, four sensors at 12 in (305 mm) spacing are employed.

Inspection of the interior loading formula presented by Westergaard shows that the maximum deflection in a two-layer, rigid pavement system may be rewritten in the following nondimensional form:

$$d_0 = D_0D/(Pl^2) = D_0kl^2/P \qquad (6)$$

where d_0 is the nondimensional sensor deflection corresponding to the measured deflection, D_0, under the applied load, P.(23) The slab flexural stiffness, D, is given by:

$$D = Eh^3/\{12(1 - u^2)\} \qquad (7)$$

Similar expressions can be derived for the other three FWD deflections, as shown below:

$$d_i = D_iD/(Pl^2) = D_ikl^2/P \qquad (i=0,3) \qquad (8)$$

In this case, d_i denotes the four nondimensional sensor deflections corresponding to the measured deflections, D_i. The nondimensional deflections are known functions of the ratio (a/l) only. In the case of a constant plate load radius, they are uniquely defined by the l-value alone. Figures 14 and 15 show the variation with l of dimensionless deflections, d_i, for a circular load with a radius a = 5.9055 in (150 mm). The curves corresponding to d_0 are defined by the interior loading maximum deflection formulae presented by Westergaard for the DL foundation and by Losberg for the ES foundation.(23,24,25) The remainder of the curves in figures 14 and 15 were derived more recently by Ioannides.(20)

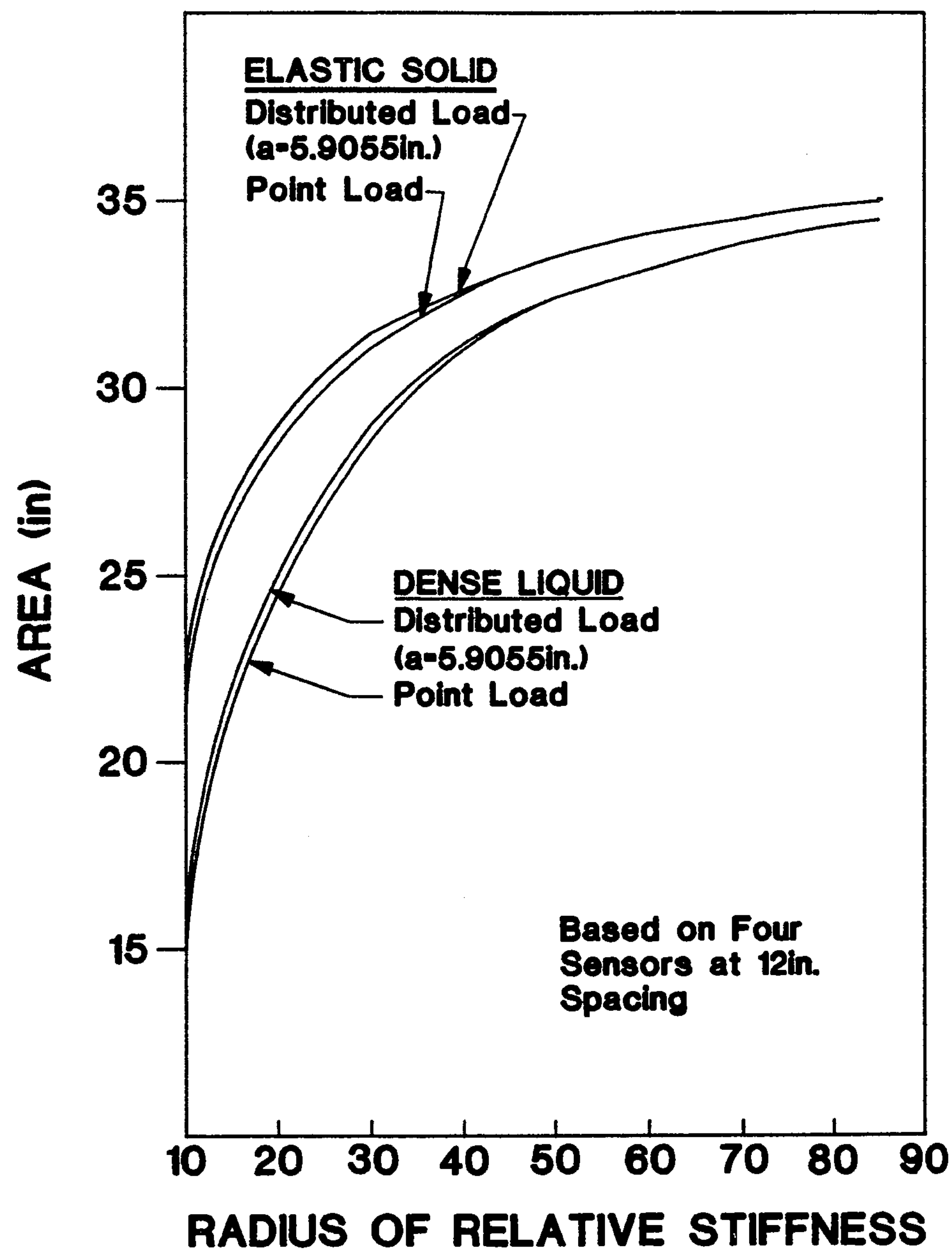


Figure 13. Variation in deflection basin AREA with radius of relative stiffness.

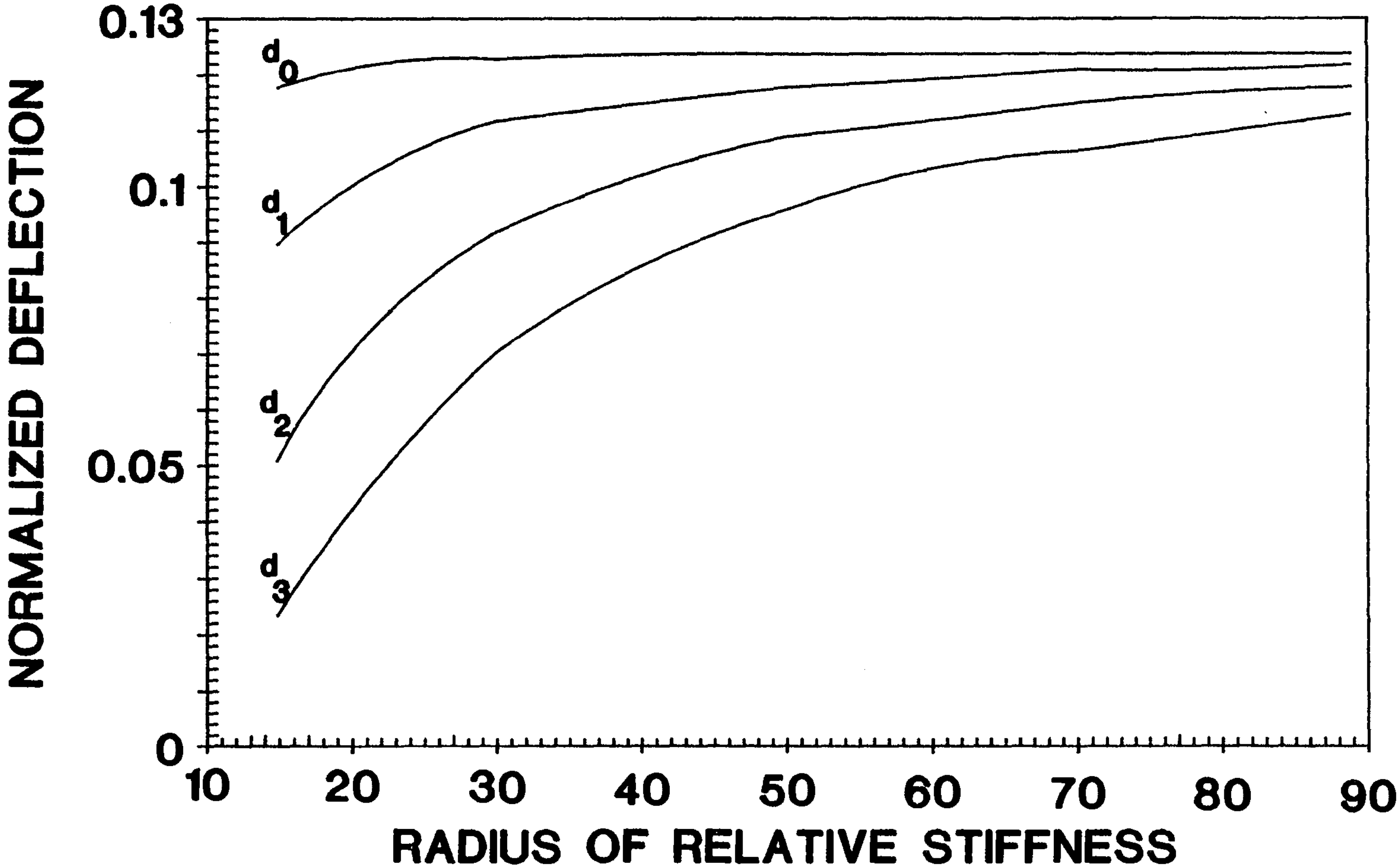


Figure 14. Variation in dimensionless deflection with radius of relative stiffness for the dense liquid foundation.

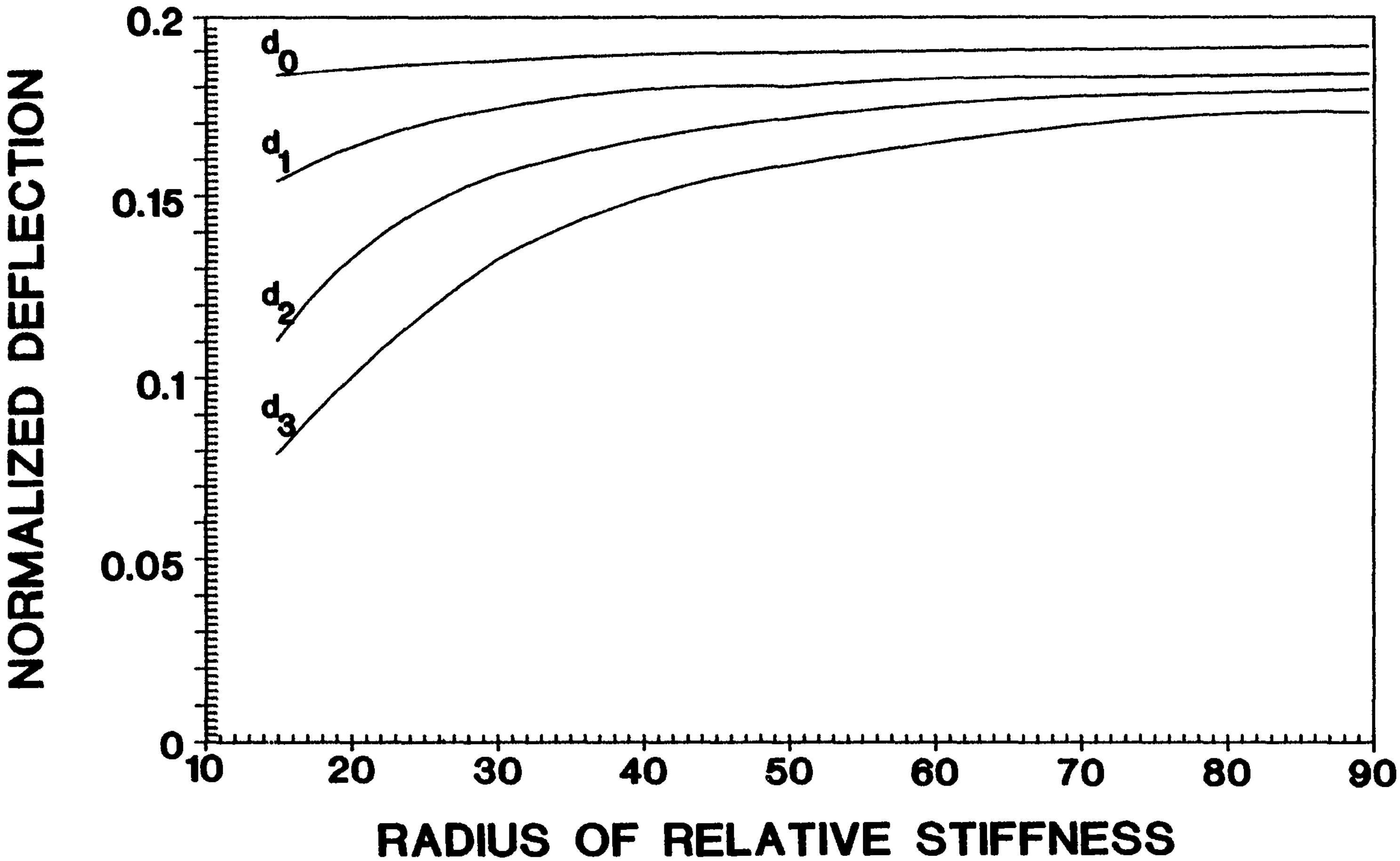


Figure 15. Variation in dimensionless deflections with radius of relative stiffness for the elastic solid foundation.

3. OUTLINE OF BACKCALCULATION PROCEDURE

On the basis of the two fundamental concepts discussed above, the backcalculation procedure proceeds as follows:

1. Drop the weight, and record the applied load, P, as well as the resulting deflections, D_0, D_1, D_2 and D_3.

2. Calculate the area of the deflection basin, AREA.

3. Enter figure 13 with the AREA-value and select the corresponding radius of relative stiffness value, l.

4. Enter figure 14 or figure 15 with this l-value and determine the corresponding dimensionless deflections, d_i.

5. Backcalculate the subgrade support values as follows:

   For the DL foundation:

   $$k = (d_i/D_i)*(P/l^2) \tag{9}$$

   For the ES foundation:

   $$C = E_s/(1 - u_s) = (d_i/D_i)*(2P/l) \tag{10}$$

   For a given value of the subgrade Poisson's Ratio (generally, u_s is between 0.4 and 0.5), equation 10 can be rewritten to yield the foundation modulus, E_s:

   $$E_s = (1 - u_s)*(d_i/D_i)*(2P/l) \tag{11}$$

6. Backcalculate the slab flexural stiffness, as follows:

   $$D = Eh^3/\{12(1 - u^2)\} = (d_i/D_i)Pl^2 \tag{12}$$

If the slab thickness, h, is known, the slab modulus can be calculated using:

$$E = \{12(1 - u^2)/h^3\}*(d_i/D_i)Pl^2 \tag{13}$$

Alternatively, if the slab modulus, E, is known, the thickness, h, can be backcalculated from:

$$h = [\{12(1 - u^2)/E\}*(d_i/D_i)Pl^2]^{1/3} \tag{14}$$

A value of 0.15 is generally assumed for the Poisson's ratio of the concrete slab.

These backcalculation equations (equations 9 through 14) provide four determinations of each pavement system parameter (k, E_s, h or E), each corresponding to one measured deflection, D_i. This provides a control on the accuracy of individual sensor readings as well as a measure of in-situ material variability and of the departure of the real system from the idealized conditions assumed in theory.

The results of the brief statistical analysis of the parameters backcalculated above provide some useful insights pertaining to the testing procedure adopted, and to the observed behavior of the in-situ pavement system. In all acceptable cases, a coefficient of variation smaller than 10 percent was obtained. Those tests showing a coefficient of variation greater than 10 percent were discarded for the following reasons. If one of the sensors had malfunctioned, the backcalculated parameters based on that particular deflection reading would have been significantly different from the backcalculated values obtained from the other deflection readings. This would alert the engineer to the possibility of an error. In addition, figures 14 and 15 indicate that d_0 and d_1 are relatively insensitive to changes in the value of l, compared to d_2 and d_3. Thus, it may be concluded that the latter are more reliable backcalculation tools. On the other hand, the actual measured deflections D_0 and D_1 are probably more accurate than D_2 and D_3, owing to their larger magnitude and the equal sensitivity of all sensors, as used in conventional practice. Therefore, it may be desirable to use sensors of increased sensitivity for measuring deflections further away from the center of the applied load. This will become increasingly important also as States move toward constructing thicker concrete slabs.

4. APPLICATION OF CLOSED-FORM BACKCALCULATION PROCEDURE

To illustrate the application of the procedure described above, a set of FWD data collected from a PCC pavement section on U.S. 23 near Chillicothe, Ohio will be used to backcalculate the pavement parameters. The radius of the load plate, a, was the standard 5.9055 in (150 mm) and the recorded load, P, was 12,569 lb (55.9 kN). Sensors were located at 0, 12, 24, and 36 in (0, 305, 610, and 915 mm) from the center of the plate. The recorded deflections were as follows:

$$D_0 = 0.0047\ (0.119\ \text{mm});\ D_1 = 0.0040\ \text{in}\ (0.102\ \text{mm});$$
$$D_2 = 0.0035\ (0.089\ \text{mm});\ D_3 = 0.0026\ \text{in}\ (0.0660\ \text{mm}).$$

Substituting these deflections in equation 3 yields:

$$\begin{aligned} AREA &= 6[1 + 2(4.0/4.7) + 2(3.5/4.7) + (2.6/9.7)] \\ &= 28.64\ \text{in}\ (727.5\ \text{mm}). \end{aligned}$$

From figure 13, the corresponding radius of relative stiffness value is:

$$\text{DL: } l_k = 27.7\ \text{in}\ (703.6\ \text{mm}).$$

Entering figure 14 with the l-value, the following nondimensional deflections are obtained:

$$\text{DL Foundation: } d_0 = 0.122;\ d_1 = 0.108;$$
$$d_2 = 0.087;\ d_3 = 0.061.$$

Thus, equations 9 and 10 give the following backcalculated subgrade support values:

For the DL Foundation:

$k = (0.122/0.0047)(12569/27.7^2)$
$= 425$ psi/in (115 kPa/mm), based on D_0;

$k = (0.108/0.0040)(12569/27.7^2)$
$= 442$ psi/in (120 kPa/mm), based on D_1;

$k = (0.087/0.0035)(12569/27.7^2)$
$= 407$ psi/in (110 kPa/mm), based on D_2;

$k = (0.061/0.0026)(12569/27.7^2)$
$= 384$ psi/in (104 kPa/mm), based on D_3.

The mean of these backcalculated k-values is 415 psi/in (113 kPa/mm) and their coefficient of variation (COV) is 5.99 percent.

The thickness is known to be 9.0 in (229 mm) from the plans. Equation 13 is used with a Poisson's Ratio of 0.15 to backcalculate the slab modulus. Thus, for the DL foundation:

$$E = \{12(1-0.15^2)/9.0^3\}*(0.122/0.0047)*12969(27.7)^2$$
$$= 4{,}028{,}019 \text{ psi } (27{,}770 \text{ MPa}), \text{ based on } D_0$$

The other backcalculated E-values are: E = 4,189,802 psi (28,890 MPa) based on D_1; E = 3,857,279 psi (26,600 MPa) based on D_2; and E = 3,644,712 psi (25,130 MPa) based on D_3. Hence, an average value of E = 3,928,953 psi (27,090 MPa) is determined, with a COV = 5.99 percent (identical to that for the k-value).

5. VARIABILITY AND RELIABILITY OF TESTING AND BACKCALCULATED VALUES

Since the closed-form backcalculation procedure described above is a relatively new approach, it is desirable to examine the validity of the results obtained from the procedure. It has been suggested that some of the backcalculated values are suspect. Specifically, the following criticisms have been presented:

1. The backcalculated modulus of elasticity (E) of the surface appears too high for the Rothsay, Minnesota sections. Another group of researchers reported E's at this site in the range of 4 to 5 million psi (27,580 to 34,470 MPa).(26)

2. An inverse relationship appears to exist between the E of the portland cement concrete surface and the effective modulus of subgrade reaction (k-value) on top of the base.

The analysis presented herein shows that the procedure is theoretically valid and that the criticisms presented above are unfounded.

1. The backcalculated E value of the surface appears too high for the Rothsay sections.

A trial and error method using a finite element program was employed by the researchers to "backcalculate" the E of the surface and k of the supporting layers.(26) The researchers tried to match the deflection under the load only. They did so by altering the E and k-values until the deflection under the load was matched. However, there are thousands of combinations of E and k-values that will produce a single deflection beneath a load. The final E and k-values were chosen based on the researcher's judgment.

Two major problems can be readily identified with this trial and error approach. First, a range of E and k-values must be chosen for the analysis. Typically, the E of the surface is fixed and the k-value is varied. Secondly, in matching the deflection beneath the load only, the deflection basin is ignored. The deflection basin shape is extremely important in the determination in the E and k-values. The trial and error method of backcalculation has largely been supplanted by current backcalculation procedures.

Plate Theory

According to basic plate theory, the E of the surface layer by itself is not an adequate description of the plate (slab) stiffness. As presented by Timoshenko, the flexural stiffness of a medium thick plate is defined as: (27)

$$D = [Eh^3/12(1 - u^2)] \qquad (15)$$

Where:

E = Modulus of Elasticity of the Material
h = Thickness of the Plate
u = Poisson's Ratio of the Material

Thus, the thickness of the plate, modulus of elasticity of the plate material, and Poisson's Ratio of the material are controlling factors in quantifying the slab's flexural stiffness. Since Poisson's Ratio is typically assumed to be 0.15 for portland cement concrete, the stiffness of the plate reduces to be a function of Eh^3 only. Therefore, the E value alone is of limited engineering significance. Examine the following situation:

	Thin/Strong Concrete Plate	Thick/Weak Concrete Plate
E	10.0×10^6 psi	1.0×10^6 psi
h	2.0 in	10.0 in
Eh^3	8.0×10^7 lb-in	1.0×10^8 lb-in

Even though the thinner slab has an E that is an order of magnitude higher than the thicker slab, the thicker slab has a higher flexural stiffness, for a given Poisson's Ratio. Therefore, looking at the E alone is misleading. Table 5 shows the Eh^3 values for all of the Phase I sections. For all sections, the Eh^3 values range from 2.25×10^9 lb-in (2.54×10^8 N-m) to 1.22×10^{10} lb-in (1.38×10^9 N-m). For the Rothsay sections only, the Eh^3 values range from 3.40×10^9 lb-in (3.84×10^8 N-m) to 6.05×10^9 lb-in (6.84×10^8 N-m), which shows that in terms of flexural stiffness, the Rothsay sections fall in the midrange of the data, even though the E-values seem very high.

Table 5. Eh^3 values for all Phase I sections.

ID	k-value	E_{pcc}	h	Eh^3
NY 1-1	549	3810000	9.0	2777490000
NY 1-3	503	3890000	9.0	2835810000
NY 1-4	534	4760000	9.0	3470040000
NY 1-6	619	4020000	9.0	2930580000
NY 1-8a	638	4100000	9.0	2988900000
NY 1-8b	548	3880000	9.0	2828520000
NY 2-3	341	5100000	9.0	3717900000
NY 2-9	471	5090000	9.0	3710610000
NY 2-11	296	6120000	9.0	4461480000
NY 2-15	273	5520000	9.0	4024080000
OH 1-1	449	4430000	9.0	3229470000
OH 1-3	440	5310000	9.0	3870990000
OH 1-4	525	5230000	9.0	3812670000
OH 1-6	431	4060000	9.0	2959740000
OH 1-7	340	4430000	9.0	3229470000
OH 1-9	351	3400000	9.0	2478600000
OH 1-10	405	4510000	9.0	3287790000
PA 1-1	731	4210000	10.0	4210000000
PA 1-2	1040	3390000	10.0	3390000000
PA 1-3	538	3230000	10.0	3230000000
PA 1-4	747	4530000	10.0	4530000000
PA 1-5	540	3620000	10.0	3620000000
NJ 2	234	6720000	9.0	4898880000
NJ 3-1	356	5400000	9.0	3936600000
NJ 3-2	210	5330000	9.0	3885570000
CA 1-1	232	6610000	8.4	3917773440
CA 1-3	349	5240000	8.4	3105768960
CA 1-5	335	5280000	11.4	7822552320
CA 1-7	433	6480000	8.4	3840721920
CA 1-9	298	6950000	8.4	4119292800
CA 2-2	1400	7040000	8.4	4172636160
CA 2-3	528	4980000	8.4	2951665920
CA 7	326	6260000	10.2	6643162080
CA 6	294	6710000	9.0	4891590000
CA 8	339	6420000	10.2	6812955360
CA 3-1	286	3530000	9.0	2573370000
CA 3-2	312	4170000	9.0	3039930000
CA 3-5	397	4380000	9.0	3193020000
FL 2	378	5550000	13.0	12193350000
FL 3	529	4160000	9.0	3032640000

Table 5. Eh^3 values for all Phase I sections (cont'd).

ID	k-value	E_{pcc}	h	Eh^3
MN 1-1	191	7090000	9.0	5168610000
MN 1-2	172	7750000	9.0	5649750000
MN 1-3	217	6660000	8.0	3409920000
MN 1-4	222	6920000	8.0	3543040000
MN 1-5	304	9130000	8.0	4674560000
MN 1-6	314	9360000	8.0	4792320000
MN 1-7	287	8300000	9.0	6050700000
MN 1-8	278	7880000	9.0	5744520000
MN 1-9	291	6670000	9.0	4862430000
MN 1-10	285	6740000	9.0	4913460000
MN 1-11	245	8030000	8.0	4111360000
MN 1-12	239	7790000	8.0	3988480000
MN 5	156	7560000	9.0	5511240000
MN 3	256	8810000	9.0	6422490000
MN 4	222	6300000	7.5	2657812500
MN 6	199	6570000	8.0	3363840000
MN 2-1	128	6780000	9.0	4942620000
MN 2-2	127	8010000	8.0	4101120000
MN 2-3	162	7320000	9.0	5336280000
MN 2-4	178	6620000	9.0	4825980000
NC 1-1	538	4630000	9.0	3375270000
NC 1-2	347	5190000	9.0	3783510000
NC 1-3	494	3970000	9.0	2894130000
NC 1-4	570	4210000	9.0	3069090000
NC 1-5	628	5500000	9.0	4009500000
NC 1-6	672	5140000	9.0	3747060000
NC 1-7	128	5090000	8.0	2606080000
NC 1-8	513	4220000	9.0	3076380000
NC 2	293	5890000	11.0	7839590000
AZ 1-1	546	3140000	9.0	2289060000
AZ 1-2	492	3440000	13.0	7557680000
AZ 1-4	344	3490000	13.0	7667530000
AZ 1-5	439	3290000	11.0	4378990000
AZ 1-6	621	3090000	9.0	2252610000
AZ 1-7	584	3690000	9.0	2690010000
AZ 2	174	5560000	10.0	5560000000
MI 1-1a	353	5450000	9.0	3973050000
MI 1-1b	300	5790000	9.0	4220910000
MI 1-4a	468	5880000	9.0	4286520000
MI 1-7a	292	6340000	9.0	4621860000
MI 1-7b	269	6090000	9.0	4439610000
MI 1-10a	436	6230000	9.0	4541670000
MI 1-10b	502	5280000	9.0	3849120000
MI 3	283	4830000	9.0	3521070000
MI 5	189	4530000	9.0	3302370000
MI 4-1	186	4380000	9.0	3193020000
MI 4-2	233	4490000	9.0	3273210000

Deflection Measurement

In examining table 5 more closely, an interesting trend is observed. The backcalculated E values for the Minnesota sections (as a group) are consistently higher than all of the other sections. Because all of the Minnesota data was collected with Minnesota's Falling Weight Deflectometer (FWD) and all of the other data was taken with a different FWD (with the exception of Arizona data), a problem may have occurred with the equipment or data collection. There are two possibilities for the discrepancy: incorrect calibration of the FWD sensors or inaccurate spacing of the sensors.

The FWD sensors may have been calibrated incorrectly. The manufacturer's recommend calibration of the sensors before any large testing job. The calibration method calls for adjustment of the sensors relative to the other sensor readings. It is possible to calibrate the sensors such that all or some of the sensors record higher or lower deflection readings.

Because of the nature of the backcalculation procedure (E- and k-values are calculated for each sensor and averaged for each particular drop of the weight), the inaccuracy of one or two sensors would be flagged. However, if all of the sensors were measuring high or low, the procedure would not be able to detect the inaccuracy. Therefore, all of the sensors would have to be calibrated to read lower deflections (potentially yielding higher E values for a given thickness). In order to examine the effect of inaccurate calibration, <u>each</u> sensor reading was increased by 20 percent. The results showed that increasing all of the deflection readings by 20 percent (thereby not changing the shape of the deflection basin) had little effect on E and k, as shown below in table 6.

Table 6. Effect of deflection error on backcalculated E and k.

MN 1-5	Results Without Using Increased Deflections	Results Using Increased Deflections
E	9.13×10^6 psi	8.91×10^6 psi
k	304 pci	256 pci

Given this analysis, it seems that the inaccurate calibration of the FWD sensors is not a possible explanation of the high E values for the Minnesota sections.

The second possibility is that the sensors were not fixed at the proper 12-in (305 mm) spacing. If the sensors were placed more than 12 in (305 mm) apart, then the actual deflection basin would be distorted. Since the deflections decrease while moving away from the load, the recorded deflections (at spacings greater than 12-in [305 mm]) would be lower than the actual deflections at 12-in (305 mm) spacings. The spacing of the sensors has a tremendous effect on the backcalculation of E and k values.

Although this is a possible problem with the data, the Minnesota DOT was informed of the desired sensor configuration before testing. It is probable that the sensor spacing was measured and checked before testing.

Finite Element Analysis

The final analysis involves the examination of deflection basins calculated using the ILLISLAB finite element program. Figures 16 through 18 illustrate deflection basins as measured by the FWD and as calculated by ILLISLAB. The design parameters (thickness, joint spacing, load, etc.) and the backcalculated E- and k-values (for a given location) were input into the finite element program. The deflections were then calculated under the load and every 12 in (305 mm) away from the load. These calculated deflections were then plotted with the actual deflection as measured by the FWD to examine the accuracy of the deflection basin. The figures show that when the actual backcalculated values are input into ILLISLAB, the resulting deflection basins accurately reflect the measured deflection basins.

The analysis of the deflection basins using ILLISLAB in combination with the analysis of the flexural strength, D, conclusively shows that the backcalculation procedure accurately reflects the field situation as well as being soundly based in accepted theory.

2. **An inverse relationship appears to exist between the modulus of elasticity (E) of the portland cement concrete surface and the effective modulus of subgrade reaction (k-value) on top of the base.**

Although it may generally be argued that the k-value describes the foundation support, it is <u>not</u> an intrinsic soil property. The k-value depends on the plate load size as well as the stiffness of the overlaying layers. The work of Westergaard, dating back to the early 1920's, quantifies the support layers through the radius of relative stiffness, l.(23) Westergaard defined the radius of relative stiffness in the following way:

$$l = [D/k] \qquad (16)$$

Where:

l = Radius of Relative Stiffness for the Pavement System
D = Flexural Strength of Slab = $[Eh^3/12(1 - u^2)]$
k = Radius of Subgrade Reaction

Through the radius of relative stiffness term, the foundation support can be related to the surface stiffness. If the results of the backcalculation procedure are theoretically accurate, then a plot of Eh^3 versus kl^4 will yield a straight line. As figure 19 illustrates, the plot of the backcalculated field data generates a straight line with an intercept of 0.00, a slope of 0.085, a correlation coefficient (r^2) of 1.00, and a standard error of estimate (SEE) of 0.00. It is important to note that the backcalculation procedure derives the E and k-values independently of each other and with <u>no</u> reference to the definition of l.

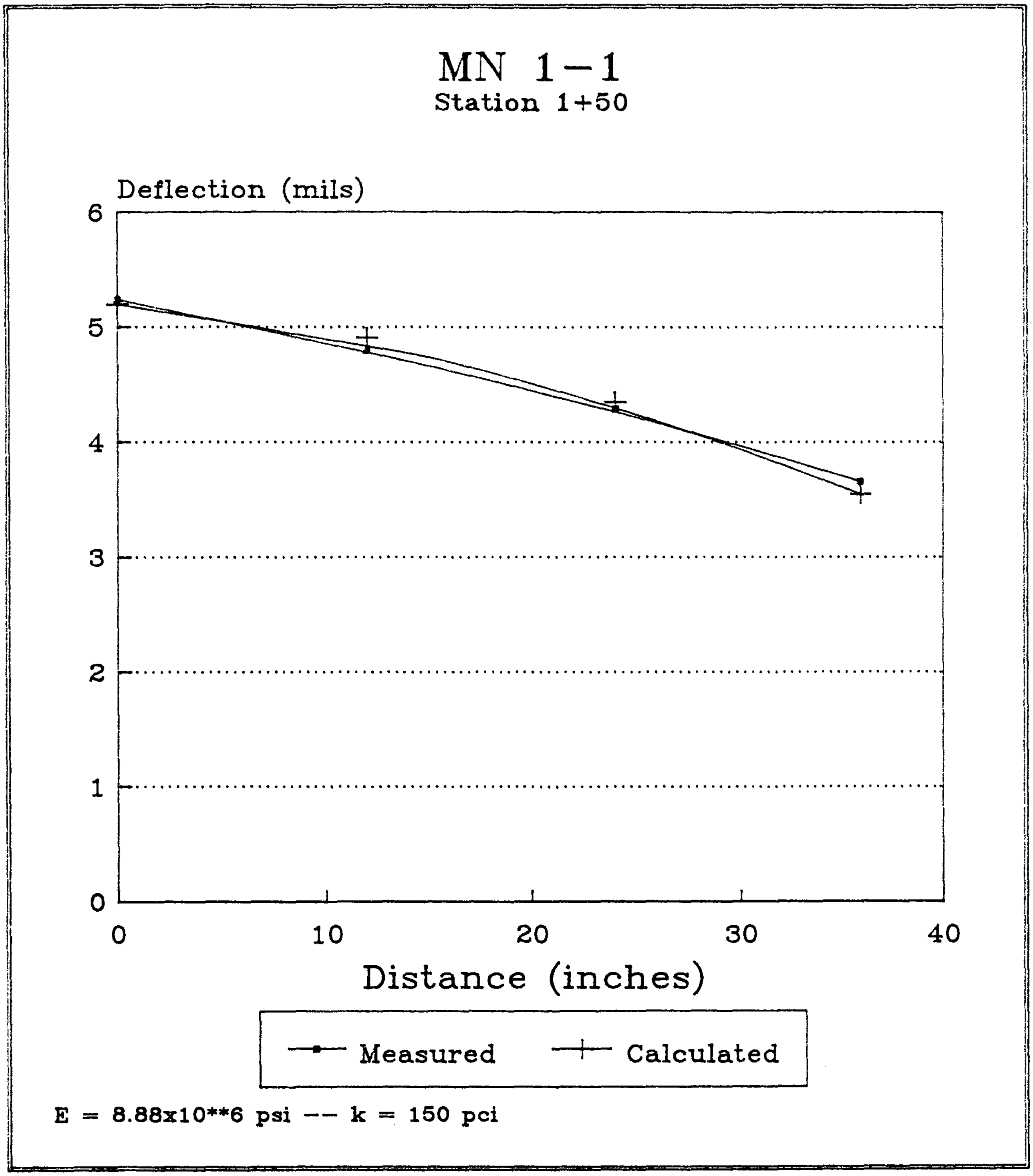


Figure 16. Deflection basin as measured by the FWD and as calculated using the closed form procedure.

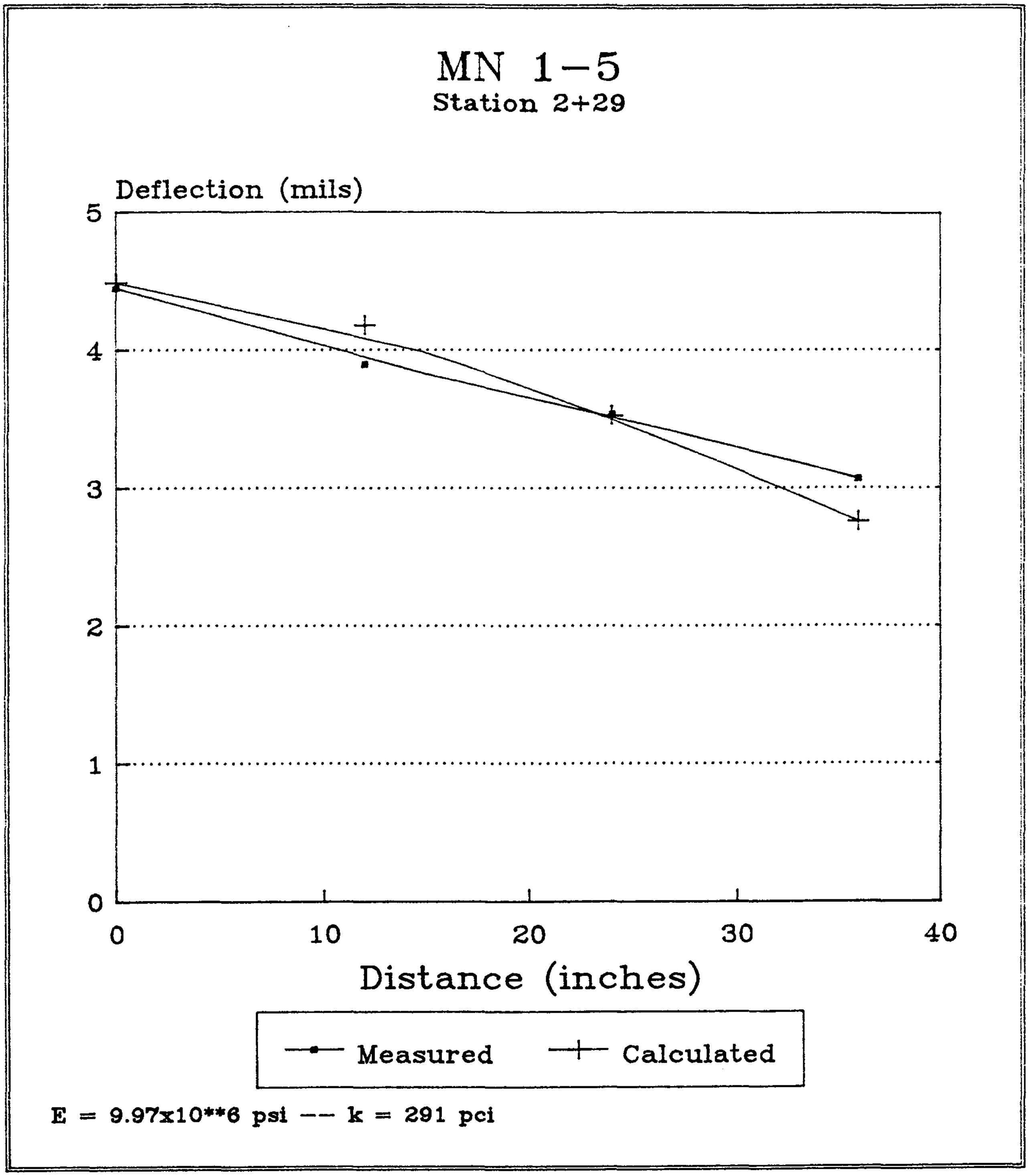


Figure 17. Deflection basin as measured by the FWD and as calculated using the closed-form procedure.

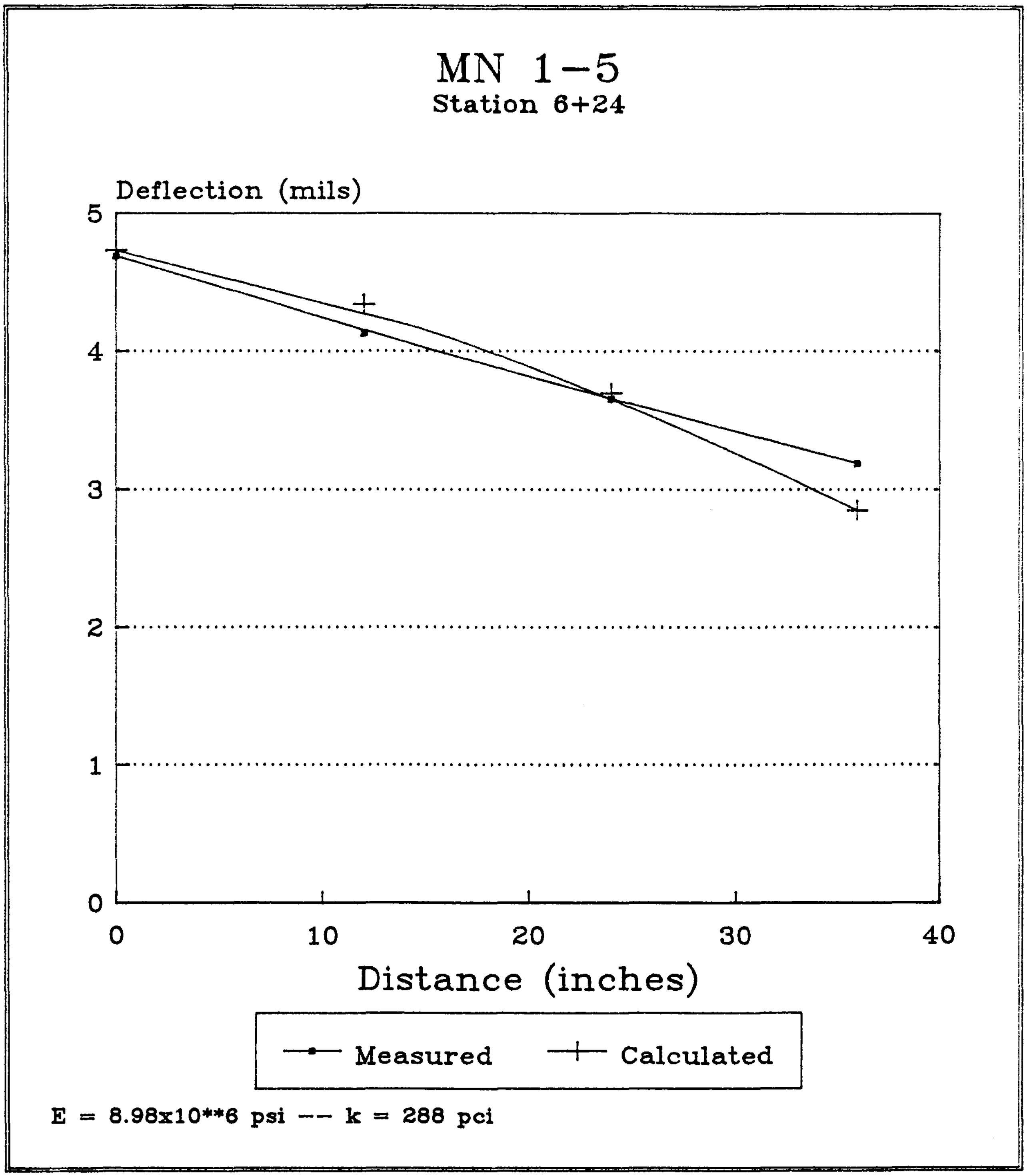


Figure 18. Deflection basin as measured by the FWD and as calculated using the closed-form procedure.

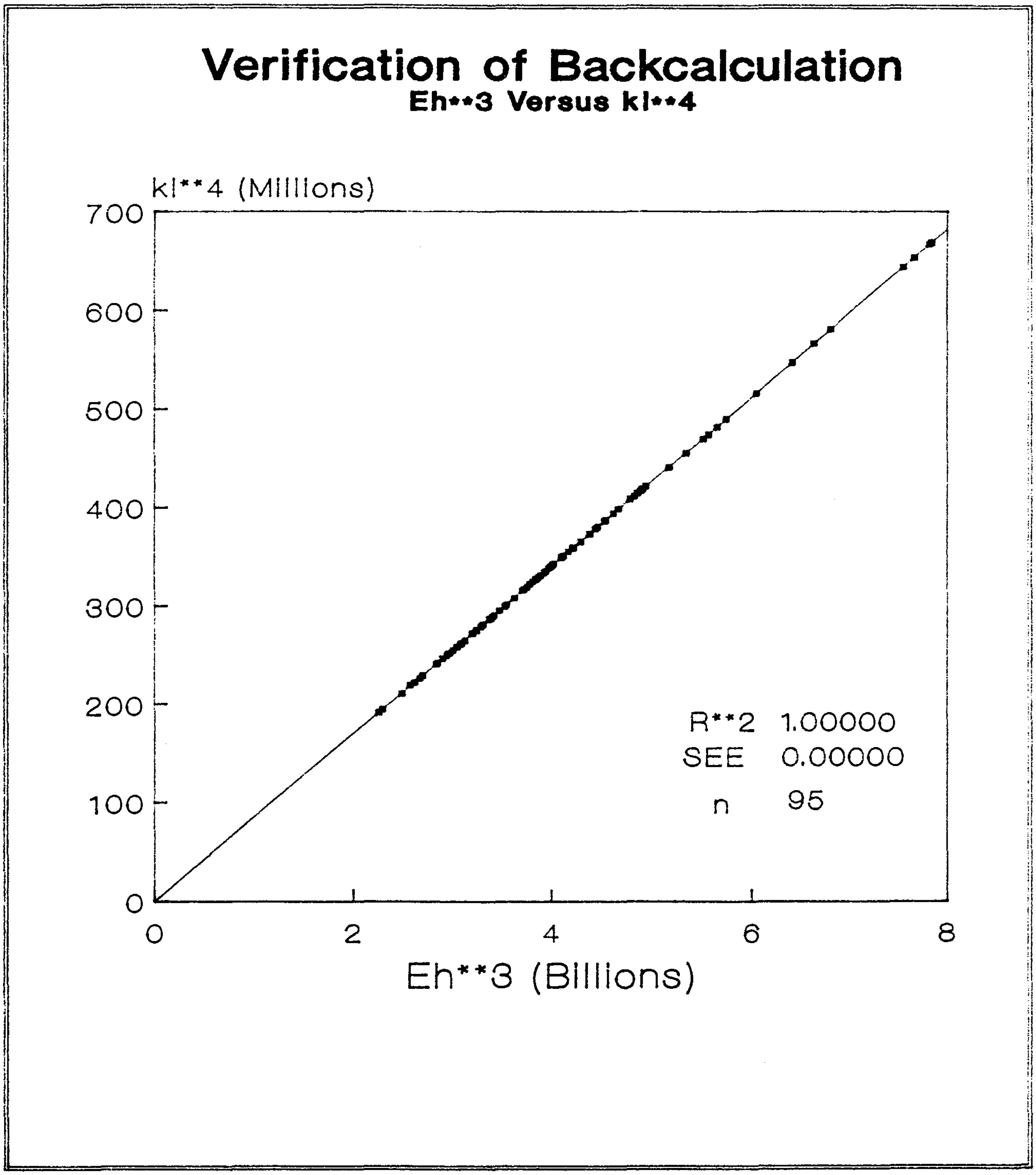


Figure 19. Eh**3 versus kl**4 for all Phase I sections.

This analysis shows that the procedure and the backcalculated values are accurate on a theoretical basis. The criticism of the method in terms of the high E-values at Rothsay has been refuted in the following ways:

1. In order to examine the stiffness of the surface, the slab thickness must be considered. Therefore, examination of the E value alone is not valid.

2. The actual measured deflection basins are nearly identical to the deflection basins as calculated using the ILLISLAB finite element program, for the given design parameters and backcalculated E and k values.

3. The plot of Eh^3 versus kl^4 shows that the E-values are theoretically accurate.

It has also been shown that the inverse relationship between the E and k-values is not a valid criticism. In fact, when examining the various parameters through Westergaard's radius of relative stiffness term, an ideal relationship exists between E of the slab and the k-value on top of the base.

6. ADVANTAGES OF CLOSED-FORM BACKCALCULATION PROCEDURE

The major strengths of the method used in this study, compared to other currently available backcalculation schemes, are listed below:

1. It is founded upon a sound and fundamental theoretical basis.

2. When the backcalculation is performed on a personal computer (computer program ILLI-BACK), execution time per deflection basin is very small (approximately 1 second). This permits the interpretation of a vast amount of NDT data in a very reasonable time period. In contrast, a typical backcalculation for a two-layer, slab-on-grade system using BISDEF takes about 600 CPU seconds to complete.(28,29)

3. Each sensor reading provides an independent estimate of the backcalculated parameters. This allows the engineer to investigate the sources of variability in field measurements.

4. Either of the two major subgrade models, namely the dense liquid or the elastic solid foundation, may be employed, providing a rare opportunity for meaningful comparisons between the two. The departure of in-situ behavior from the theoretical assumptions is reflected in the backcalculation statistics obtained. This affords an estimate of the relative location of the actual pavement system behavior within the spectrum defined by the two extreme soil idealizations.

5. There is no need for the provision of seed moduli in this approach. These moduli greatly affect the accuracy of other backcalculation schemes.(30) If the slab thickness is known, however, the slab modulus can be estimated. Conversely, if the slab modulus is known, the slab thickness is backcalculated.

6. The method is general in nature, and is not based on a limited number of particular cases as are database approaches.(31,32) This permits very useful and theoretically valid inferences to be made for a given pavement system by examining data collected from a wide variety of other dissimilar systems.

7. Application of the procedure to actual field data from recent or on-going projects has confirmed that it yields very realistic, consistent, and reliable results.(33) Calculations performed follow a definite and closed loop, all but eliminating the probability of calculation errors.

8. A number of interesting topics may be examined using this method. Among others, these include the effect of number and spacing of sensors; correlation between backcalculated and intrinsic system properties, particularly with respect to the dynamic effect of the NDT procedure; examination of the effect of friction and bonding between layers and comparison with results from an equivalent combined thickness approach; and field determination of properties necessary in other aspects of pavement design, such as dowel concrete interaction and the amount of load transferred by the critical dowel as well as by the entire dowel assembly.

CHAPTER 6 DATABASE DESCRIPTION

The database that is used for this research project was created using the UNIFY Relational Database Management System.(34) The UNIFY Database System is a collection of programs that is designed to allow the user to store and retrieve data. The system resides on an IBM PC-AT with 640 K RAM and a 30 Mb hard disk. No math chip is required for the system as the database itself does not perform any calculations.

The database includes projects from both Phase I (concrete pavement performance) and Phase II (structural overlay rehabilitation). The UNIFY system and the entire database system occupy approximately 20 Mb of hard disk space. Of this, the Phase I projects occupy approximately 12 Mb and the Phase II projects occupy approximately 8 Mb.

The data elements included in the database were based on the LTPP data collection guide.(1) Two databases were created to accommodate the large amount of data that was collected for this research project. The "Inventory" database contains inventory data, maintenance data, rehabilitation data, and environmental data. The "Monitoring" database contains information pertaining to monitored field data and traffic data. The type of information included in each database is listed in table 7.

Each of the items in table 7 has a separate data sheet. In addition to those data sheets, each database has a comment data sheet. These comment data sheets contain any additional or unique information that would otherwise not be included in the database. This additional data may prove useful in the analysis of the section.

Each screen in the database is set up to match the data sheets that were created for the project. The data is entered directly by the user from the information contained on these sheets. Screen data input consists of accessing the particular screen needed by entering through each database UNIFY Main Menu (figure 20), an Inventory Main Menu (figure 21), a Monitoring and Traffic Main Menu (figure 22), and a series of Sub-Menus. All screen information is tied to an Identification Key that consists of a section ID, a State code, and a project ID code. These codes have already been programmed into the system. No information can be added to any of the screens until the section ID is first entered. Information can be added to, modified, inquired upon, or deleted from the database at any time. This is a convenient means of keeping the database updated.

Once information has been input into the system, it can be accessed through the screens by individual section, by individual project, by each State, or by all sections projects and States. There is a provision in the database for running error checks. This routine checks for invalid character input for the field parameters on the screens.

There is also a provision for listing for data elements. Each database has its own data listing procedure and will list information from that particular database only. This data can also be listed by individual sections, individual projects, by each State, or by entire project. The error checking routine and the data listing procedure can be output to the screen, to a specified file, or to the printer.

Table 7. Listing of major data items contained in the database.

INVENTORY DATABASE

Inventory Data

Geometric, Shoulder, and Drainage Information
General Survey Information
Layer Descriptions
Longitudinal and Transverse Joint Data
Concrete Mixture Data
Base and Subbase Material Properties
Subgrade Properties
Age and Major Improvements

Maintenance Data

Historical Maintenance Information

Rehabilitation Data

Historical Rehabilitation Data

Environmental Data

General Environmental Data
Annual Historical Environmental Data
Average Monthly Historical Data

MONITORING DATABASE

Monitoring Data

Deflection Testing Data
Pavement Roughness Information
Distress Survey Information

Traffic Data

Average Daily Traffic
Percent Trucks
Equivalent Single-Axle Load Applications

```
[mainmenu]                    UNIFY Release 3.2
                               UNIFY Main Menu
IMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMM;
:                                                                              :
:                                                                              :
:                    1. Design and Create a New Data Base                      :
:                                                                              :
:                    2. Create or Modify Screen Forms                          :
:                                                                              :
:                    3. SQL - Query/DML Language                               :
:                                                                              :
:                    4. Edit SQL or RPT Command Files                          :
:                                                                              :
:                    5. Add, Modify or Delete Menus                            :
:                                                                              :
:                    6. Data Base Design Utilities                             :
:                                                                              :
:                    7. System Administration                                  :
:                                                                              :
:                                                                              :
:                                                                              :
: SELECTION: ripper                                                            :
HMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMM<
 F1-select ^U-up RET-down F2-home F3-previous F4-clear F5-exit F6-help /-more
```

Figure 20. Main UNIFY database menu.

```
[ripper]                      UNIFY Release 3.2
                          Ripper Data Base Main Menu
IMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMM;
:                                                                            :
:                                                                            :
:                     1. Ripper Required Information                         :
:                                                                            :
:                     2. Ripper Inventory Data                               :
:                                                                            :
:                     3. Ripper Maintenance Data                             :
:                                                                            :
:                     4. Ripper Environmental Data                           :
:                                                                            :
:                     5. Ripper Rehabilitation Data                          :
:                                                                            :
:                     6. Ripper Error Checking Routines                      :
:                                                                            :
:                     7. Ripper Data Listing Procedure                       :
:                                                                            :
:                                                                            :
:                                                                            :
: SELECTION:                                                                 :
HMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMM<
 F1-select ^U-up RET-down F2-home F3-previous F4-clear F5-exit F6-help /-more
```

Figure 21. Inventory database menu.

```
[ripper]                      UNIFY Release 3.2
                        Ripper Monitoring and Traffic Menu
IMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMM;
:                                                                            :
:                                                                            :
:                                                                            :
:                                                                            :
:                       1. Required Ripper Data Base Info                    :
:                                                                            :
:                       2. Ripper Monitoring Data - Sheet 1                  :
:                                                                            :
:                       3. Ripper Monitoring Data - Sheets 2-12              :
:                                                                            :
:                       4. Ripper Traffic Data                               :
:                                                                            :
:                       5. Monitoring Error Checking                         :
:                                                                            :
:                       6. Ripper Data Listing Procedure                     :
:                                                                            :
:                                                                            :
:                                                                            :
: SELECTION:                                                                 :
HMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMM<
 F1-select ^U-up RET-down F2-home F3-previous F4-clear F5-exit F6-help /-more
```

Figure 22. Monitoring database menu.

CHAPTER 7 REFERENCES

1. Rauhut, J. B., M. I. Darter, R. L. Lytton, P. R. Jordahl, M. P. Gardner, G. L. Fitts, and K. D. Smith, "Data Collection Guide for Long-Term Pavement Performance Studies," Draft Report Prepared for the Strategic Highway Research Program (SHRP), January 1988.

2. Smith, K. D., M. I. Darter, J. B. Rauhut, and K. T. Hall, "Distress Identification Manual for the Long-Term Pavement Performance (LTPP) Studies," Draft Report Prepared for the Strategic Highway Research Program (SHRP), December 1987.

3. Crovetti, J. A. and M. I. Darter, "Appendix C - Void Detection Procedures," Appendix to Final Report, NCHRP Project 1-21, March 1985.

4. Foxworthy, P. T., "Concepts for the Development of a Nondestructive Testing and Evaluation System for Rigid Airfield Pavements," Ph.D. Dissertation, June 1985.

5. "Traffic Monitoring Guide," U.S. Department of Transportation, Federal Highway Administration, Office of Highway Planning, June 1985.

6. Darter, M. I., J. M. Becker, M. B. Snyder and R. E. Smith, "Portland Cement Concrete Pavement Evaluation System," National Cooperative Highway Research Program Report No. 277, September 1985.

7. Garza, I. F., W. D. Cunagin, and T. Scullion, "Traffic Monitoring Data Software (Phase I) - Users Manual," Draft Report, Contract DTFH61-86-Z-00124, July 1988.

8. Cunagin, W. D., "Recommended Traffic Data Collection Procedures for the Strategic Highway Research Program Long-Term Pavement Performance Studies," Draft Report Prepared for SHRP, March 1988.

9. Majidzadeh, K., and G. J. Ilves, "Structural Design of Roadway Shoulders," FHWA/RD-86/089, Volume 1, December 1985.

10. Dempsey, B. J., M. I. Darter, and S. H. Carpenter, "Improving Subdrainage and Shoulders of Existing Pavements - State of the Art," FHWA/RD-81/077, August 1982.

11. "Guide for the Design of Foundations and Shoulders for Concrete Pavements," ACI 325.3R-85, 1987.

12. "AASHTO Guide for the Design of Pavement Structures," American Association of State, Highway, and Transportation Officials, 1986.

13. Moulton, L. K., "Highway Subdrainage Manual," Federal Highway Administration, Report No. FHWA-TS-80-224, 1980.

14. Talbot, J. R. and D. C. Ralston, "Earth Dam Seepage Control - SCS Experience in Seepage and Leakage from Dams and Impoundments," Proceedings, Geotechnical Engineering Division Symposium, American Society of Civil Engineers, 1985.

15. Carpenter, S. H., M. I. Darter, and B. J. Dempsey, "A Pavement Moisture Accelerated Distress (MAD) Identification System," Vol. I, FHWA/RD-81/079, Vol. II, FHWA/RD-81/080, September 1981.

16. Haynes, J. A., and E. J. Yoder, "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test," Highway Research Record No. 39, 1963.

17. Hole, F. D., "Suggested Terminology for Describing Soils as Three Dimensional Bodies," Soil Science Proceedings, Vol. 17(2), 1953.

18. Thornthwaite, C. W., "An Approach Towards a Rational Classification of Climate," Geographical Review, Vol. 38, 1948.

19. Liu, S., and R. L. Lytton, "Rainfall Infiltration, Drainage and Load-Carrying Capacity of Pavements," Transportation Research Board, Research Record No. 993, 1985.

20. Ioannides, A. M., "Dimensional Analysis in NDT Rigid Pavement Evaluation," Journal of Transportation Engineering, ASCE, July, 1988.

21. Hoffman, M. S., and M. R. Thompson, "Mechanistic Interpretation of Nondestructive Pavement Testing Deflections," Civil Engineering Studies, Transportation Engineering Series No. 32, Illinois Cooperative Highway and Transportation Research Program Series No. 190, University of Illinois, at Urbana, IL., 1981.

22. Ioannides, A. M., Discussion of "Response and Performance of Alternate Launch and Recovery Surfaces that Contain Layers of Stabilized Material," by R. R. Costigan and M. R. Thompson, Transportation Research Record 1095, Transportation Research Board, 1986.

23. Westergaard, H. M., "Stresses in Concrete Runways of Airports," Highway Research Board Proceedings, 19th Annual Meeting, 1939.

24. Ioannides, A. M., M. R. Thompson, and E. J. Barenberg, "Westergaard Solutions Reconsidered," Transportation Research Board, National Research Council, Washington, D.C., 1985.

25. Losberg, A., "Structurally Reinforced Concrete Pavements," Doktorsavhandlingar Vid Chalmers Tekniska Hogskola, Goteborg, Sweden, 1960.

26. Krauthammer, T. and H. Khanlarzadeh, "Numerical Assessment of Pavement Test Sections," Transportation Research Record Number 1117, 1987.

27. Timoshenko, S. and S. Woinowsky-Kreiger, Theory of Plates and Shells, 2nd Edition, McGraw-Hill, New York, 1959.

28. Bush, A. J., and D. R. Alexander, "Pavement Evaluation Using Deflection Basin Measurements and Layered Theory," Transportation Research Record 1022, Transportation Research Board, National Research Council, Washington, D.C.,1985.

29. Van Cauwelaert, F. J., W. J. Barker, T. D. White and D. R. Alexander, "A Competent Multilayer Solution and Backcalculation Procedure for Personal Computers, " Presented at the First International Symposium on Nondestructive Testing of Pavements and Backcalculation of Moduli, June 29-30, 1988.

30. Mahoney, J. P., N. F. Coetzee, R. N. Stubstad, and S. W. Lee, "A Performance Comparison of Selected Backcalculation Computer Programs," Presented at the First International Symposium on Nondestructive Testing of Pavements and Backcalculation of Moduli, ASTM, June 29-30, 1988.

31. Anderson, M., "A Data Base Method for Backcalculation of Composite Pavement Layer Moduli," Presented at the First International Symposium on Nondestructive Testing of Pavements and Backcalculation of Moduli, June 29-30, 1988.

32. Tia, M., and K. S. Eom, "Development of DBCONPAS Computer Program for Estimating of Concrete Pavement Parameters from FWD Data," Presented at the First International Symposium on Nondestructive Testing of Pavements and Backcalculation of Moduli, June 29-30, 1988.

33. Ioannides, A. M., E. J. Barenberg, and J. A. Lary, "Interpretation of Falling Weight Deflectometer Results Using Principles of Dimensional Analysis," Submitted for publication, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 18-20, 1989.

34. UNIFY Relational Database Management System, Version 3.2, Unify Corporation, 1985.

CHAPTER 8 ANNOTATED BIBLIOGRAPHY

1. EXPERIMENTAL PROJECTS AND PAVEMENT PERFORMANCE

"Appropriateness of Design, Construction and Rehabilitation Actions on Concrete Pavement Sections of Interstate 75 in Sarasota and Manatee Counties," Office of Inspector General, State of Florida Department of Transportation, July 1986, 56 pp.

This report analyzes the decision-making process involved in the design and construction of the section of I-75 which is badly deteriorated. This pavement required major rehabilitation only 4 years after construction, when it was projected to not need rehabilitation for 25 years. The recommendations mainly concern the processes involved in the decision-making and do not thoroughly cover a technical analysis of the pavement.

Armaghani, J. M., T. J. Larsen, and L. J. Smith, "Design-Related Distress in Concrete Pavements," Concrete International, August 1988, pp. 43-49.

This article discusses the causes of premature cracking in a PCC pavement on an econocrete base. Pertinent design variables include the use of a bond breaker between concrete layers, tied and doweled joints, and variable, skewed transverse joints. The results of tests, analyses, and field inspection of this pavement, on I-75 in Manatee County, Florida are presented.

Armaghani, J. M., T. J. Larsen, and L. L. Smith, "Temperature Response of Concrete Pavements," Florida Department of Transportation, Paper Presented at the 66th Annual Meeting of the Transportation Research Board, January 1987, 39 pp.

A specially designed Test Road was constructed to simulate actual design features of Florida Highways. This Test Road has been instrumented with linear variable differential transducers (LVDT's) and thermocouples at various locations. A Data Acquisition and Control Unit was used to record and store simultaneously the pavement displacements and the temperatures at specified time intervals. Vertical displacements at the slab corners, edge and center were collected and from 1983 to present and evaluated. The findings indicate that the pavement resistance to traffic loads would vary substantially during the daily temperature cycle.

Arnold, C. J., M. A. Chiunti, and K. S. Bancroft, "Jointed Concrete Pavements in Michigan: Design, Performance and Repair," Proceedings, Second International Conference on Concrete Pavement Design, Purdue University, April 1981, pp. 67-77.

The report covers background information concerning the performance and problems related to postwar pavements with 99-ft (30.2 m) reinforced slabs, load transfer, and base plates under the joints. New pavements have been designed with successively shorter slab lengths. Load transfer and

reinforcement are still used. A major installation of experimental pavement (U.S. 10, north of Clare) is discussed. Experience to date has resulted in a recent change to incorporate free draining base materials with subbase drains in future Interstate projects. The effects of base drainage on the performance of the concrete pavement as well as the inter-relationships with aggregate quality are demonstrated for an experimental installation having extreme variations in drainability. A few details of concrete shoulder design and some examples of compatible slab length considerations are suggested, along with brief comments on corrosion-resistant load transfer dowels.

Arnold, C. J., "The Relationship of Aggregate Durability to Concrete Pavement Performance, and the Associated Effects of Base Drainability," Research Report No. R-1158, Michigan Transportation Commission, January 1981, 22 pp.

This report discusses the impact of base drainability on pavement performance on U.S. 10 at Clare, Michigan. The conclusions are that aggregate sources should be carefully studied to avoid D-cracking problems, and poor base drainage has a deleterious effect on concrete pavement performance.

Bryden, J. E. and R. G. Phillips, "The Catskill-Cairo Experimental Rigid Pavement: A Five-Year Progress Report," Research Report 17, Engineering Research and Development Bureau, New York State Department of Transportation, November 1973, 59 pp.

This test road combines a variety of subgrade conditions with various subbase and base materials, as well as several pavement designs. In all, 30 different test sections were included for study of combinations of naturally occurring and experimental features. Carrying traffic since 1968, all test sections are performing well. The only noticeable distress to appear so far is mid-slab and double-joint cracking in some sections. Sections with 20-ft (6.1 m) nonreinforced slabs show some potential advantages over 60-ft 10-in (18.5 m) reinforced slabs. Sections without transverse load-transfer devices are performing as well as those with them. Since the plate-bearing test has been unable to detect differences in pavement strength between the test sections or at different times of the year, serious doubt is cast on its value.

Bryden, J. E. and R. G. Phillips, "The Catskill-Cairo Experimental Rigid Pavement: Construction and Materials Testing," Research Report 2, Engineering Research and Development Bureau, New York State Department of Transportation, December 1971, 47 pp.

Extensive testing during construction provides necessary background for evaluation of relative performance of the experimental features. In addition, the following conclusions are among the most important presented concerning the validity of certain variables and the overall experimental design: (1) the two types of subbase materials used, for all practical purposes, are identical, (2) the slipform paver is a fast, efficient method of constructing concrete pavement, (3) the validity of the plate bearing test is extremely doubtful, and (4) the pavement thickness variable is invalid due to construction variation.

"The Catskill-Cairo Experimental Rigid Pavement: Construction and Materials Testing, Supplement: Field and Laboratory Test Results." No Date, 16 pp.

Subgrade properties and plate bearing test results on subgrade, base and surface for the Catskill-Cairo experimental project are presented in this document.

Chiunti, M. A., "Experimental Short Slab Pavements," Construction Report, Research Report No. R-1016, Michigan State Highway Commission, August 1976, 20 pp.

This report describes the pavement construction of the experimental project on U.S. 10, near Clare, Michigan. It provides construction details of the three different designs used in the experiment, along with a description of Michigan's standard pavement design. Design variables for the project include joint spacing, base type, drainage, and load transfer.

Christory, J. P., "Experience des Autoroutes de Degagement de la Region Parisienne," <u>Fifth International Symposium on Concrete Pavements</u>, Aix-La-Chapelle, June 1986.

Two projects are described in the Paris region that have the following characteristics: widened lanes, tied shoulders, treated subgrade, and lean concrete bases. They are on well-traveled routes with 8-15 percent truck traffic.

Cook, J. P., I. Minkarah, and J. F. McDonough, "Determination of Importance of Various Parameters on Performance of Rigid Pavement Joints," Research performed under agreement No. 3061E, Ohio Department of Transportation in Cooperation with the FHWA, University of Cincinnati, August 1981, 32 pp.

This report is a continuation of the studies done on the experimental project on Route 23 at Chillicothe, with the benefit of observations over a longer period of time. Some of the recommendations resulting from this study are: design of the joint seal in relation to both short-term and long-term movement; joint spacing on stabilized bases to control mid-slab cracking and reduce vertical joint deflection, and on aggregate bases to delay mid-slab cracking; the continued use of dowel bars, but not of coated dowels; measurement of vertical joint deflection and of temperatures of both the pavement and the air.

Croney, J., "The Performance of Experimental Pavements Constructed on Normal Highways," <u>The Design and Performance of Road Pavements</u>, Transport and Road Research Laboratory, 1977.

This chapter discusses the performance of experimental portland cement and asphalt concrete experimental pavements constructed in Britain during the period from 1930 to 1966. For the PCC pavements, variables such as slab thickness and type and thickness of the subbase were considered. In

addition, slab length, amount of reinforcement, the effectiveness of load transfer devices, and the feasibility and economics of using prestressed slabs were investigated.

Delton, J. P., "Non-Conventional vs. Conventional Concrete Pavements in Arizona," Proceedings, Second International Conference on Concrete Pavement Design, Purdue University, April 1981, pp. 351-358.

Pavement faulting or step-off has been one of the major problems on Arizona's portland cement concrete pavements. Other problems have included warping and curling, texture loss, and shoulder joint failures. In 1974 work was initiated in an attempt to solve some or all of these problems by building and studying "non-conventional" portland cement concrete pavement sections on State Route 360, the Superstition Freeway. These sections are adjacent to a conventional section (9-in [229 mm] concrete slab on 6-in [102 mm] cement-treated base over a 4-in [102 mm] aggregate base). To date, full-depth 13- and 11-in (330 and 279 mm) concrete pavements placed directly on the subgrade and a prestressed 6-in (152 mm) concrete pavement on a 4-in (102 mm) lean concrete base have been constructed. Currently, a 9-in (229 mm) concrete pavement on 4-in (102 mm) of lean concrete base is under construction. The main items of study are joint performance and ride quality. Full-depth concrete pavements result in lower joint deflections in comparison to conventional concrete pavements. The additional slab thickness and joint surface area do not overcome the effects of joint separation from shrinkage due to curing and temperature changes. Loss of rideability is somewhat less on full-depth concrete pavements compared to conventional concrete pavements.

Eisenmann, J., D. Birmann, and G. Leykauf, "Research Results on the Bond Between Cement Treated Subbases and Concrete Slabs," International Seminar on Drainage and Erodibility at the Concrete Slab-Subbase-Shoulder Interfaces, Paris, France, March 1983, 23 pp.

In West Germany, concrete construction principles are applied which ensure a bond at the interface of slab and cement-treated base (CTB). The bond stresses and deflections due to traffic are reduced and the possibility of pumping phenomena decreased. The measures to promote bond in the interface are restricted to a thorough cleaning by brooms and to keeping the CTB wet for several hours before concrete paving. In addition to behavior at the slab/base interface, full-depth construction, the effect of dowels on pumping, and concrete overlays are discussed.

"Experimental Project-Concrete Pavement Interstate 95, Nash and Halifax Counties, North Carolina," Internal North Carolina Department of Transportation Reports, 1968-1984.

The performance of the experimental sections are traced in these reports which summarize annual condition surveys. The project is a concrete pavement on Interstate 95, in Nash and Halifax Counties. Variables of the experimental design include: joint orientation, load transfer, JPCP vs. JRCP, base type and thickness, and joint spacing.

"Experimental Short-Slab Pavements," Michigan Project No. 73 F-136, Work Plan No. 34, Michigan State Highway and Transportation Commission, March 1974, 41 pp.

The purpose of this project is to compare the performance of several types of pavement systems. Six experimental pavement types will be constructed for comparison with Michigan's standard jointed reinforced concrete pavement. These pavements will be constructed at two different locations. The variables will be: JPCP without dowels; JRCP with dowels; and asphalt-treated bases (permeable and nonpermeable).

Gluais, G., and J. L. Nissoux, "The Thick Concrete Slab Without Foundation: Initial Results of Five Experimental Projects," Bulletin de Liaison des Laboratoires des Ponts et Chaussees, No. 95, May-June 1978, pp. 91-98.

Two American projects are at the origin of studies conducted in France on the thick concrete slab without subbase. The advantages of such a structure have been judged sufficient to justify the extent of the research undertaken to define and develop the optimum conditions of use of this new concrete pavement structure. This article describes the characteristics of the five experimental pavements built between 1977 and 1978, and gives an initial balance sheet of the conclusions drawn. Descriptions of some of these pavements have already been published. The evolution of these pavements under traffic will make it possible to specify the level of traffic beyond which special characteristics of the improved subgrade (drainage course or treated course) will have to be imposed. The use of a drainage course as an improved subgrade is indeed unusual, and makes it difficult to feed the spreader with concrete.

Gregory, J. M., "The Performance of Unreinforced Concrete Roads Constructed Between 1970 and 1979, Research Report 79, Transport and Road Research Laboratory, 1987, 17 pp.

Thirty-six nonreinforced concrete pavements constructed between 1970 and 1979 were surveyed to provide information of the structural performance, the performance of joints, and the surface characteristics. A statistical analysis of the data was performed to identify a number of relationships related to design, materials, construction, and maintenance.

Halverson, A. D., "Concrete Pavements on Treated Bases, Long-Term Performance Report—1986" Investigation No. 193, Office of Materials, Standards, and Research, Minnesota Department of Transportation, 51 pp.

This report is a follow-up to the 1972 study of the sections located on I-94 near Rothsay, Minnesota, constructed in 1972. At this time, the pavement exhibited extensive cracking and severe faulting and rehabilitation was needed. Prerestoration evaluation indicated that: doweled pavements outperformed the nondoweled pavements with respect to faulting; aggregate base sections show the least longitudinal cracking; performance is more consistent in the thicker pavement sections; 8-in (203 mm) thick pavement on cement-treated base (CTB) is the best performer of the 8-in (203 mm) sections

on the basis of long-term crack propagation; 9-in (229 mm) thick pavements on both CTB and aggregate base outperformed the 9-in (229 mm) pavement on asphalt-treated base (ATB) on the basis of crack propagation; nondoweled 9-in (229 mm) pavements on CTB and aggregate bases perform best on the basis of cracks/mile, but the degree of faulting is undesirable; doweled 9-in (229 mm) pavements on CTB and aggregate bases outperform the 9-in (229 mm) pavement on ATB with respect to cracks/mile; treated bases, especially the ATB, offer no significant advantage over conventional aggregate base, while doweled pavements offer a considerable advantage with respect to ride.

Hoffman, G. L., "Subbase Permeability and Pavement Performance," Research Project 79-3, Report No. FHWA-PA-RD-79-3, Pennsylvania Department of Transportation, January 1982, 43 pp.

An experimental project was constructed to demonstrate the feasibility of providing good support and good internal drainage at a competitive cost. An additional long-term objective of the project was to determine the significance of the permeability of subbase layer(s) materials on pavement performance. Five types of subbases, ranging from a very impermeable cement stabilized material to a very permeable, uniformly-graded crushed aggregate, were incorporated in the project. The study documented the manufacturing of the materials, the associated unit costs, and the ability of the contractor to handle, place and pave on the various subbases.

Hoffman, G. L., "Subbase Permeability and Pavement Performance," <u>Transportation Research Record</u> Number 849, 1982, pp. 12-18.

This is essentially a summary of the above report of the same title, describing the experimental drainage project constructed near Kittanning, Pennsylvania.

"Investigation of Roadway Design Variables to Reduce D-Cracking," Final Report 78-1, Missouri Cooperative Highway Research Program, December 1987, 33 pp.

In 1975, a field study was initiated to determine the effectiveness of certain design variables in reducing the occurrence of D-cracking in Missouri. Eight test sections were constructed on I-35 in Daviess County in 1977. Common to the test sections was a 9-in (229 mm) JRCP slab using the same type and source of aggregate, a 4-in (102 mm) base course, and the shoulder design, which included a permeable open-graded aggregate for drainage. The variables evaluated included different <u>coarse</u> aggregate types and maximum top sizes, the use of a moisture barrier between the slab and the base, and the use of different base courses, including permeable, open-graded aggregate, dense-graded aggregate, plant mix, and cement-treated. At the time the study was concluded, no evidence of D-cracking was found at any of the sections. Pumping was evident on all of the dense-graded aggregate bases.

"Joint Spacing in Concrete Pavements: 10-Year Reports on Six Experimental Projects," Highway Research Board, Research Report 17-B, 1956.

The experimental projects discussed were constructed in 1940-1941 in California, Kentucky, Michigan, Minnesota, Missouri and Oregon. The variables included: climatic region, contraction joint spacing, expansion joint spacing, load transfer, pavement thickness, and reinforcement.

Kazmierowski, T. J., G. A. Wrong, and W. A. Phang, "Design, Construction and Performance of Four Experimental Concrete Pavement Sections in Ontario," Ontario Ministry of Transportation and Communications, December 1985, 74 pp.

In order to evaluate the relative performance of several concrete pavement designs, an experimental rigid pavement was constructed in 1982 using four different pavement designs, three shoulder designs and two types of surface textures. A summary of the design and construction details and the initial performance results of the project is documented in this paper. Early performance observations indicate superior performance of the free-draining base materials. In addition, some anomalous behavior based on pavement cracking and roughness suggest additional areas of process control are warranted. Continuing performance verification of preliminary conclusions indicate the new designs have resulted in a significant saving in construction materials and costs, plus increased durability and performance.

Kazmierowski, T. J. and G. A. Wrong, "Six Year's Experience with Experimental Concrete Pavement Sections in Ontario," Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp.473-489.

This paper is a follow up to the previous report, including observations after six years of performance of the Ontario experimental concrete pavements. Significant findings included the poor performance of the lean concrete bases, the inadequacy of the nondoweled sections, a recommendation for widened concrete lanes in future construction, the need for timely joint sawing, and the need for high quality joint and crack sealing.

Kinne, M. S., "Performance Evaluation of Florida Econocrete Test Road," Research Report FL/DOT/BMR-85/290, Florida Department of Transportation, May 1985, 62 pp.

An experimental project was constructed to evaluate the use of econocrete. Four different tests were run on the Econocrete Base Test Road. The Mays Meter was used to collect data for the present serviceability index (PSI), the Benkelman Beam for 18- and 27-kip deflections, and the Dynaflect Dynamic Deflection Determination System to collect data leading to the joint efficiency. Using data from the testing after construction, linear regression analyses were performed.

Krauthammer, T. and H. Khanlarzadeh, "Numerical Assessment of Pavement Test Sections," Transportation Research Record, Number 1117, TRB, pp. 66-75.

A numerical study was performed at the University of Minnesota, Department of Transportation, for assessing and explaining the observed performance of several highway pavement test sections. The test sections that were considered in this study are located near Rothsay, on I-94, and near Olivia, on TH 71. For the Olivia site the objective was to investigate the effect of a thin layer of bituminous bond breaker on the reflective cracking of the new pavement slab, while for the Rothsay site the goal was to understand the differences between an asphalt-treated base (ATB) and an aggregate base in affecting pavement performance. The approach was to employ the finite element method and to perform deflection testing on the sections. The results from those simulations were compared to FWD test data for model correlation, and finally parametric studies were performed in order to assess the long-term behavior of the pavements under consideration. As a result of the study it was possible to derive preliminary relationships between tensile and shear stress ratios, which affect the long-term pavement performance and the pavement properties. Based on these results, it was possible to provide rational explanations of observed pavement conditions and to draw significant conclusions on improved procedures for pavement rehabilitation.

Larsen, T. J., and S. F. Mayfield, "Florida Econocrete Test Road Performance Evaluation," Research Report FL/DOT/BMR-84/288, Florida Department of Transportation, August 1984, 98 pp.

The construction, instrumentation, and performance of the 33 segments of the Econocrete Base Test Road is documented. In this report longevity and present serviceability were primarily utilized in ranking individual segments. In addition, an extensive deflection analysis was conducted using Benkelman Beams and Dynaflect. The testing program is fully discussed and preliminary raw data are summarized.

Larsen, T. J., and J. M. Armaghani, "Florida Econocrete Test Road: A 10-Year Progress Report," Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp. 547-560.

This paper provides an update on the performance of Florida's Econocrete Base Test Road. The 10-year performance study was based on the structural response in load deflection tests and the performance ratings using the Mays Meter. All concrete sections were compared to a the then Florida DOT standard concrete pavement section. The experimental sections generally performed equivalent or better than the concrete reference section.

Lichtenstein, M., "Experimental Section on Autoroute A9," Bulletin de Liaison des Laboratoires des Ponts et Chaussees, No. 94, March-April 1978, pp. 121- 132.

The Vinassan test section on Autoroute A9 tests three structures comprising 9.8, 7.9, and 6.7 in (250, 200, and 170 mm) of concrete slab, the concreted

width ranging from 27.2 to 36.1 ft (8.3 to 11 m), resting respectively on 5.9, 4.7, and 7.1 in (150, 120, and 180 mm) of gravel-cement mixture, the whole resting on a silty soil. The influence of the extra width of concrete is appreciable in respect to the deflections of the edge of the traffic bearing slab (35 percent reduction) and negligible in respect to the stresses and strains. Traffic on the motorway pavement varied between 3,700 and 6,000 vehicles per day between 1972 and 1977. The number of heavy vehicles is considerable: by the end of 1976 the pavement had carried 1.2 million heavy vehicles. Measurements of slab deflections after 5 years confirm the influence of slab thickness. Deteriorations are of the transverse cracking type, but are nonexistent in the 9.8 in (250 mm) slab.

Little, R. J., and L. J. McKenzie, "Performance of Pavement Test Sections in the Rehabilitated AASHO Test Road," Report No. FHWA-IL-PR-76, Illinois Department of Transportation, June 1977, 94 pp.

The performance of the original concrete sections from the AASHO Test Road has been studied. Findings indicate that the original AASHO performance equation for rigid pavements predicted a greater service life expectation for the thicker pavement slabs than was actually being observed. To agree with the observed performance, a modified performance equation that slightly increased the performance expectation for the pavement sections 8-in (203 mm) thick and reduced the expectation for the pavement sections 9.5-, 11-, and 12.5-in (241, 279, and 318 mm) thick was developed. Stabilized aggregate subbases, especially the Bituminous Aggregate Materials (BAM), improved rigid pavement performance. On the BAM subbase, rigid pavement sections developed fewer major cracks, and pavements with joints spaced at 100 ft (30.5 m) had the most uniform winter joint opening. Dowel bar corrosion was the primary cause of joint lockup in the rigid pavement test sections.

"Location, Description and Performance of Test Sections," Missouri Road Test, AASHO Satellite Program Investigation 62-2, Missouri State Highway Department Division of Materials and Research, Unpublished In-House Reports (Not for Publication).

This document includes updated evaluations (through 1985) of the AASHO Satellite Experimental Sections still in use in Missouri. There exist only four or five sections of the original concrete pavements that have not yet been overlaid. Construction and performance data are summarized for these experimental projects, which were constructed between 1962 and 1971.

MacLeod, D. R. and C. L. Monismith, "Performance of Portland Cement Concrete Pavements," Final Report TE 79-1, Institute of Transportation Studies, University of California, Berkeley, February 1979, 102 pp.

The basic objective of this research is to develop a performance model for concrete pavement behavior under repeated heavy loadings to be used as a system input to a pavement management system. The major portion of the study has been concentrated on highways located in the San Francisco Bay Area. This investigation examines the relationship between faulting, roughness, and traffic, and the relationship between slab breakup (cracking) and traffic.

Majidzadeh, K. and R. Elmitiny, "Long-Term Observations of Performance of Experimental Pavements in Ohio," Report No. FHWA/OH-81/009, July 1982, 144 pp.

This report presents long-term evaluation data and analyses for eight experimental projects constructed in Ohio. The study projects include both rigid and flexible pavements. Pavement age is currently approaching 10 years for some projects. The pavements were extensively monitored and tested at the time of construction and during 1979 and 1980 as part of this research study. Collected data included pavement condition rating (PCR) of visible distress, Dynaflect deflection, test properties of core and subgrade samples, and estimated remaining structural life and overlay requirements. Only one of the projects contained JPCP. The variables in that project were: pavement thickness, joint spacing, joint sealant, drainage, base type, and base thickness.

Minkarah, I. and J. P. Cook, "A Study of the Effect of the Environment on an Experimental Portland Cement Concrete Pavement," Research Report No. OHIO-DOT-19-75, August 1976, 66 pp.

The objective of the present study was to evaluate the effects of the pavement environment on an experimental concrete pavement in Ross County, Ohio. Variables included in the experimental pavement were joint spacing, subbase stabilization, coating of dowel bars, configuration of the saw cut, and the use of skewed joints. Horizontal slab movements caused by temperature and vertical movement of the slab ends under known axle loads were measured. A complete record is included of mid-slab cracking and crack growth. Also included is a summary of the surface spalling of the pavement and the spalling of the bottom of the pavement at the joints.

Minkarah, I. and J. P. Cook, "A Study of the Field Performance of an Experimental Cement Concrete Pavement," Executive Summary Report, Research performed under agreement No. 2634, Ohio Department of Transportation in Cooperation with the FHWA, University of Cincinnati, May 1975, 41 pp.

An experimental project is described on Route 23 in Chillicothe. The purpose of the investigation was to develop an improved contraction joint for concrete pavements. The factors which were considered included stabilized subbases, dowel coating, joint spacing, configuration of the saw cut and skewed joints. The movement of the pavements were measured and the effects of the variables on the condition of the pavement over a period of 1 year were studied. The surface condition of the pavement is still excellent, perhaps due to the relative newness of the sections.

Mori, K. Y., "Pavement Design and Rehabilitation in California," California Department of Transportation, 23 pp.

California's practice in the design and rehabilitation of asphalt concrete and jointed plain concrete pavements is described. The article discusses what are thought to be the most important factors in both design and rehabilitation. One of the major emphases is on the provision of positive structural section drainage.

Neal, B. F., and J. H. Woodstrom, "Faulting of Portland Cement Concrete Pavements," FHWA-CA-TL-5167-77-20, Final Report, July 1967, 73 pp.

This is the final report of a 10-year long study on faulting. It has been found that faulting begins almost immediately after a pavement is opened to traffic. To prevent or reduce the problem, it is necessary to eliminate as many of the factors as possible which lead to faulting. Of utmost importance is the elimination of the major sources of transportable fines, which are usually found in an erodible base and untreated shoulder material. The use of a lean concrete base (LCB) shows great potential for providing a nonerodible base. Some of the experimental shoulder treatments described herein may also be effective, but need to be tried in conjunction with the improved base. Rapid removal of free water from under the slab is highly desirable and two types of drainage systems are described.

Neal, B. F. and J. H. Woodstrom, "Performance of PCC Pavements in California," FHWA-CA-TL-78-06, California Department of Transportation, February 1978, 34 pp.

A rating system based on present serviceability index (PSI) was used to rate and periodically re-rate some 130 individual concrete paving projects of various ages. Indications are that many of the pavements will last 30 years or longer, but a few will fail in less than 20 years. Average ratings of new pavements, based on 20 projects, was about 4.3 PSI, which was disappointingly low. Discussed in the report are various features affecting pavement performance, such as joints, cracks, and surface texture. An end-result specification for weakened plane joints, which has been successfully implemented by CALTRANS on several projects, is included.

Neal, B. F., "California PCC Pavement Faulting Studies: A Summary," Report No. FHWA/CA/TL-85/06, California Department of Transportation, December 1985, 37 pp.

A summary of the findings from several California Department of Transportation studies of faulting since 1968 are presented in this interim report. Causes of faulting were discovered and mitigation measures developed. These measures have been implemented on new construction projects, but experimentation is going on for retrofitting older projects.

Neal, B. F., "Evaluation of Design Changes and Experimental PCC Construction Features," Report No. FHWA/CA/TL-85/07, California Department of Transportation, December 1985, 103 pp.

This report is divided into four parts. Part I deals with a continuously reinforced concrete pavement (CRCP) and other experimental features intended as design improvements to reduce pavement maintenance costs. Part II covers field trials with four different types of joint sealant materials. It also reports on the first edge drain installation in California for the purpose of removing surface-infiltrated water. Part III concerns experimental shoulder treatments, the prime variable being PCC shoulders. Part IV deals with other experimental features incorporated into construction projects under the FHWA Construction-Evaluated Research Program. These features include: (1) bridge

approach slabs constructed (during pavement rehabilitation) with accelerated-set concrete mixtures, (2) the use of asphalt-treated permeable base (ATPB) as both a drainage layer and a base for concrete pavement, and (3) the use of a cement-treated permeable base (CTPB) in a highway roadbed structural section as a drainage layer for ground water control. The use of reinforcement in pavements, joint seals in nonmountainous areas, and concrete shoulders is not considered cost-effective and should not be implemented in California. The use of edge drains has already been adopted as standard practice. Both ATPB and CTPB are considered excellent for drainage layers and satisfactory as bases for concrete pavements. Their use has also been adopted.

New Jersey Department of Transportation, HPR Study 7708, no author or publication date available.

This report describes an experimental project constructed in 1979 on Route 676 near Camden. Variables evaluated include stabilized and nonstabilized open-graded drainage layers.

Nissoux, J. L., "French Cement Concrete Pavements--Effectiveness of Recent Construction Practices," Paper Presented at the Second International Conference on Concrete Pavement Design, Purdue University, April 1981, 30 pp. + Figures.

The pavements with short slabs, no dowels and treated subbases appear to postpone and effectively slow down the evolution of degradations such as slab pumping, faulting, and cracking. The performance of recently built pavements with these improvements and handling a high traffic level is presently somewhat better than that of older pavements built according to the original "Californian" technique. Further investigations are still necessary. The performance of the thick slabs in service, some of which have handled about 1 to 2 million commercial vehicles, is satisfactory and makes it possible to hope that this structure will be suited to medium-traffic pavements. The cost of this structure, although often lower then that of competing structures, is not as attractive as indicated by the first estimates. The introduction of a draining layer is the main reason for this.

Nowak, J. R., and J. Gaunt, "The Full-Scale Unreinforced Concrete Experiment on Longford-Stanwell Road, B.379F: Performance During the First Twenty Years," Road Research Laboratory, Report LR 349, 1970.

This report describes the structural performance during the first 20 years of the experimental nonreinforced concrete pavement placed in 1948 on the Longford-Stanwell Road, B. 379. The road consists of about 2 km (1.25 mile) of two-lane single carriageway and comprises 26 experimental slabs of varying lengths. The objectives of the experiment are: (1) study the behavior of nonreinforced concrete slabs ranging in length from 45 ft to 690 ft (13.7 to 210 m) and having contraction joints at 15-ft (4.6 m) intervals, and (2) measure the horizontal movements of slabs, due to annual temperature changes, at expansion joints and also at contraction joints in selected slabs.

Nowak, J. R., "The Concrete Pavement-Design Experiment on Trunk Road A1 at Alconbury Hill: Twenty Years' Performance," TRRL Laboratory Report 887, Transport and Road Research Laboratory, 1979, 25 pp.

This report describes the structural performance during the first 20 years of experimental concrete pavements constructed in 1957. The experimental design included 35 different sections which had carried 13.5 million standard axles. The factors being evaluated were slab thickness, concrete strength, reinforcement, and the type and thickness of the base. It was found that slab thickness was the most important variable, that high strength concrete gave better performance than normal concrete, and that lean concrete bases were more effective than granular bases.

Page, G. C., and L. W. Harper, "Florida Econocrete Test Road Post Construction and Materials Report," Research Report FL/DOT/OMR-80/221, Florida Department of Transportation, November 1980, 130 pp.

Design, construction, construction control, materials testing, and project administration of a research project evaluating the feasibility of constructing a two-course pavement system utilizing three strength levels of econocrete as a base layer in conjunction with thin pavement surfaces are presented in this interim report. Monolithic and overlay composite construction and elastic jointed pavement techniques are discussed. The variations in the concrete sections of interest included different econocrete strengths, joint spacing, reinforcement, base type, and pavement thickness.

"A Performance Evaluation of Existing Plain and Reinforced Concrete Pavements in Wisconsin," Wisconsin Department of Transportation, November 1976, 33 pp + Appendices.

This report summarizes the results of plain and conventional pavement performance evaluations recently completed by Wisconsin Department of Transportation and FHWA personnel in Wisconsin. It concludes that plain nondoweled concrete pavements are superior in performance, less costly to construct, and require less maintenance than conventional reinforced pavements with dowels.

Perry, J. M., "Evaluation of Plain Concrete Pavement," Progress Report No. 2, Study No. 73-4, Pavement Management Section, Wisconsin Department of Transportation, June 1988, 48 pp.

This report updates a previous study of plain concrete pavements in Wisconsin, based on concern about severe faulting. Analysis of 1987 faulting survey data showed that many plain concrete pavements were reaching terminal serviceability levels after only 6 to 10 years. The use of doweled joints, shown to reduce faulting and improve load transfer, are recommended for future plain concrete pavements that are expected to carry moderate to heavy traffic.

Peterson, R., "Construction Report, Experimental PCC Concrete Paving," Project FF-3-002(14)211, Materials and Research Division, North Dakota State Highway Department, September 1978, 29 pp.

This project is located on U.S. 2 from Rugby to Leeds. A two-course monolithic pavement, where the lower course had a lower design strength than the surface course, was used on this project. The lower course consisted of econocrete composed of pit-run aggregate, fly ash, and cement. The surface course was regular PCC pavement. The contractor used two belt placer spreading machines and one paver to lay the 6-in (152 mm) econocrete base and the 3-in (76 mm) concrete surface. There was no reinforcement and no load transfer devices. Joint spacing was 14 ft to 18 ft (4.3 to 5.5 m) on a 20° skew.

Raisanen, D. L. and D. D. Anderson, "Concrete Pavements on Treated Bases," Final Report, Investigation No. 193, Office of Research Coordination, Minnesota Department of Highways, 1972, 24 pp.

The purpose of this study is to determine the feasibility of constructing a concrete pavement over asphalt-treated and cement-treated bases (ATB and CTB) and the structural requirements of a roadbed which would meet the increased demands of traffic volumes and loads expected in the future. The test project, located on I-94 near Rothsay, Minnesota, consists of two sections each of ATB, CTB and conventional aggregate base. Built in 1970, the concrete pavement on each test section was divided into a 9-in (229 mm) section and an 8-in (203 mm) section. In each section, there were two subsections, with and without dowels. The sections were not old enough to draw definitive conclusions as to the performance of the different types.

Reagal, F. V., E. M. Lancaster, J. W. Guinnee, J. R. Ostrander, "Missouri Road Test Proposal," Missouri State Highway Commission, (Not for Publication), No Date, 23 pp.

This study discusses the construction of satellite test sections in Missouri. The test program as proposed is quite extensive. Variables to be considered for the concrete sections are: pavement thickness, pavement type, load transfer, joint spacing, base type and slab thickness.

Santoro, R. R., and C. L. Younger, "Evaluation of an Experimental Contraction Jointed Pavement," Report No. FHWA/NJ-84/002, New Jersey Department of Transportation, July 1983, 35 pp.

This follow-up report presents the findings of a long-term monitoring effort to evaluate the performance characteristics of the subject pavement relative to New Jersey's standard expansion joint design. The study data indicate that neither the nonsealed contraction joints nor the terminal end anchor devices used in the design have performed as intended. The contraction joints have progressively opened due to the intrusion of incompressibles, resulting in the development of excessive pavement pressures.

Santoro, R. R., "Slipform Paving with an Experimental Contraction Joint Design, (Route I-80, Section 1P)", Research Report No. 75-007-7779, New Jersey Department of Transportation, April 1975.

A 9-in jointed plain concrete pavement, with 15-ft (4.6 m) joint spacing, dowels, and nonsealed joints was constructed in lieu of the Department's standard expansion joint pavement design. Findings from the study were disappointing because of the failure to achieve a significant improvement in rideability, the absence of a substantial cost reduction, and the increased construction requirements imposed by slipforming and the special contraction joint design employed. The absolute necessity for the timely sawing of transverse contraction joints was found to be the most critical phase of the overall pavement construction. Guidelines for the selection of safe sawing rates are presented in the event similar pavement construction is undertaken in the future. It is recommended that the Department continue to construct concrete pavements by conventional methods using the current standard pavement design.

Shober, S. F., "Evaluation of Plain Concrete Pavement," Progress Report No. 1, Research Unit, Wisconsin Department of Transportation, September 1975, 68 pp.

The intent of the research described in this report is to evaluate the performance of plain concrete pavements, with nondoweled joints and skewed joints. Early findings on this project indicated that nondoweled joints tended to fault more and earlier in the pavement's life than doweled joints. Dowels were also found to contribute to joint load transfer efficiency. These results were preliminary, as they had not sustained sufficient traffic to adequately evaluate their performance.

Spellman, D. L., J. H. Woodstrom, B. F. Neal, and P. E. Mason, "Recent Experimental PCC Pavements in California," Interim Report, Materials and Research Department, California Division of Highways, June 1973, 56 pp.

The construction of experimental concrete pavement sections in California is described. The predominant experimental feature was continuously reinforced concrete pavement with three different types of reinforcement. Also included were nonreinforced sections with (1) weakened plane joints at about one-half the normal intervals, (2) higher cement content, (3) over-designed thickness, and (4) a lean concrete (4-sack) base. Design and construction details are presented along with a comparison of the construction costs of the various sections. Early performance characteristics are also discussed. The pavements will be monitored periodically to determine relative performance.

Stark, D., "The Significance of Pavement Design and Materials in D-Cracking," Report No. FHWA/OH-86/008, December 1986, 92 pp.

A two-phase program was undertaken to verify, under field conditions, that reducing maximum aggregate particle size can minimize or eliminate D-cracking. This study was carried out also to determine the role of other materials and environmental factors in D-cracking which are not amenable to

laboratory study. One phase consisted of repeat pavement surveys of existing pavements to determine whether reducing maximum particle sizes has alleviated D-cracking. The other (primary) phase consisted of monitoring the performance of a test road near Vermilion, Ohio using visual inspections and moisture measurements and examinations of concrete cores. Visual inspections confirm that reducing the maximum particle size does minimize or eliminate D-cracking. Other observations indicate that concrete pavement on clay subgrade, stabilized and granular bases with and without artificial drains, and vapor barriers performed similarly with respect to the initial development of D-cracking. Joint sealant type had no significant effect on D-cracking. Moisture measurements of cores showed an increase in degree of saturation of concrete after 1 year, with a general leveling off after that period. Saturation levels were somewhat higher near the bottom than near the top of the slab. Examination of cores revealed that D-cracking is developing upward from the bottom of the slab. Other observations revealed that where maximum aggregate particle size was reduced to avoid D-cracking, a greater incidence of intermediate transverse cracking developed with faulting. The project included 104 200-ft (61 m) long test sections, constructed with various combinations of 11 different cross sections, 10 different material variables, and 11 other miscellaneous variables.

"Summary Report on the Econocrete Test Road, 1978 to 1982," Florida Department of Transportation, No Date, 4 pp.

This brief report is an attempt to summarize the data and experience on the Ft. Myers Econocrete Test Road. In this summary, an emphasis is placed on information which could be useful in comparing econocrete as a base material to other bases such as natural embankment or cement-treated soils.

Temple, W. H., "Performance Evaluation of Louisiana's AASHO Satellite Test Sections," Research Report No. FHWA-LA-79/122, Louisiana Department of Transportation and Development, July 1979, 42 pp.

A sampling of nine concrete and nine asphalt pavements were evaluated for performance. Also a records search provided information as to the structural rehabilitation required on Louisiana's 110 AASHO Satellite test projects which average 18 years of age. It was concluded that: (1) a majority of the concrete pavements constructed from the mid-1950's to the early 1960's are in good to excellent condition, (2) these pavements will meet or exceed their design traffic lives, (3) a majority of the asphalt pavements constructed from the mid-1950's to the early 1960's have not attained their design traffic lives prior to the end of their structural life, (4) the approximate time between initial construction and overlay is 13 to 15 years. Of the 49 concrete pavements, those which were not overlaid included variations in pavement thickness, base type and thickness.

Vyce, J. M., "A Summary of Experimental Concrete Pavements in New York," Report No. FHWA/NY/RR-88/141, New York State Department of Transportation, June 1988, 46 pp.

This report summarizes a 22-year study of concrete pavement design features. The study included construction of a test road with numerous variables, two roads with several major design changes, and several roads among the first to incorporate a major change in transverse load-transfer devices. The work involved the first major contract in New York to use a slip-form paver, the first short-slab nonreinforced pavements, the first concrete shoulders, and skewed transverse joints. Performance of all these items is discussed, along with a number of minor changes. Many types of measurements are reviewed, along with their practicality and importance in assessing pavement performance. In addition, several pieces of equipment were developed or refined and others were evaluated. This large monitoring program will provide standards for measurement and interpretation on future work. Many findings have been implemented and have improved performance and/or cost-effectiveness of concrete pavements in New York. Experience gained through this work also led to improved assessment of other features, including rationales for not investigating some design options.

Vyce, J. M. and R. G. Phillips, "The Catskill-Cairo Experimental Rigid Pavement: A Ten-Year Progress Report," Research Report 91, Engineering Research and Development Bureau, New York State Department of Transportation, July 1981, 17 pp.

The Catskill-Cairo experimental rigid pavement has been open to traffic for over 10 years with very little change in condition. Test sections include 60-ft 10-in (18.5 m) slab lengths reinforced with mesh and containing load-transfer devices, 20-ft (6.1 m) slab lengths with no mesh reinforcement, but both with and without load-transfer devices, all constructed on three different base types. All sections are in good condition, with minimal roughness, little faulting, and light cracking. More important, there are not significant differences in performance of the various designs. The absence of distress and of performance differences among the various designs is most likely due to the light traffic volumes carried by the test road. This situation led to the conclusion that none of the design alternatives can be eliminated based on performance of this test road after 10 years.

Vyce, J. M., "Short-Slab Unreinforced Concrete Pavement and Shoulders: A Five-Year Performance Summary," Research Report 95, Engineering Research and Development Bureau, New York State Department of Transportation, May 1982, 27 pp.

In 1975, two contracts for I-88 were built with short nonreinforced concrete slabs and concrete shoulders. They were separated by another contract with standard 63-ft 6-in (19.4 m) mesh-reinforced slabs and asphalt shoulders, built for control purposes. Also, two portions on each short-slab contract were built with slab lengths of 23-ft 4-in (7.1 m) and 26-ft 8-in (8.1 m), and one of these sections on each contract was constructed with no longitudinal joint between lanes. In addition, a concrete secondary road relocated as part of another nearby I-88 contract was placed 7 in (178 mm) thick without reinforcement, with slab lengths in the pattern of 18-22-16-20 ft (5.3-6.7-4.9-6.1 m). The results of intensive monitoring and observation on the experimental and control pavements, as well as recommended design considerations, are presented in this report. It was found that the relatively substantial distress was occurring on only one of the experimental contracts, indicating that several material, design and construction variations were responsible for a significant portion of the distress. These included the quality of the subgrade material, subbase thickness, and treatment of the longitudinal lane-shoulder joint.

"1985 Wisconsin Interstate Pavement Rehabilitation Study," Wisconsin Department of Transportation, Division of Highways and Transportation Services, 1985, 67 pp.

An evaluation of the condition and needs of Wisconsin's Interstate Highway System was performed in 1985 and presented in this report. This document updated the previous condition assessment (performed in 1982). Specific recommendations covering short- and long-term rehabilitation strategies were made for those sections having recommended rehabilitation years prior to 1989. In addition, many general conclusions and recommendations were made relative to the condition and needs of Wisconsin's Interstate Highway System. Among some of these general recommendations are: more use of positive subdrainage, further investigation of the effectiveness of cracking and seating, doweling of full-depth concrete repairs, use of positive load transfer devices (dowel bars) at transverse contraction joints, and use of tied PCC shoulders to eliminate many of the problems associated with AC shoulders (i.e., settlement, separation, saturated bases, and frost heave).

Wu, Shie-Shin and T. M. Hearne, Jr, "Performance of Concrete Pavement with Econocrete Base," Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp. 683-695.

This report describes a 3-lane section of I-85 in Randolph and Davidson Counties in North Carolina. The pavement was constructed of 11 in (279 mm) of plain doweled concrete slab, on a 5 in (127 mm) econocrete base. The transverse joint spacing was 25-23-19-18 ft (7.6-7.0-5.8-5.5 m). This pavement was opened to traffic in January 1984 and experienced pumping by the spring of 1984. It was found that the cross section was a bathtub design and that water entered the pavement from a number of sources and then flowed along the slab-base interface. Among other findings, it was recommended that positive drainage be provided, and that the use of inserts to form the longitudinal joint be stopped.

2. SUBDRAINAGE

Baldwin, J. S., and J. G. Jarvis, "Evaluation of a Free Draining Base Course," (NEEP No. 28), Research Project 58, West Virginia Department of Highways, February 1986, 71 pp.

The adverse effects of entrapped water within a pavement system are generally well known. In an attempt to reduce or eliminate the detriment, the West Virginia Department of Highways designed and constructed its first major project which utilized a free-draining concept. The free-draining system used is basically a 4-in (102 mm) bituminous-stabilized, open-graded base course, underlain by engineering fabric and connected to the surface drainage by numerous lateral aggregate-filled fabric underdrains. This final report documents the design, construction, and completion of a 3-year testing and evaluation program of the experimental pavement.

Baldwin, J. S., "The Use of Open-Graded/Free-Draining Layers in Pavement Systems-A National Synthesis Report," Transportation Research Record 1121, TRB, pp. 86-89.

The effects of excessive and uncontrolled water entrapped in the various components of a paving system are known or suspected to have been responsible for unsatisfactory highway performance to outright failures of both concrete and asphalt pavements. In order to eliminate or at least reduce the detriment, almost half of the highway and transportation agencies across the nation have been giving serious attention to the problem by designing and constructing free draining pavement systems. In an effort to ascertain just how much and what kind of attention is being given free draining pavements on a national scale and in order to gain some insight into the performance characteristics of such systems designed to date, the Transportation Research Board's committee on Subsurface Drainage prepared a questionnaire for national distribution in the fall of 1985. This report attempts to summarize the response to that questionnaire.

Carpenter, S. H., "Selecting AASHTO Drainage Coefficients," Presented at the 67th Annual Meeting of the Transportation Research Board, January 1989.

This paper documents a rational procedure that has been developed to analyze a pavement and its component materials in order to select the drainage coefficients required for the 1986 AASHTO design procedure for pavement structures.

Cedergren, H. R., "Open-Graded Aggregate Drainage Under Pavements," National Sand and Gravel Association Publication No. 125, January 1980, 19 pp.

This speech, presented at the 1980 annual convention of NSGA, discusses the background of nondrained pavements and their deterioration. It is suggested that quality drainage aggregates, when properly used, can be used to provide better pavements. The components of an effective subsurface drainage design include an open-graded base layer, a method of filtering out the intrusion of fines, collector pipes, and transverse outlets.

"Combatting Concrete Pavement Slab Pumping by Drainage of Interfaces and Use of Low-Erodability Materials," Permanent International Association of Road Congresses (PIARC), October 3, 1986, 80 pp.

The occurrence of pumping requires three conditions: heavy loads, a source of free water in the pavement, and erodible materials. The heavy traffic cannot be removed from a pavement, but the other two causes of pumping can be addressed. Information is presented in this report to allow designers and managers of pavement to provide improved drainage to pavements and to design base course with nonerodible materials.

de Beer, M., and E. Horak, "The Effect of Poor Drainage on Pavement Structures Studied Under Accelerated Testing," Draft ATC Paper, TP/15/87, 12 pp.

This paper reports the results of a study on the effect of Excess Pore Water Pressure (EPWP) on several types of pavement structures, including asphalt-treated bases, cement-treated bases, granular bases, and concrete bases. It was shown that nondurable materials must be avoided, especially in the upper layers of the pavement structure. Cemented base and granular base structures require preventive maintenance and adequate drainage to avoid moisture accelerated distress (MAD). Faulting and pumping on concrete pavement structures can also be limited by the use of durable base layers and concrete reinforcement to limit deflections.

Forsyth, R. A., G. K. Wells, and J. H. Woodstrom, "The Economic Impact of Pavement Subsurface Drainage," Public Works, January 1988, 5 pp.

Nondrained pavements or pavements in a wet condition sustain damage many times higher than dry pavements. Several different methods of draining pavements are discussed, including geotextiles, stabilized permeable layers, and edge drains. An attempt is made to quantify the benefits of drained pavements in terms of increased life and reduced costs. It is also suggested that benefits are to be derived from retrofitting edge drains in existing pavements.

Forsyth, R. A., G. K. Wells, and J. H. Woodstrom, "The Road to Drained Pavements," Civil Engineering, March 1987, pp. 66-69.

This article summarizes some of the background studies that have led California to incorporate new drainage requirements into its 1987 Highway Design Manual. Caltrans believes that it is conservative in predicting that its new, drained rigid pavement design will last 50 percent longer than its nondrained counterpart, and have 35 percent lower costs per square yard per year. Drainage items considered include geotextiles, asphalt treated permeable material (ATPM), edge drains, and a nontreated permeable material. California is also investigating cement-treated permeable material (CTPM).

Green, T. M. and E. C. Novak, "Drainage and Foundation Studies for an Experimental Short Slab Pavement," Research Report No. R-1041, Michigan State Highway Commission, February 1977, 30 pp.

This report covers an aspect of the experimental project located on U.S. 10. Specifically, it presents a characterization of the subgrade, subbase and base materials; describes the installation of subbase drains; discusses the testing methods used on the various materials and their results; and presents some preliminary observations on the performance of the sections.

Hallin, J. P., "Edge Drains Oust Excess Moisture," Roads & Bridges, March 1988, pp. 84-90.

The need for drained pavements is discussed in this article. Recommendations are presented for the construction of drainable bases in new pavements, covering gradation of the base layer, the use of edge drains, and transverse outlets. The use of filter materials and location of all of the drainage elements in the pavement structure are also discussed. Some guidelines for retrofitting drainage as a rehabilitation measure are also suggested.

Highlands, K. L. and G. L. Hoffman, "Subbase Permeability and Pavement Performance," Final Report, Research Project 79-3, Pennsylvania Department of Transportation, September 1987, 36 pp.

This project, constructed by PennDOT, demonstrated that open-graded, permeable subbase materials can be designed, are constructible, and competitively priced, while providing pavement support and good internal drainage. Five different subbase designs were constructed. The pavements were subjected to Falling Weight Deflectometer (FWD) testing and periodic roughness measurements to measure the pavements' performance.

Hoffman, G. L., "In-Situ Permeability of Bases (FPTD)," Report No. PA 80-11, Pennsylvania Department of Transportation, April 1981.

The Field Permeability Testing Device (FPTB) was evaluated in the field on a construction project on Routes 66 and 422 in Kittanning, Pennsylvania. The FPTD equipment was found to be theoretically sound in its approach to determining field permeabilities of aggregates.

Hoffman, G. L, "Pavement Base Drain Evaluation," Research Project 78-5, Report No. FHWA-PA-RD-78-5, Pennsylvania Department of Transportation, June 1981, 82 pp.

Portions of the Department's highway drainage system design have recently been revised. Essentially, the longitudinal drainage trench was moved closer to the pavement/shoulder joint, and the fine concrete sand layer was eliminated as a trench backfill material. The specified backfill material is the coarser crushed aggregate. This report deals with the evaluation of the effects of these changes on pavement performance and compares the new pavement drain system to the older pipe underdrain system at the same site.

Kozlov, G. S., "Improved Drainage and Frost Action Criteria for New Jersey Pavement Design, Volume III - Road Subsurface Drainage Design," Research Report No. 84-003-7740, New Jersey Department of Transportation, September 1983.

This volume is a technical guide presenting all aspects of subsurface drainage from design through construction and maintenance of underdrainage systems. Procedures with examples for calculating infiltration rates, pipe sizes, aggregate selection and sample specifications are provided. Basic solutions of ground water drainage are also included.

Kozlov, G. S., V. E. Mottola and G. Mehalchick, "Improved Drainage and Frost Action Criteria for New Jersey Pavement Design, Volume I - Investigations for Subsurface Drainage Design," Research Report No. 84-003-7740, New Jersey Department of Transportation, June 1983, 194 pp.

This volume describes a concerted effort to resolve the problem of internal (subsurface) road drainage. Field investigation of the existing conditions on New Jersey Highways indicated a definite need for such solutions. The fundamental objective of this project was to formulate the design methods and the construction and maintenance procedures for such a drainage system. In this report an assessment of the state of the art is presented along with a description of laboratory efforts to identify optimal materials for pavement drainage layers. As a result of those efforts, nonstabilized open-graded and bituminous-stabilized open-graded materials were developed.

Kozlov, G. S., V. E. Mottola and G. Mehalchick, "Improved Drainage and Frost Action Criteria for New Jersey Pavement Design, Volume II - Experimental Subsurface Drainage Applications," Research Report No. 84-003-7740, New Jersey Department of Transportation, July 1983.

This volume describes the initial investigation into the need for subsurface drainage in New Jersey. In addition, the construction, instrumentation and initial monitoring of several roadways employing an innovative underdrainage system containing open-graded bases and longitudinal drains is presented. Also documented are the results of model test track experiments performed at the University of Illinois.

"Longitudinal Edge Drains in Rigid Pavement Systems," Report No. FHWA-TS-86-208, Federal Highway Administration, July 1986, 135 pp.

This report, a summary of the findings of a four-State study of longitudinal edge drain systems in rigid pavements, discusses design philosophies, design criteria, construction practices, and field performance comparisons concerning longitudinal edge drains. Trench drains and drainable asphalt concrete layers were found to be cost-effective in terms of their original cost and in their ability to remove water. Performance comparisons with 7-year-old pavements without drainage provisions indicate that edge drains can extend pavement life. Maintenance of the edge drain system was cited as a major contributor affecting pavement performance.

Malasheskie, G. J., "Open-Graded Subbase to Provide All-Season Pavement Subdrainage," Pennsylvania Department of Transportation, Paper Prepared for Presentation at the 66th Annual Meeting of the Transportation Research Board, 1987, 15 pp.

The Pennsylvania Department of Transportation designed and constructed an experimental project located on Routes 66 and 422 near Kittanning, to demonstrate the feasibility of providing good support and good internal drainage to a pavement at a competitive cost. As part of the project, five types of subbases with a wide range of laboratory measured permeabilities were placed. The base courses included the Departments standard dense-graded granular material, cement-treated base material assumed impermeable, and permeable base materials with various degrees of permeability. It was concluded that an open-graded subbase (OGS) drainage layer in the pavements is not only possible, but practical and should lead to long-term good performance of the pavement system.

Marks, V. J., "Improving Subgrade Support Values with Longitudinal Drains," Interim Report, Project MLR-84-3, Iowa Department of Transportation, January 1984, 13 pp.

The program of monitoring Iowa pavements with a road rater and improving subgrade support values through installation of longitudinal drains is described. The Iowa Department of Transportation began installing longitudinal subdrains in 1978 at a depth of 24 in (610 mm). The trend in Iowa has been to deeper longitudinal drains with the present standard being 48 in (1220 mm) deep. A limited amount of data indicates that the deeper longitudinal drains are providing a greater benefit to the subgrade support value.

Mathis, D. M., "Design and Construction of Permeable Base Pavements," Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989, pp. 663-670.

This paper presents the state of the practice in pavement drainage for new or reconstructed asphalt concrete and portland cement concrete pavements. Permeable base practices throughout the United States are synthesized.

Merrien, P., and J. L. Nissoux, "Porous Concrete Hard Shoulders: Study of Materials," Bulletin de Liaison des Laboratoires des Ponts et Chaussees, No. 92, November-December, 1977, pp. 142-148.

The development of porous concrete hard shoulders in France since 1975 alongside hydraulic pavements has been undertaken in parallel with thorough research on the material in question. This article presents the most recent results concerning the choice of the method of compaction, the behavior of these concretes in the presence of pure water, and their resistance to fatigue. These positive results as a whole confirm the value of the use of this material to obtain a mass with high draining properties which laterally evacuates water from the paving foundation interface.

Moulton, L. K., "Highway Subdrainage Design," Report No. FHWA-TS-80-224, Federal Highway Administration, August 1980, 162 pp.

This report describes all of the major inputs which must be considered in the design of subdrainage systems for highways. Included are discussions on the various sources of water, on the importance of highway and subsurface geometry, on the importance of materials in subsurface design, on determining the quantity of water to be removed, and also on construction and maintenance procedures. Emphasis is placed on determining all requirements for quickly removing the water from the pavement system.

Ray, M., "A European Synthesis on Drainage, Subbase Erodability, and Load Transfer in Concrete Pavements," Proceedings, <u>Second International Conference on Concrete Pavement Design</u>, Purdue University, April 1981, pp. 27-39.

European research and field experience during the period from 1977 to 1081 are presented in this paper. For nondoweled pavements, improvements in load transfer due to reduction in slab length, widened lanes, increased coarse aggregate size, and reduced hydraulic and thermal shrinkage are discussed. For doweled pavements, developments include the reduction in the number of dowels, anticorrosion and slipping devices, placement accuracy, and vibration placement equipment. New draining cross sections in use in Europe are presented.

van Wijk, A. J., and C. W. Lovell, "Importance of Drainage to Rigid Pavement Performance," 24 pp.

The development of providing drainage for PCC pavements is discussed, along with the benefits to be obtained from a properly drained pavement. Pumping can be controlled through the use of properly designed permeable layers or edge drains in original construction. Retrofitting edge drains will not always be successful, especially in a pavement with fairly impermeable or erodible bases or shoulders.

van Wijk, A. J., and C. W. Lovell, "Prediction of Subbase Erosion Caused by Pavement Pumping," <u>Transportation Research Record</u> 1099, 1986, pp. 45-57.

The characterization of the surface erosion of rigid pavement base and shoulder materials is described. Three testing methods were used in the study. The results show that nonstabilized materials are not capable of resisting surface erosion in a concrete pavement. Laboratory-developed guidelines for the design of erosion-resistant base or shoulder materials are presented.

Wells, G. K., "Evaluation of Edge Drain Performance," Report No. FHWA/CA/TL-85/15, California Department of Transportation, November 1985, 52 pp.

Portland cement concrete pavements (PCCP) constructed in California are of the plain jointed design. With the passage of time, these concrete pavements generally develop faulting, or step-off, at the transverse joints. However, the magnitude and rate of this faulting varies considerably throughout the State. Faulting, through the pumping process, has been a major factor in slab cracking and subsequent poor pavement performance. Previous Caltrans research revealed that the mechanism of pavement faulting is caused by the pumping process. One of the major contributors is surface infiltrated water that has become trapped in the relatively impermeable structural elements. This study presents an evaluation of the effectiveness of edge drains as a method of providing rapid drainage of surface-infiltrated water and thereby delaying or preventing pumping and subsequent faulting. Over a 3-year period, research indicates that edge drains are very effective in reducing faulting of concrete pavements in California.

3. LOAD TRANSFER

Arnold, C. J., "Performance of Several Types of Corrosion Resistant Load Transfer Bars, for as Much as 21 years of Service in Concrete Pavements," Research Report No. R-1151, Michigan Department of Transportation, August 1980, 21 pp.

Corrosion resistant dowel bars have been used in Michigan's jointed pavements both on an experimental basis and as a design standard for many years. The results of pull-out tests and coring to examine in place dowels are presented here. Several different dowel coatings and pavements of different ages are included in this study. It was concluded that wider use of plastic coated dowels be considered.

Black, K. N., R. M. Larson, and L. R. Staunton, "Evaluation of Stainless Steel Pipes for use as Dowel Bars," Public Roads, Vol. 52, No. 2, September 1988, pp. 37-43.

This report presents the results from a laboratory study conducted to evaluate the effectiveness of using stainless steel pipes as dowel bars. Solid stainless steel dowel bars were tested, as well as two hollow stainless steel bars of different thickness, and one hollow bar filled with concrete. It was found that the concrete filled bars performed the best, perhaps because of their larger bearing area.

Bolourchi, Z., W. H. Temple, and S. C. Shah, "Evaluation of Load Transfer Devices," Research Report No. FHWA-LA-97, Louisiana Department of Highways, November 1975, 53 pp.

This report describes the procedures and findings of a study conducted to evaluate two types of load transfer devices used in Louisiana-- steel dowel bars and starlugs. It is concluded that for a given slab length the dowel bar projects exhibited less faulting, better load transfer capability, and less pavement deterioration. Several factors limit the relevancy of these findings.

"Load Transfer at Transverse Contraction Joints and Design of Concrete Pavements," PIARC Technical Committee on Concrete Roads, Report of Subcommittee No. 1, December 1978.

For several decades, doweling has been a controversial question among highway engineers. This report addresses the question "to dowel or not to dowel," through a review of the international literature on the subject and results from questionnaires received from 25 countries. In light of the many experiments and seemingly contradictory practices found in various countries, this report is an attempt to show cases for which the use of dowels is and is not necessary.

Okamoto, P. A., "Field Evaluation of Dowel Placement Along a Section of I-45 in Texas," Report to Demonstration Projects Division, HHO-41, FHWA, Construction Technologies Laboratories, Inc., May 1987, 46 pp.

The results from a field investigation into the effectiveness of the radar technique for measuring dowel bar misalignment and the effectiveness of an automatic dowel bar inserter to properly place dowel bars in rigid pavements are reported here. It was concluded that the radar device has several limitations, but is reasonably effective in evaluating dowel bar misalignment. The automatic dowel bar inserter was also found to perform well in comparison to conventional basket construction.

Stelzenmuller, W. B., L. L. Smith, and T. J. Larsen, "Load Transfer at Contraction Joints in Plain Portland Cement Concrete Pavements," Research Report 90-D, Florida Department of Transportation, April 1973, 64 pp.

A study was initiated in 1960 to develop information on the validity of Florida's design standards for portland cement concrete highways, with particular reference to spacing of contraction joints and to aggregate interlock and the use of load transfer devices. Test sites were selected in three urban areas: Tampa, Pensacola, and Jacksonville. Joint movement, faulting, and load deflection measurements over a 4-year period are reported. Three of the original seven test sites were rechecked in 1972 to determine changes in the 8 years following the study. It was concluded that the design policy of the Department of Transportation with respect to load transfer at contraction joints is adequate. It was found that doweled joints show less faulting than nondoweled joints and that dowels increase the effective load transfer when properly designed and installed. Aggregate interlock provides 40 percent or more load transfer during warm weather, but frequently less than 40 percent during cold weather.

Tayabji, S. D., "Dowel Placement Tolerances," FHWA/RD-86/042, Construction Technologies Laboratory, May 1986, 36 pp.

An investigation was conducted to develop placement tolerances for dowels at concrete pavement transverse joints. It was found that a theoretical analysis of dowel misalignment was too complex. Laboratory testing of concrete slabs incorporating dowels with different levels of misalignment were then completed. They indicate that pull-out loads were relatively low for misalignment levels of less than 1 in per 18-in (25 mm per 457 mm) dowel, although insufficient data were collected to establish recommended levels.

Tayabji, S. D., and P. A. Okamoto, "Evaluation of Automatic Dowel Bar Inserter and Radar Method for Checking Dowel Bar Placement," Report to FHWA Demonstration Projects Division, HHO-41, Construction Technology Laboratories, October 1986, 29 pp.

This report presents the results of both a laboratory and field investigation conducted in Idaho to determine the effectiveness of a radar device in evaluating dowel bar misalignment and to evaluate the effectiveness of an

automatic dowel bar inserter to properly place dowel bars in rigid pavements. It was found that the radar device can be used to locate dowel bars, although the accuracy became better as the misalignment increased. Accuracy was dependent on the operator and equipment variability. Despite problems with the inserter, it was also concluded that this type of equipment can be used to place dowel bars within specified tolerances.

Vyce, J. M., "Performance of Load-Transfer Devices," Research Report 140, New York State Department of Transportation, July 1987, 29 pp.

The performance of several different types of load-transfer devices (LTD's) for transverse joints is reported in this study. Problems with the standard LTD's caused NY DOT to investigate plastic-coated dowels in the 1970's. They then considered epoxy-coated I-beams and a trial installation of fiberglass dowels. After 10 to 14 years of performance, these LTD's have shown no corrosion or other deterioration that would have caused loss of load transfer or abnormal pavement stress.

4. JOINTS AND JOINT SEALING

Armaghani, J. M., J. M. Lybas, M. Tia, and B. E. Ruth, "Concrete Pavement Joint Stiffness Evaluation," Transportation Research Record 1099, 1986, pp. 22-36.

This paper describes a procedure to determine the stiffness of an nondoweled joint in a concrete pavement using FEACONS III, a finite element analysis program, and the results from loading tests with a Falling Weight Deflectometer performed on a test pavement. The authors conclude that joint efficiency is an unrealistic measure of joint stiffness, due to its variability with changes in loading positions and thermal conditions.

Bryden, J. E. and R. A. Lorini, "Performance of Preformed Compression Sealers in Transverse Pavement Joints," Final Report on Research Project 57-1, Engineering Research and Development Bureau, New York State Department of Transportation, March 1980, 45 pp.

Preformed compression sealers, 1.25-in (31.8 mm) wide, were installed in 0.625-in (15.9 mm) wide transverse contraction joints in concrete pavement and observed for up to 10 years. Construction operations were monitored, and laboratory tests were conducted to determine sealer properties. Some fine material infiltrated below the sealer after 6 to 10 years of service, but most sealers generally performed well for that period. Many joints closed tighter than the 0.625 in (15.9 mm) minimum design width. Although the narrow joints resulted in more compression set in the sealers, they had less infiltration and spalling than the wider ones. For larger sizes of preformed sealers such as included in this study, compression to 40 percent of original width appears to improve joint sealing. Results of force-deflection and recovery tests performed on new sealer samples related to compression set occurring during field service, but not to infiltration. Force-deflection and recovery properties of the sealers generally decreased after field aging, but these decreases did not relate to sealer performance. Preconditioning sealer samples at elevated (212 $^{\circ}F$ [100 $^{\circ}C$]) or reduced (+14 $^{\circ}F$ and -20 $^{\circ}F$ [-10 $^{\circ}C$ and -29 $^{\circ}C$]) temperatures produced changes in force-deflection and recovery properties, but these changes did not relate to field performance or to changes in the same properties during field aging. A laboratory cycle test, which simulates joint infiltration, appeared to relate to field infiltration.

"Concrete Pavement Jointing and Sealing Methods," Pennsylvania Department of Transportation, Paper Prepared for the 54th Annual Meeting of the Transportation Research Board, 1975, 21 pp.

The experimental pavement consisted of combinations of various sealant materials (5), joint shapes (3) and slab lengths (3). A total of 1020 transverse joints were involved. The materials include conventional and "improved" rubberized asphalt, cold-poured, two-component polymers and preformed neoprene seals. Various sizes of step-cut joints were tried. These joints have a wider cut at the top to improve the width-to-depth ratio of the sealant. Shorter than normal slab lengths were also tested. Preliminary observations indicate that the improved rubberized asphalt sealant performs better than the conventional grade material. The cold-poured polymers were found to require careful mixing and handling to obtain satisfactory results. When properly placed, these materials also appear to provide good sealing qualities. The neoprene seals are also performing very well. Preliminary results indicate that the shorter slab lengths offer better joint seal performance and the 1/2 in wide by 3/4 in (13 x 19 mm) deep joint cut is more satisfactory than the method conventionally used.

Cook, J. P. and I. Minkarah, "Development of an Improved Contraction Joint for Portland Cement Concrete Pavements," Executive Summary Report, Research performed under agreement No. 2337, Ohio Department of Transportation in Cooperation with the FHWA, University of Cincinnati, August 1973, 22 pp.

This report describes an experimental project on Route 23 at Chillicothe. The detailed objective of this study is to isolate each of the variables which affect pavement movement and study its effect. The variables are joint configuration, joint spacing, subbase type, and dowel type. The study included 10 groups of approximately 10 joints each. At the end of this preliminary investigation, the following recommendations were made on the optimal pavement design: 21-ft (6.4 m) joint spacing, stabilized subbase, 0.25-in (6.4 mm) saw cut joint with beveled edge and chlorinated rubber base cure on the joint faces.

Cook, J. P., "A History of Joints in Concrete Pavements," <u>Journal of the Construction Division</u>, ASCE, March 1975, pp. 29-36.

This article traces the history of the design of pavement joints and indicates when changes were made in those factors that influence design.

DeYoung, C., "Spacing of Undoweled Joints in Plain Concrete Pavement," Highway Research Record, Number 112, January 1965, 9 pp.

In 1955 the Iowa State Highway Commission constructed approximately 16 mi (26 km) of experimental portland cement concrete pavement containing sections without contraction joints and sections in which the joints were sawed at intervals of 20, 50, or 80 ft (6.1, 15.2, or 24.4 m). None of the joints were doweled. Approximately half of the joints were not sealed. After 8 years of service the pavement test sections have an average slab length ranging from 19 to 37 ft (5.8 to 11.3 m). Faulting of at least 0.0625-in (1.6 mm) is evident at 70 percent of the joints spaced at 20 ft (6.1 m). The incidence of faulting is greater for the joints spaced at 50 and 80 ft (15.2 and 24.4 m). Measurements made in April, 1964, revealed that more than 92 percent of the joints were open at least 1/16-in (1.6 mm) more than their original sawed width of 0.125 in (3.2 mm). The effect of joint sealing was not conclusive, but it appears that the seal may have been useful in keeping debris out of the joints spaced at 20 ft (6.1 m). The study concludes that to control transverse cracking, joint spacing of 19 to 37 ft (5.8 to 11.3 m) may be adequate; aggregate interlock is not maintained by pavements with joint spacing as short as 20 ft (6.1 m); the incidence of faulting is the least for the 20-ft (6.1 m) spacing.

Hubrecht, L., "The Namur Test Road: Behaviour of Various Types of Contraction Joints After Five Years of Traffic," Proceedings, Third International Conference on Concrete Pavement Design, Purdue University, April 1985, pp. 625-632.

Observations and measurements of six types of contraction joints were made on a test road consisting of an 8- to 9-in (203 to 229 mm) concrete overlay of an asphalt concrete pavement. In 1984, after 5 years of service, it was found that there was a difference in deflection between the doweled and nondoweled joints, although it was not of great significance. No difference was found between the sealed and nonsealed joints, although it was noted that this pavement is very well drained. The influence on this study of the construction of a concrete overlay on a still sound asphalt concrete pavement could not be determined.

Jones, G. M., D. E. Peterson, and R. K. Vyas, "Evaluation of Preformed Elastomeric Joint Sealing Systems and Practices," Final Report, Project 4-9, Prepared for Highway Research Board, NCHRP, Utah State Highway Department, June 1971, 101 pp.

Preformed elastomeric joint sealers made available by producers were studied in the laboratory and in the field to determine the properties and performance criteria that would improve their performance. A field test was performed at two different locations using a variety of sealants, joint shapes, and joint spacings.

Loza, G. A. and D. I. Anderson "Evaluation of Concrete Joint Sealants: Clear Creek Summit to Belknap Interchange ID-70-1(31)7," Report No. UDOT-MR-88-3, Utah Department of Transportation, June 1988, 20 pp.

Seven sealants, including two silicones, three hot-pour materials, and one PVC-Coal Tar, were placed on a stretch of pavement in 1984. The materials were placed in 0.374-in (9.5 mm) wide joints, on slab lengths varying from 12 to 18 ft (3.7 to 5.5 m). After 3 years of performance, it was found that the silicone sealants exhibited good performance with respect to adhesion and cohesion and rejection of incompressibles. However, there was a high occurrence of concrete failures near the joints. The hot-pour sealants performed poorly, showing high degrees of failure in cohesion, adhesion, and the presence of incompressibles. The PVC-Coal Tar performed poorly, with extensive adhesion failures and moderate to extensive levels of incompressibles in the sealant.

McKenzie, L. J., R. J. Little, and P. G. Dierstein, "Behavior of Contraction Joints in the Rehabilitated AASHO Test Road," Report No. FHWA-IL-PR-75, Illinois Department of Transportation, March 1977, 79 pp.

Rehabilitating the AASHO Test Road pavement provided an opportunity to compare the behavior of doweled contraction joints spaced at 15-ft (4.6 m) intervals in nonreinforced pavement overlying a granular subbase with those sawed at 40- and 100-ft (12.2 and 30.5 m) intervals in reinforced pavements on both granular and stabilized subbases. Faulting decreased as joint interval decreased and as pavement thickness increased. Faulting was reduced where the subbase was stabilized. The cumulative amount of faulting per pavement mile was largest for 15-ft (4.6 m) panels even though they had the least fault per joint. This fact partly accounts for pavements with 40-ft (12.2 m) joints being smoother than those with 15-ft (4.6 m) joints. The amount of spalling per mile of pavement increased as the joint interval decreased, although the number of major spalls per joint tended to increase as joint interval and joint opening increased. Transverse cracking between the joints increased as joint interval increased, but it was reduced over a stabilized subbase. The amount of D-cracking per mile of pavement increased as the number of joints and cracks increased and as the pavement aged. The best overall pavement behavior and the lowest Roughness Index were associated with pavements that had the fewest joints, particularly on a Bituminous Aggregate Material (BAM) subbase.

Shober, S. F., "Are Pavement Joint Sealants Always Necessary," <u>Concrete Construction</u>, March 1987, pp. 289-297.

A 9-in (229 mm) thick concrete test pavement was built over an 8-in (203 mm) thick crushed gravel base and a well-drained subgrade. The pavement was reinforced with mesh, joints were doweled and shoulders were not paved. The highway was subjected to 7000 vehicles a day, 30 in (762 mm) of rainfall a year, and ambient temperatures ranging from -40 ^{o}F to 100 ^{o}F (-40 $_{o}$C to 38 $_{o}$ C). Four joint spacings (20, 40, 60, and 80 ft [6.1, 12.2, 18.3, and 24.4 m) and 5 joint sealants were tested in sections about 1000 ft (305 mm) long. The conclusions from the report are: (1) the pavements with nonsealed joints performed better than the pavements with sealed joints, (2) the pavements with shorter joint spacings performed better than the pavements with longer joint spacing, (3) some sealants can keep joints effectively sealed for ten years, if the joints are designed properly, and, (4) joint sealing costs are not justified.

Shober, S. F., "Portland Cement Concrete Pavement Performance as Influenced by Sealed and Unsealed Contraction Joints," Wisconsin Department of Transportation, 24 pp.

In 1974, a carefully designed joint and sealant study began with the objectives of evaluating the effect of joint spacing and joint sealing/nonsealing on total pavement performance and to evaluate joint sealants. This study was conducted on a new 9-in (229 mm) jointed reinforced concrete pavement on a well-drained subgrade, employed five joint sealants, and considered joint spacings of 20, 40, 60, and 80 ft (6.1, 12.2, 18.3, and 24.4 m). A total of 22 test sections were evaluated, including 8 control sections in which the joints were not sealed and 14 test sections in which the joints were kept sealed. Based upon 10 years of monitoring, the following conclusions were suggested: (1) some sealants served well for ten years, (2) short joint spacings gave the best pavement performance, and (3) pavement with unsealed joints had better performance than the pavement with sealed joints. It was also suggested that joint sealing appears not to be beneficial for doweled contraction joints.

Tayabji, S. D., "Optimized Performance at Concrete Pavement Joints," Proceedings, <u>Third International Conference on Concrete Pavement Design</u>, Purdue University, April 1985, pp. 595-603.

The results of an investigation conducted to develop improved performance of concrete pavement joints are presented in this paper. The investigation included development of a finite element computer program, laboratory testing, and evaluation of techniques to improve structural response at joints. The computer program, JSLAB, can analyze a large number of jointed concrete slabs consisting of a one- or two-layer concrete pavement system resting on a Winkler (spring) foundation. JSLAB was used to evaluate the effect of fewer nonuniformly-spaced dowel bars, tied concrete shoulders, and widened lanes on joint performance. Theoretical analysis indicates that deflections and stresses at a transverse joint with 6 or 7 nonuniformly-spaced dowels are similar to those for a conventional joint with 12 uniformly spaced dowels. It was also concluded that solid round dowel bars are the most cost-effective mechanical load transfer device for new construction.

Thornton, J. B., "Rigid Pavement Joints and Sealants Study," Georgia Highway Department Research Project No. 6903, Final Report, September 1975, 144 pp.

This publication is a final report on a study initiated to evaluate the relative performance of a number of different types of rigid pavement joint sealants. The project has been designed to provide information on the effect that several joint variables might have on the performance of the joint sealant design. These design variables include two joint spacings, two joint orientations, and eight joint sealants.

5. CONCRETE SHOULDERS

Arnold, C. J., and M. A. Chiunti, "Experimental Concrete and Bituminous Shoulder Construction Report (Experimental Work Plan No. 4)," Research Report No. R-844, Michigan State Highway Commission, January 1973, 12 pp.

This report covers construction of experimental shoulders on Interstate 69 in Michigan to compare certain design improvements using both concrete and bituminous materials. The purpose of the study is to evaluate the cost and performance of experimental concrete and bituminous shoulders in comparison with the standard bituminous shoulder for Michigan Interstate freeway construction.

Chiunti, M. A., "Experimental Concrete and Bituminous Shoulders (Progress Report)," Research Report No. R-898, Michigan State Highway and Transportation Commission, February 1974, 11 pp.

This progress report covers the findings to date of the evaluation of experimental concrete and bituminous shoulders. It was found that after 1 year the standard shoulders had developed cracking along the edges, that the modified bituminous shoulders showed no visible deterioration, and that there were no performance-related failures of the concrete shoulder sections.

Colley, B. E., C. G. Ball, and P. Arriyavat, "Evaluation of Concrete Pavements with Tied Shoulders or Widened Lanes," Transportation Research Record Number 666, pp. 39-45.

This report describes an experimental project, consisting of field and lab test sections. The field sections are located on T.H. 14 and T.H. 90, Minnesota. They were instrumented to measure strains and deflections and the results were compared to the lab results and to the theoretical strains and deflections as predicted by Westergaard theory. Variables on the pavements in the field included: lane width, pavement thickness, joint spacing, and dowel placement.

Keller, E. J., "Evaluation of Portland Cement Concrete Shoulders in Georgia," GDOT Research Project No. 8004, Georgia Department of Transportation, April 1988, 66 pp.

Georgia has been constructing doweled PCC pavements with tied PCC shoulders for 11 years and it is now their standard design. The decision to follow this design is based on the desire to reduce pavement faulting and slab cracking, improve load transfer between the mainline pavement and the shoulder, and provide a reservoir for a longitudinal joint sealant between the mainline pavement and the shoulder. This study evaluates the projects constructed to date in Georgia and compares their performance to doweled and nondoweled PCC pavements with AC shoulders. It was concluded that the use of tied PCC shoulders and dowels in a pavement with either an AC-stabilized of lean concrete base produced the lowest pavement faulting.

Korfhage, G. R., "Effect of Concrete Shoulders, Lane Widening and Frozen Subgrade on Concrete Pavement Performance," Draft Final Report, Investigation No. 209, Minnesota Department of Transportation, December 1987, 14 pp.

This is the fourth and final report issued as part of a project to determine the effect of concrete shoulders, lane widening and frozen subgrade on the performance of concrete pavements. Data were collected from five locations in southeastern Minnesota. The results presented include the following observations: loads applied while the subgrade is frozen have little effect on the pavement; pavement curl can significantly affect deflection; the reduction in deflection from the addition of tied shoulders was difficult to determine; and pavement deflections can be significantly reduced by constructing pavement lanes wider than 12 ft (3.7 m) and placing an edge line 12 ft (3.7 m) from the centerline.

Lokken, E. C., "What We Have Learned to Date from Experimental Concrete Shoulder Projects," Highway Research Record Number 434, 1973, pp. 43-53.

Concrete shoulders have been in use on urban expressways for years in some areas. Several experimental projects, most of them initiated under the FHWA National Experimental and Evaluation Program, have been built in recent years, and more are under construction or planned. The paper summarizes performance to date, examines design details developed from experimental projects and makes recommendations for design for maximum safety and economy. The paper suggests that further study should be given to (1) the effects on temperature and moisture conditions within the pavement system as the result of a light-colored shoulder surface, and (2) the effect on structural capacity of a concrete roadway pavement when a tied and keyed concrete shoulder is added.

"Portland Cement Concrete Shoulders," Research and Development Report No. 27, Illinois Division of Highways, July 1970.

The first section of experimental portland cement concrete shoulders in Illinois was built in 1965. A second section was built in 1966 and a third in 1967. These shoulders, all of which were constructed of full-strength plain concrete without reinforcement, have been placed adjacent to conventionally reinforced pavement, continuously reinforced pavement, and a bituminous concrete overlay systems. Other principal variables are the presence and absence of tiebars, the presence and absence of granular subbase, the spacing of transverse joints, and warning rumble strip treatments. Experience to date is considered to indicate that when various findings of the project are applied, the economic and long-term behavior of Portland cement concrete shoulders can be viewed with optimism.

Ray, G. K., "Concrete Shoulders and Lane Widening—Structural Benefits and Improved Pavement Performance," Paper Prepared for Presentation at the 1985 Missouri/Kansas PCC Paving Workshop, Kansas City, Missouri, November 5, 1985.

This paper details standard practice in design, construction and benefits of concrete shoulders and widened lanes. It provides brief details of construction, performance and a summary from existing pavements. It suggests that the main benefit of concrete shoulders and widened lanes is the reduction of mainline pavement thickness due to reduction in stress at the pavement edge.

Sawan, J. S., M. I. Darter, and B. J. Dempsey, "Structural Analysis and Design of PCC Shoulders," Department of Civil Engineering, University of Illinois, March 1980.

A structural evaluation of concrete shoulders has been conducted and a comprehensive design procedure for plain jointed concrete shoulders developed. The procedure can be used to provide concrete shoulders either for rehabilitation of existing pavement or for new pavement construction. All major factors that are known to affect the behavior of concrete shoulders are considered in the mechanistic design approach: encroaching moving trucks, parked trucks, foundation support, longitudinal joint load transfer, shoulder slab thickness and tapering, width of shoulder, and traffic lane slab. The finite element structural analysis technique was used along with a concrete fatigue damage model to sum damage for both moving encroaching trucks and for parked trucks. A relationship was established between the accumulated fatigue damage and slab cracking. Thus, the shoulder can be designed for an allowable amount of cracking which can vary depending on the performance level desired. Procedures for tieing the concrete shoulder to the mainline slab are recommended to provide adequate load transfer and to avoid joint spalling. Long-term low maintenance performance of the concrete shoulder, along with significant improvement in performance of the traffic lane, can be obtained both for new construction and rehabilitation purposes if the shoulder is designed properly.

Slavis, C., "Portland Cement Concrete Shoulder Performance in the United States (1965-1980)," Proceedings, <u>Second International Conference on Concrete Pavement Design</u>, Purdue University, April 1981, pp. 331-341.

During the 1960's and early 1970's, several states constructed concrete shoulders to evaluate their performance and develop design standards. These projects include the Illinois projects of 1965, 1966, 1967 and 1971, the Kentucky project of 1972, and the 1971 projects in Iowa, Michigan, Nebraska, Pennsylvania, and Texas. Variables included in these evaluation studies included thickness, pavement type, joint spacing, tie bars, and subbase. In 1973, a report was prepared and presented to the Highway Research Board. Major conclusions of the report centered around design requirements and construction. At the time of this early study the concrete shoulders ranged in age from 0 to 7 years, with only three installations being over 2 years old. These shoulders are now a minimum of 7 years old, and the original Illinois installations have been in service for up to 15 years. The purpose of this paper is to report on the inspection and reevaluation of these concrete shoulders. A comparison of both the conclusions of the 1973 report and an evaluation of current design practices based on 15 years of experience has been made. Recommended modifications to current design practices indicated by the current study findings are included. The major recommendation is that the shoulder have the same thickness as the pavement.

Slavis, C. V., and C. G. Ball, "Verification of the Structural Benefits of Concrete Shoulders by Field Measurement," Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1985, pp. 267-274.

A field study was conducted to evaluate the effect of concrete shoulders on concrete pavement performance. Pavement deflections and strains were measured along tied shoulder joints and along free edges at two project locations. In addition, a theoretical analysis was performed to determine the effect of tied shoulders on concrete pavement response. Study results indicate that pavement response is improved for pavements using a tied shoulder as compared to pavements not using a tied shoulder. It is concluded that for application to the AASHTO thickness design procedure, only one-half of the design 18-kip (80 kN) ESAL applications need to be considered for concrete pavements incorporating a tied concrete shoulder, resulting in a reduction of one inch in the required mainline slab thickness. In theory, the benefits of shoulders should apply equally to add-on shoulders to extend existing pavement life.

Tayabji, S. D., C. G. Ball, and P. A. Okamoto, "Effect of Concrete Shoulders on Concrete Pavement Performance," Transportation Research Record Number 954.

This is a study of the three sections on I-90 between Albert Lea and Fairmont, Minnesota. The variables include tied concrete shoulders, widened lanes, 8- and 9-in (203 and 229 mm) slabs, JRCP and JPCP, and random and uniform joint spacing. The objectives of this study were to measure the load-induced strains and deflections in those pavement sections incorporating tied concrete shoulders and to determine the effects of concrete shoulders on concrete pavement performance.

6. PAVEMENT DESIGN

Bordonado, G., G. Colombier, D. Ponchon, and F. Verhee, "Recent Developments in French Concrete Pavement Construction and Behavioral Assessment," Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1985, pp. 237-244.

Concrete pavement structural design and construction techniques have undergone significant and sometimes innovative developments since 1975. Conventional structures with nondoweled slabs over a lean concrete subbase have demonstrated their good performance. The structure consisting of thick slabs placed over an untreated porous subbase does not entail any application problems; its performance has formed the subject of many investigations. The gritting of fresh concrete combined with partial surface baring on projects in 1984 may be regarded as an operational technique. Concrete compacting is carried out effectively. Completed pavements are checked regularly to evaluate their performance.

Concrete Roads: Practical Guide for Technology Transfer," Permanent International Association of Road Congresses (PIARC), May 1987, 115 pp.

This document includes information to be used by countries wishing to determine the suitability of constructing concrete pavements. It summarizes current knowledge from the countries where concrete pavement technology is used, and includes sections on: the essentials of concrete pavement technology; the structural and thickness design and the sensitivity of various elements of the technique; and the material characteristics and factors affecting construction. The use of cement-treated layers is not considered.

Eisenmann, J., "Features of Old and New Concrete Pavement Structures," 1981.

This article describes current construction practices for concrete pavements in West Germany, and shows how they evolve from previous practice. The basic features are widened lanes, dowels in the wheel paths, 5 m (16.4 ft) joint spacing, tied shoulders, and cement-treated bases.

"Guide for Design and Construction of Concrete Parking Lots," reported by ACI Committee 330, <u>ACI Materials Journal</u>, November-December 1987, pp. 532-558.

Information on site investigation, thickness determination, design of joints and other details, paving operations, and quality assurance procedures during construction are discussed, as well as maintenance and repair. There are many similarities between the design and construction of concrete slabs for parking lots and for highways and streets. However, there are also differences, and it is suggested that taking into account these differences can result in the construction of an economical and serviceable structure.

Iwama, S. and T. Fukuda, "Design Method and Researches of Concrete Pavements in Japan," submitted to the International Workshop on the Theoretical Design of Concrete Pavements, June 6-7, 1986, The Netherlands, 19 pp.

The current methods used for concrete pavement design are summarized. These include the recognition of load stresses and the lateral placement of loads on a pavement. Thermal stresses due to curling are also considered. Revisions to this design method consider the effect of cement stabilized bases and differences in the lateral distribution of traffic loadings from the original calculations. New flexural fatigue strength equations for concrete are presented based on recent laboratory work. It is recommended that further work be done to incorporate the effects of erosion.

Johnson, T. C., R. L. Berg, E. J. Chamberlain, and D. M. Cole, "Frost Action Predictive Techniques for Roads and Airfields: A Comprehensive Survey of Research Findings," DOT/FAA/PM-85/23 (CRREL Report 86-18), U.S. DOT, Federal Aviation Administration, December 1986, 52 pp.

This document present 6 years of research on the prediction of frost-susceptible soils. The four principal study areas covered are: selection and validation of the most effective laboratory index tests to indicate the susceptibility of soils to detrimental frost action; the development of a device for nondestructive monitoring of changes in soil moisture content and density during freezing and thawing; the improvement and validation of a mathematical model of frost heave that was developed earlier; the development of laboratory test methods for characterizing seasonal changes in the resilient modulus of a wide variety of granular soils and validation of those methods by nondestructive testing.

Lamprecht, H., and A. Vollpracht, "Synoptic Table on Standards and Practices for Concrete Roads in Europe," International Symposium on Concrete Roads, Aachen, June 2-4, 1986, 20 pp.

Standard practices regarding pavement design and construction are summarized for fifteen European countries. Criteria for joints, bases and subbases, drainage, paving, finishing, and curing are presented.

Lister, N. W., and J. Porter, "Philosophy and Experience of Full-Scale Pavement Testing at the Transport and Road Research Laboratory," TRRL, 18 pp.

The basic method of pavement design recommended by TRRL is described. It combines laboratory testing of materials to provide inputs to mathematical models of pavement behavior, and full-scale pavement testing under controlled conditions.

Mayhew, H. C., and H. M. Harding, "Thickness Design of Concrete Roads," Research Report 87, Transport and Road Research Laboratory, 1987, 13 pp.

At the time of this report, design curves in use were based on observations of experimental roads prior to 1970. By 1985 these pavements had undergone considerably more loading. The performance of 29 nonreinforced and 42 reinforced pavements were analyzed to establish new design curves. The structural performance was assessed and pavement life equations were derived by multiple regression analysis. Significant factors affecting pavement life were slab thickness, concrete strength, and foundation support. In reinforced pavements, the amount of steel was also important. It was also found that a unique relationship existed between design life and the ratio of concrete strength to load and thermal stresses, based on multilayer analysis.

Ray, M., "The Design of Concrete Pavements: Existing Choices in France," Bulletin de Liaison des Laboratoires des Ponts et Chaussees, No. 91, September-October, 1977, pp. 81-124.

In France, the pavement design used until 1975 was short slabs, without dowels, resting on a hydraulic binder-treated subbase. Certain irregularities (pumping, faulting) on fairly recent sections induced highway officials to carry out in-depth action: creation of a work group, major research programs, fact-finding trips abroad and experimental road sections. This article describes the primary conclusions of those investigations, as well as the procedure followed during the analysis. It was found that under conditions prevailing in France, the technique without dowels could be regarded as a satisfactory technical and economic compromise at the present time. However, as the old road structures exhibited inadequate drainage and a certain sensitivity to subbase erosion, porous concrete in the shoulder and vibrated lean concrete in the subbase was used. Finally, independently of the conventional structures, the "thick slab" structure resting directly on a draining or treated subgrade was found to be an interesting new approach.

Sigeru, I., "Experimental Studies on the Structural Design of Concrete Pavement," Pavement Laboratory, Public Works Research Institute, Ministry of Construction, Japan, May 1964, 108 pp.

The results of studies on concrete pavements from 1955 to 1962 have resulted in the proposal of several revisions in the approach to concrete pavement design. Included in these studies were the calculation of longitudinal edge stresses taking into account actual loading patterns combined with variations in thermal stress. New fatigue data for concrete was generated and should be used to design the thickness of concrete pavement. It is suggested that a reinforced longitudinal edge will increase the life of concrete pavement slabs.

"Thickness Design for Concrete Highway and Street Pavements," Portland Cement Association, 47 pp.

This document discusses thickness design for JPCP, JRCP, and CRCP. Special factors which are considered include: the degree of load transfer at the transverse joints; the effect of using a concrete shoulder; the effect of using a lean concrete base; the influences of fatigue and erosion; and the impact of tridem axles on thickness design, especially from the standpoint of erosion.

Van Breemen, W., "Current Design of Concrete Pavement in New Jersey," Highway Research Board, Proceedings of the 28th Annual Meeting, December 1948, pp. 77-91.

This paper discusses the concrete pavement design for heavy trucks in New Jersey, current at the time of publication. It includes a discussion of pavement thickness, slab length and width, reinforcing steel, the doweling of expansion joints, dowel coatings, joint filler, and base construction. Information is given in regard to the performance of certain expansion joints employed in the past. Also discussed are the effects of corrosion on dowels and the theory behind the inclusion of expansion joints. Traffic data, the results of load transfer tests on various sizes, shapes, and lengths of dowels, and the general behavior of contraction joint pavements are included.

Wimsatt, A. J., B. F. McCullough, and N. H. Burns, "Methods of Analyzing and Factors Influencing Frictional Effects of Subbases," Preliminary Review Copy, Research Report 459-2F, Texas State Department of Highways and Public Transportation, November 1987, 158 pp.

Several items are reported in this document. It includes: a review of available information relating to the subbase frictional effect; experimental results of push-off tests on an unbound shell subbase layer underlying an inservice jointed reinforced concrete pavement in Houston, Texas; results of push-off tests to find the effects of subbase depth and surface texture on the frictional resistance of an asphalt concrete pavement; results of correlating actual crack spacing values for continuously reinforced concrete pavements to values predicted by the CRCP computer program using the subbase friction information found from this study; results of estimating subbase friction using the indirect tensile strength testing of subbase cores; and implications of the subbase frictional effect on concrete pavements.

7. PAVEMENT ANALYSIS

Eisenmann, J., "Westergaard's Theory for Calculation of Traffic Stresses." 11 pp.

This paper presents a calculation method for the k-value and the ending stresses of a three layer system using Westergaard's formulas and Odemark's theory of equivalence. In this case, the effect of friction between the concrete slab and cement-treated base course is of interest since its use (in combination with doweled joints) is extensive in the Federal Republic of Germany.

Fukuda, T., "Mechanism of Stress Distribution in Concrete Pavements," <u>Proceedings of JSCE</u>, No. 242, October 1975, pp. 63-72 (in Japanese).

Fukuda, T., "Influence of Load Contact Patterns on Stress Distribution in Concrete Pavements," The Technology Reports of the Tohoku University, Vol. 42, No. 1, June 1977, pp. 215-227.

Fukuda, T., M. Koyanagawa, and S. Murai, "Condition Survey of Concrete Pavements and its Evaluation," Proceedings, <u>Third International Conference on Concrete Pavement Design</u>, Purdue University, April 1985, pp. 519-523.

Concrete pavements were surveyed in the Tohoku region of Japan and the cracking patterns were studied using multivariate analysis. It was found that slab width and the volume of heavy truck traffic had a large effect on the cracking pattern.

Gregory, J. M., "Analysis of Data from a Survey of Unreinforced Concrete Roads Built 1970-1979," Materials Memorandum No. 133, Working Paper, Transport and Road Research Laboratory, 1986, 50 pp.

Data were collected during the period from 1981 to 1983 on 36 nonreinforced concrete roads constructed between 1970 and 1979. The extensive data includes design, material, and construction inputs known to affect the structural and other performance indicators of concrete pavements. Environmental conditions during and after paving were impossible to acquire. A total of 19 conclusions are presented from the results of the analysis of variance.

Kraemer, C., "An Overview of the European Practice with Concrete Pavements," Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1985, pp. 15-21.

The scope of this paper is to give a general picture of European practice with concrete pavements, particularly regarding the design, performance, and rehabilitation of highway pavements. The main sources used are the National Reports submitted to the 17th PIARC World Road Congress held in Sydney, Australia, in October, 1983; the conclusions of the Congress; and the Report of the PIARC Technical Committee on Concrete Roads, which deals with a series of subjects, handled on an international basis. Emphasis is placed on the latest trends in the field. A wide range of subjects is considered within the objectives of the Conference.

Murai, S. and T. Fukuda, "Stress Computation of Multi-Layered Pavement Structures," The Technology Reports of the Tohoku University, Vol. 49, No. 2, December 1984, pp. 163-173.

Ray, M., "A European Synthesis on Drainage, Subbase Erodibility, and Load Transfer in Concrete Pavements," Proceedings, Second International Conference on Concrete Pavement Design, Purdue University, April 1981, pp. 27-39.

European research and field practice from 1977 to 1981 are described as concerns load transfer (at contraction joints of nonreinforced short slab pavements), drainage of slab-subbase contact, and subbase and shoulder erodibility. For nondoweled pavements, new developments leading to better load-transfer efficiency concern mainly the reduction in slab length, the extra slab width beyond shoulder line, the reduction in hydraulic and thermal shrinkage, and the increase in coarse aggregate diameter. For doweled pavements, new developments concern the reduction in the number of dowels, anticorrosion and slipping devices, placement accuracy and vibration placement equipment. Water problems encountered in Europe for pavements with and without dowels are presented; the new draining cross sections in each country are described together with measurements characterizing their efficiency. Recent theoretical and practical research results on pumping hydrodynamics and subbase erodibility are presented.

Tia, M., J. M. Armaghani, et. al., "Feacons III Computer Program for an Analysis of Jointed Concrete Pavements," Paper submitted for consideration for publication in Transportation Research Record, 1987, 35 pp.

This paper describes the computer program FEACONS III (Finite Element Analysis of CONcrete Slabs). It was developed to provide a suitable analytical model to analyze the behavior of concrete pavements effectively and realistically. The program has been used extensively in the analysis of existing concrete pavements and a test road in Florida. The analytical model and computational procedure used by the program are described and the analytical results from the program are compared to the actual measured results for a few cases. The program was shown to be both versatile and effective in the analysis of concrete pavement performance.

Ytterberg, R. F., "Shrinkage and Curling of Slabs on Grade, Part II—Warping and Curling" Concrete International, May 1987, pp. 54-61.

The author discusses the occurrence of warping and curling stresses in slabs on grade. Various historical approaches are discussed and special attention is given to the calculation of critical slab length and the differences between modern approaches and Westergaard's research. While most of the research is related to enclosed industrial floor slabs on grade, the author feels that it also relates to pavement slabs on grade. It is suggested that longer joint spacing reduces the number of curled or warped slab ends and that increased slab thickness at the slab ends (based on cantilever design) may reduce cracking of these slabs. It is also noted that several slab on grade design methods (WRI, COE, PCA) allow for reduced thickness of slabs as the subgrade modulus increases, but that the stresses increase in the slab and cause increased cracking.

8. LIFE CYCLE COSTS

Jung, F. W., "Modeling of Life-Cycle Costs of Pavement Rehabilitation," Transportation Research Record, Number 1060, 1986, pp 1-8.

The rehabilitation of pavements can be regarded as a cycle of gradual depreciation and subsequent replacement of the capital invested in the pavement layers of roads. A form for the life cycle length function is derived and discussed. Ontario 1984 costs are used to give an example of annualized costs. An approach to sidestepping the issue of salvage value is described.

Vyce, J. M., "A Life-Cycle Cost Analysis for Asphalt and Concrete Pavements," Special Report 82, New York State Department of Transportation, February 1985, 40 pp.

Data were reviewed on pavement life, the timing and extent of both routine and contract maintenance, and the costs incurred in these operations to complete a comprehensive life-cycle cost analysis for asphalt and concrete pavements. The information is to be used to determine whether asphalt or concrete should be used in new construction.

9. WEIGH-IN-MOTION

Chira-Chavala, T., D. A. Maxwell, and H. S. Nassiri, "Weigh-in-Motion Sampling Plan for Truck-Weight Data in Texas: Method and Plan Development," Transportation Research Record Number 1060, 1986, pp. 105-111.

This paper describes a plan for Texas to collect truck weight data using WIM equipment. The methodology used is documented along with the actual development of the plan, which is based on a probability sampling framework aimed at capturing maximum variability of truck weights and types within the state. It is suggested that this method can be applied to any State, region, or district with minimal or no modification.

Cunagin, W. D., "Use of Weigh-in-Motion Systems for Data Collection and Enforcement," NCHRP Synthesis 124, Transportation Research Board, National Research Council, September 1986, 34 pp.

The development of Weigh-in-Motion (WIM) equipment, WIM data needs and uses, WIM data requirements, and current programs and research relating to WIM are discussed. Information on currently available WIM equipment and the States' experiences with that equipment are also presented.

Cunagin, W. D., "Recommended Traffic Data Collection Procedures for the Strategic Highway Research Program Long-Term Pavement Performance Studies," Draft Report Prepared for SHRP, March 1988, 33 pp.

The importance of reliable and accurate traffic data to the success of the Strategic Highway Research Program (SHRP) studies is a primary concern. This paper discusses the most efficient means of obtaining past traffic information and also recommends ways to monitor the traffic on the test sections in the future. The role that Weigh-in-Motion (WIM) will play in the studies is also discussed.

Cunagin, W. D., A. B. Grubbs, Jr., and N. A. Ayoub, "Portable Sensors and Equipment for Traffic Data Collection," Transportation Research Record Number 1060, 1986, pp. 127-139.

This study evaluates portable traffic sensor alternatives with respect to their technical effectiveness, the probability of user acceptance, and their cost. The types and amounts of traffic data normally collected were used to determine performance requirements for portable sensors and data collection equipment.

Huft, D. L., "The South Dakota Bridge Weigh-in-Motion System," Transportation Research Record Number 1060, 1986, pp. 111-120.

The development of the South Dakota Department of Transportation's system to collect truck weight information using WIM data is described. South Dakota acquired the equipment, developed software, and permanently instrumented two bridges in 1983. As of 1985, eighteen bridge WIM sites were being used to conduct the State's Truck Weight Study on interstate, main rural, secondary, and urban highways.

"Second National Weigh-in-Motion Conference, Vols. I and II," Georgia Department of Transportation, Atlanta, Georgia, May 20-24, 1985.

The proceedings from this conference include information on the design applications for WIM, Systems Applications, State experiences with WIM, and an update on the current status of WIM research projects.