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Rehabilitation of Concrete Pavements

Volume I: Repair Rehabilitation Techniques

Research, Development, and Technology
Turner-Fairbank Highway Research Center
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FOREWORD

This report is one volume of a four-volume set presenting the results of a research study to develop improved evaluation procedures and rehabilitation techniques for concrete pavements. Each report includes the Table of Contents for all four volumes. Eight rehabilitation techniques were selected for detailed investigation by field inspection and analytical study. These eight techniques are diamond grinding, load transfer restoration, edge support, full-depth repair, partial-depth repair, bonded concrete overlays, unbonded concrete overlays, and crack-and-seat with AC overlay. Based on analysis of the field data, a series of distress models were developed to predict the performance of the various rehabilitation techniques under a variety of conditions. These models and other information were then used to develop a comprehensive prototype system for jointed plain, jointed reinforced, and continuously reinforced pavement evaluation and rehabilitation.

This report will be of interest to engineers involved in planning, designing, or performing rehabilitation of 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.

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and Highway Operations
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NOTICE

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16. Abstract

Extensive field, laboratory and analytical studies were conducted into the evaluation and rehabilitation of concrete pavements. Field studies included over 350 rehabilitated pavement sections throughout the U.S., and the construction of two field experiments. A laboratory study was conducted on anchoring dowels in full-depth repairs. Analyses of field and laboratory data identified performance characteristics, improved design and construction procedures, and provided deterioration models for rehabilitated pavements. A concrete pavement advisory system was developed to assist engineers in project level evaluation and rehabilitation.

The repair techniques in Volume I include full-depth repair, partial-depth repair, load transfer restoration, edge support and diamond grinding.

This volume is the first in a series. The others in the series are:

FHWA NO.	Vol. No.	Title
RD-88-072	II	Overlay Rehabilitation Techniques
RD-88-073	III	Conc. Pvt. Eval. and Rehab. System
RD-88-074	IV	Appendixes

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

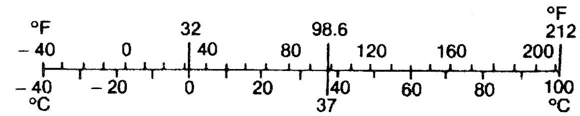
Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
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.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
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		
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		
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		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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CHAPTER 1

INTRODUCTION

1.0 STUDY OBJECTIVE

The overall objective of this study was to develop improved evaluation procedures and rehabilitation techniques for concrete pavements. This objective was accomplished through extensive field, laboratory and analytical studies that have provided new knowledge and understanding of the performance of rehabilitated concrete pavements. New and unique evaluation and rehabilitation procedures and techniques were developed that will be very useful to practicing pavement engineers.

This final report, presented in four volumes, documents all of the results developed under the contract, "Determination of Rehabilitation Methods For Rigid Pavements", conducted for the Federal Highway Administration (FHWA). This volume documents the results of the study on Repair Rehabilitation Techniques.

1.1 FIELD STUDIES

The field studies involved a large and extensive field survey of 361 rehabilitation sections of jointed plain and reinforced concrete pavement. These sections were located in 24 States as shown in figure 1 and table 1. Eight rehabilitation techniques were selected for detailed study:

- Diamond grinding.
- Load transfer restoration.
- Edge support.
- Full-depth repair.
- Partial-depth repair.
- Bonded concrete overlays.
- Unbonded concrete overlays.
- Crack and seat and AC overlay.

The extent of the pavement surveys is more fully summarized in table 2, which shows the number of database records and the contents of each record for each of the rehabilitation techniques. Considering full-depth repairs for example, there were 96 different projects located in 22 States, these consisted of 233 different repair designs, for a total of 2001 actual full-depth repairs surveyed.

There were five basic data types that were deemed necessary for the development of performance prediction models and the development and improvement of design and construction procedures. These include:

- Field condition data.
- Original pavement structural design, in situ conditions, and historical improvement data.
- Rehabilitation design data.
- Historical traffic volumes, vehicle classifications and accumulated 18-kip [80 kN] equivalent single-axle loadings.
- Environmental data.

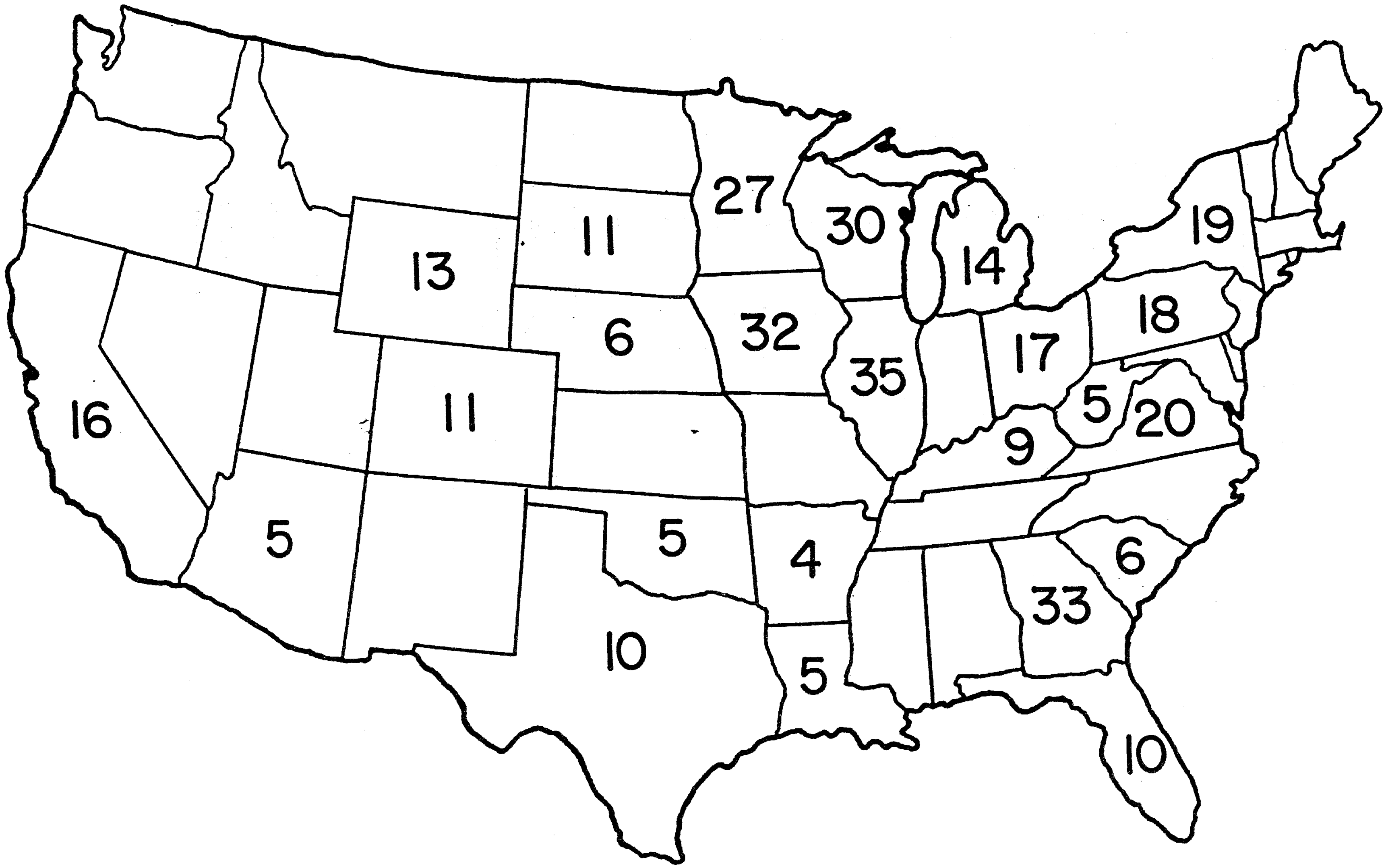

Figure 1. General location of all rehabilitation projects surveyed.

Table 1. Breakdown of rehabilitation techniques by State.

STATE	FDR	PDR	DGD	LTR	CAS	UNBOL	BOL	ES	TOTAL
Arizona	1	1	3	0	0	0	0	0	5
Arkansas	1	0	1	0	1	0	0	1	4
California	3	0	7	0	6	0	0	0	16
Colorado	2	0	1	1	2	3	0	2	11
Florida	3	0	2	0	5	0	0	0	10
Georgia	5	6	16	2	0	2	0	0	31
Illinois	11	1	6	2	12	2	0	1	35
Iowa	5	0	2	0	0	0	25	0	32
Kentucky	0	0	0	0	9	0	0	0	9
Louisiana	1	1	1	1	0	0	1	0	5
Michigan	8	1	0	0	0	2	0	2	13
Minnesota	7	5	7	0	2	0	0	1	22
Nebraska	6	3	0	0	0	0	0	0	9
New York	1	1	2	2	10	0	2	0	18
Ohio	6	1	6	1	0	3	0	1	18
Oklahoma	1	0	2	2	0	0	0	0	5
Pennsylvania	5	2	3	1	2	2	0	2	17
South Carolina	2	1	2	0	0	0	0	1	6
South Dakota	0	3	3	0	3	0	1	0	10
Texas	6	3	0	0	0	0	0	0	9
Virginia	10	5	2	1	0	0	0	0	18
West Virginia	2	0	0	0	3	0	0	0	5
Wisconsin	8	1	5	0	15	0	0	0	29
Wyoming	2	1	5	0	0	0	2	2	12
Total	96	36	76	13	70	14	31	13	349

NOTE: FDR = full-depth repair
PDR = partial-depth repair
DGD = diamond grinding
LTR = load transfer restoration
CAS = crack and seat and AC overlay
UNBOL = unbonded concrete overlay
BOL = bonded concrete overlay
ES = edge support (tied PCC shoulder or edge beam)

* Represents the number of different uniform sections in the database. In addition, there are typically two replicate sample units for each different design.

Table 2. Summary of monitoring and design data for each rehabilitation technique.

DATABASE TYPE: MONITORING DATA

DATABASE	CONTENTS OF EACH RECORD	NUMBER OF RECORDS
FULL-DEPTH REPAIR	INDIVIDUAL PATCH DISTRESSES	2001
PARTIAL-DEPTH REPAIR	INDIVIDUAL PATCH DISTRESSES	1296
DIAMOND GRINDING	SAMPLE UNIT DISTRESSES	134
CRACK AND SEAT	SAMPLE UNIT DISTRESSES	120
BONDED OVERLAYS	SAMPLE UNIT DISTRESSES	50
UNBONDED OVERLAYS	SAMPLE UNIT DISTRESSES	21
EDGE SUPPORT	SAMPLE UNIT DISTRESSES	24
LOAD TRANSFER REST.	INDIVIDUAL JOINT AND CRACK DISTRESSES	421

DATABASE TYPE: DESIGN DATA

DATABASE	CONTENTS OF EACH RECORD	NUMBER OF RECORDS
FULL-DEPTH REPAIR	INDIVIDUAL PATCH DESIGN	233
PARTIAL-DEPTH REPAIR	INDIVIDUAL PATCH DESIGN	87
DIAMOND GRINDING	GRINDING TECHNIQUE DESIGN	105
CRACK AND SEAT	CRACK AND SEAT DESIGN	114
BONDED OVERLAYS	OVERLAY DESIGN	39
UNBONDED OVERLAYS	OVERLAY DESIGN	19
EDGE SUPPORT	SHOULDER/EDGE BEAM DESIGN	17
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The data sources and collection procedures used in this research study are described below.

1.1.1 Field Condition Surveys

A standard field condition survey was performed on each project or uniform section. The procedures used in the collection of condition data closely follow those described in NCHRP Project 1-19 (COPES) study for field data collection.(1) The distress identification manual developed for the COPES study was used as a standard for the identification and measurement of distresses and their severity levels.

The term "uniform section" was defined in the COPES study as a section of pavement with "uniform characteristics along its length including structural design, joint design and spacing, reinforcement, truck traffic, subgrade conditions, and distress".(3) To properly incorporate rehabilitation technique variation (e.g., different full-depth repair designs, different overlay thicknesses, etc.) into the uniform section concept, it was necessary to expand the definition of a uniform section to include uniformity of rehabilitation design.

Preliminary Work

The first step in project selection was to contact State department of transportation personnel to determine their interest in participating in the study. Project description forms were then sent to those States who were interested and willing to participate. The State personnel then selected representative rehabilitation projects that included one or more of the eight techniques, and filled out a project description form for each section.

The project description forms from all over the country were reviewed, any inappropriate sections excluded (where one or more of the eight rehabilitation techniques were not included for example), and detailed data collection forms were sent to the State for the selected projects in their State. Upon completion of these data collection forms, data entry into the database was begun. If important data items were missing, an additional written request was sent to the State for this information. In some cases, this information was retrieved in person.

The beginning and ending markers (stations, mileposts, landmarks) of the project were determined as best as possible in the office by verbal communication with State department of transportation personnel, prior to the commencement of surveying procedures. These steps ensured that any changes in uniform section pertaining to variations in the design of the original pavement or rehabilitation design would not be overlooked.

Field Work

After the preliminary identification of the uniform sections to be surveyed, the following procedures were used in the field data collection process.

- A two-person trained survey crew made at least one pass over the project areas at the posted speed. During the pass, changes in the pavement condition, in situ foundation conditions (cut/fill) and drainage were noted. This pass was used to determine whether one or more uniform sections were necessary on the basis of pavement distress, grade or drainage variation.

- The uniform sections were surveyed by representative sampling. Usually two 1000-ft [305 m] sample units were surveyed per uniform section. Sections of considerable length (greater than ten miles [16.1 km]), received a third sample to ensure reasonable coverage. The location of the sample units was selected randomly; however, sample units were selected such that grade conditions (cut/fill) along their lengths were as uniform as possible. Also, in consideration of the fact that a project or sample unit might require additional evaluation at some future date, many of the sample units were located at milepost markers for easier future identification.

- A very comprehensive distress survey was conducted along each sample unit. The condition of both lanes was measured where traffic or other conditions did not pose a serious safety hazard to the survey crew. The outer lane survey was conducted from the outer shoulder of the pavement and, likewise, the inner lane survey was conducted from the inner shoulder. Measurements of faulting and joint widths were taken 1 foot [0.3 m] from the PCC slab lane edge. Also, photographs of the pavement, general topography and other distresses were recorded.

- The presence of subsurface drainage and the condition of subsurface drainage facilities were noted.

1.1.2 Original Pavement and Rehabilitation Design Factors

For the collection of this data, the as-built original construction and rehabilitation construction plans, as well as special provisions for the rehabilitation projects, were obtained for each project. Much of the required data was obtained from these records; however, consultation with State department of transportation personnel was also necessary to collect additional information. Finally, data from other sources such as published reports were also used.

A detailed listing of the variables collected under this study pertaining to original pavement and rehabilitation design and rehabilitation field monitoring is included in volume IV.

1.1.3 Traffic Data

Values for the average annual daily traffic and percent heavy commercial truck traffic were also collected from the State department of transportation records. Historical information was collected where the data was available; however, in some instances only current traffic levels were obtained. For the determination of the number of equivalent 18-kip [80 kN] single-axle loadings (ESALs) accumulated on each project, FHWA W-4 truck axle load distribution data were utilized to compute the truck factors over the life of the pavements. The number of accumulated axle loads from the time of original pavement construction until the time each rehabilitation technique was applied, and from then until the time of survey, was calculated for each project.

1.1.4 Environmental Data

The average monthly precipitation and average daily minimum, maximum and mean temperatures were taken from National Oceanic and Atmospheric Administration data. The nearest weather station was assumed to be representative of the conditions at the project site. The mean Freezing Index was interpolated from the contour map developed by the Corps of Engineers for the continental United States.(3) The climatic zone as classified by Carpenter was also determined for each site.(3)

1.2 LABORATORY STUDIES

Laboratory studies included the first comprehensive testing of dowel anchoring procedures and designs. Full-scale repeated shear loading of dowels was conducted for up to one million load repetitions using slabs cut from I-70 in Illinois. Many different design, material and construction variables were considered in a factorial type experimental design.

1.3 ANALYTICAL STUDIES

Analytical studies were accomplished primarily to develop prediction models for rehabilitated pavement deterioration so that the service life of different rehabilitation techniques could be estimated. Twelve distress models were developed including reflective cracking, faulting, rutting, and serviceability for most of the above rehabilitation techniques. These models were incorporated into the evaluation and rehabilitation system.

1.4 EVALUATION AND REHABILITATION SYSTEM

A comprehensive concrete pavement evaluation and rehabilitation system was developed for jointed plain, jointed reinforced and continuously reinforced concrete pavements. This system is intended to assist the design engineer in the following rehabilitation project design activities:

- Project data collection.
- Evaluation of present condition.
- Prediction of future condition without rehabilitation.
- Physical testing recommendations.
- Selection of feasible rehabilitation approaches.
- Development of detailed rehabilitation recommendations.
- Prediction of performance of the rehabilitation strategy.
- Cost analysis and selection of the preferred rehabilitation alternative.

The results of this research are published in four volumes:

- Volume I — Repair Rehabilitation Techniques
- Volume II — Overlay Rehabilitation Techniques
- Volume III — Concrete Pavement Evaluation/Rehabilitation System
- Volume IV — Appendixes

Each of these volumes are stand-alone volumes that present the data, analyses and conclusions for each of the rehabilitation techniques and the evaluation and rehabilitation system.

CHAPTER 2

DIAMOND GRINDING

2.0 RESEARCH APPROACH

Diamond grinding of jointed portland cement concrete (PCC) pavements has been part of experimental and routine restoration work since 1965.(10,11,12) The first major project ground in that year was recently reground to restore rideability. Within about the last 10 years, diamond grinding work has increased greatly. The capabilities of diamond grinding equipment has also increased greatly during this time period.(10)

To date there has been no nationwide documentation of the performance of diamond grinding. Several specifications exist for diamond grinding and the technique has proven very effective in several States. It has been very effective in the removal of faulting and surface wear. However, the overall effectiveness of the technique in terms of extending pavement life has not been determined and verified through field performance throughout the country.

All available references were reviewed for diamond grinding of jointed concrete pavements. Some new publications are available that have added considerable knowledge to the design, construction and performance of diamond grinding.(2,3,5,6,10,11,15)

The development of an extensive database containing information on the original pavement design, traffic, environmental conditions and performance of diamond grinding was required to determine the effectiveness of grinding. The database was developed in order to allow analysis to include the consideration of many factors which might affect performance.

To obtain all of the necessary database elements the following methods and sources were utilized:

- Extensive field surveys including mapping of cracks, physical measurements and subjective ratings were conducted on each project to document the current condition of the ground pavement.
- The design of the original pavement structure was determined from "as-built" plans and verbal communication with State DOT personnel.
- Environmental data were taken from historical documentation of temperature and precipitation by the National Oceanic and Atmospheric Administration.
- Traffic estimates, including average daily traffic and percent commercial trucks, were obtained from the State DOT's. For the calculation of accumulated axle loads on each project, Federal Highway Administration historical W-4 tables on axle load distributions for respective States and pavement classifications were used.

Physical test data were not collected. This data would have greatly increased the ability to analyze and interpret the pavement deterioration identified from visual surveys. The most useful tests would include heavy load deflection testing and coring (plus laboratory testing). An understanding of the physical properties of the pavement layers, loss of support, load transfer and gradations (of the base) would have made it possible to conduct structural, material and drainability evaluations.

2.1 DATABASE AND DATA COLLECTION

A total of 76 diamond grinding sections obtained from 19 different States were included in the database. Two sample units having a length of about 1000 ft [305 m] were obtained from each of the sections where possible (114 sample units total). The projects included in the database represent many of the diamond grinding projects constructed after 1976 when this type of work began in earnest throughout the country. These pavements were field surveyed between June 1985 and July 1986. Figure 2 shows the general location of the diamond grinding projects. A fair distribution exists in the different geographic and climatic zones.

A detailed description of the field and office data collection procedures is given in volume IV. There were five basic data types that were necessary for the development of life prediction models and for analysis aimed towards the development and improvement of design and construction procedures. These include:

- Field condition data.
- Original pavement structural design, and historical improvement data.
- Rehabilitation design factors.
- Historical traffic volumes and classifications, W-4 load concrete tables and the calculation of accumulated 18-kip [80 kN] equivalent single axle loadings.
- Environmental data.

A complete list of all of the variables considered in the field surveys is given in table 3. The design variables for the original pavement which are contained in the database are given in table 4.

The database is comprehensive, containing as many projects as were available or could be included within available resources. This was done to provide a wide range of data to facilitate regression analysis for the development of performance models.

Figures 3 and 4 give the age and accumulated 18-kip [80 kN] equivalent single axle load (ESAL) distribution (since grinding). The age distribution indicates the relative newness of the grinding technique with a mean of 4 years and a range of 1 to 9 years. The ESAL distribution (after grinding) shows a mean of 2 million and a range of 0.22 to 7.81 million in the outside traffic lane.

The physical design of the pavements are summarized as follows:

Pavement type:	39 JRCP, 75 JPCP
Slab thickness:	7 to 12 in [17.8 to 30.5 cm]
Joint spacing:	15 to 100 ft
Base type:	54 percent granular and 46 percent stabilized
Load transfer:	38 percent doweled and 62 percent undoweled
Shoulder type:	95 percent AC, 5 percent tied PCC
Subdrainage:	82 percent none and 18 percent edge drains

Subgrade and climate factors show the following ranges:

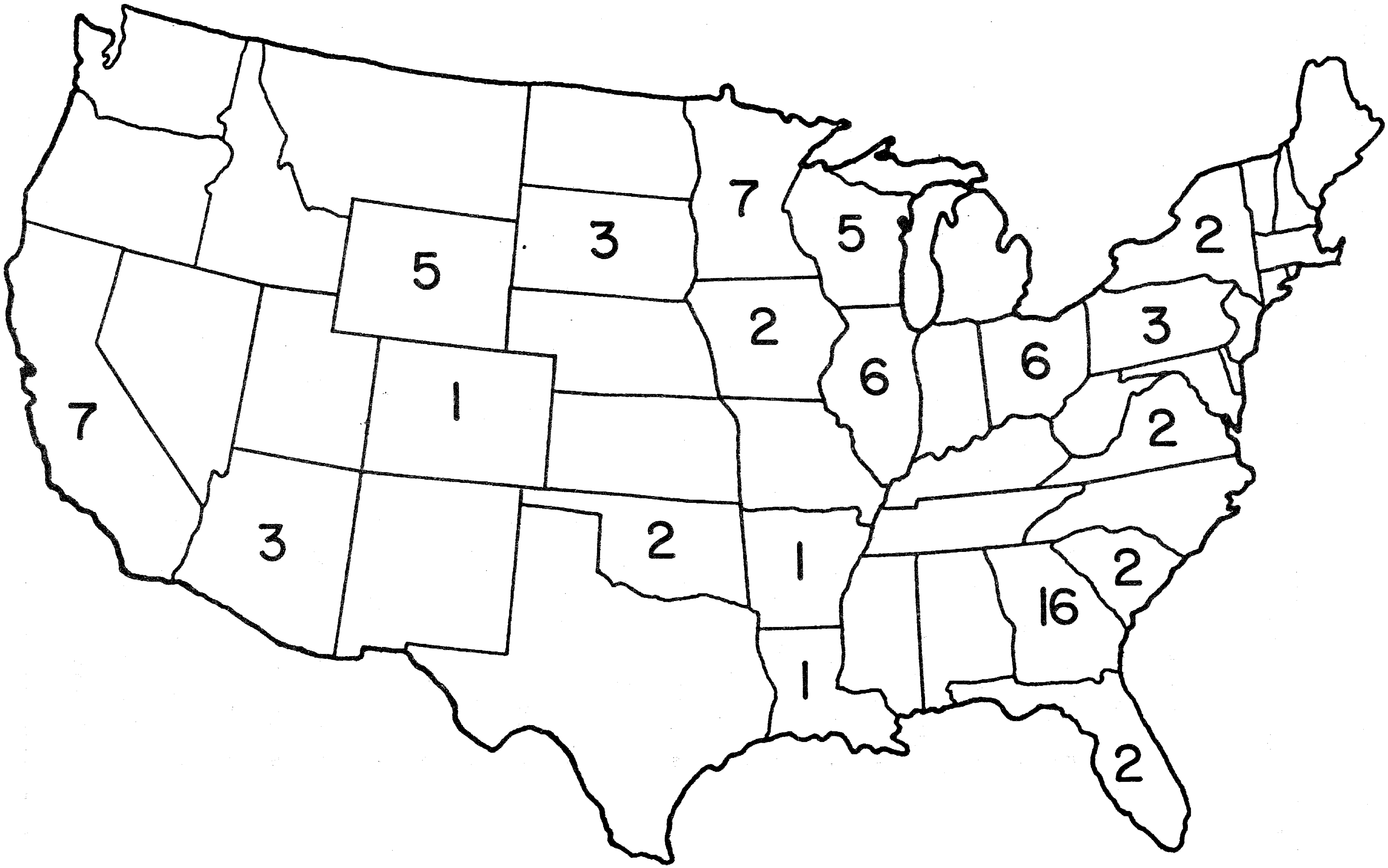


Figure 2. Location of diamond grinding sections included in the database.

Table 3. Pavement condition variables collected in the field surveys.

FIELD DATA VARIABLES:

General:

- Sample Unit.
- Foundation of Sample Unit.
- Condition of Drainage Ditches.
- Subsurface Drainage Present and Functional.
- Number of Transverse Joints in the Sample Unit.

Slab Distress Variables:

- Transverse Cracking.
- Transverse "D" Cracking.
- Longitudinal Cracking.
- Longitudinal "D" Cracking.
- Longitudinal Joint Spalling.
- Scaling, Crazing, Map Cracking, Shrinkage Cracking.

Joint Distress Summary:

- Spalling Transverse Joint.
- Corner Spalling.
- Pumping.
- Mean Faulting over Sample Unit.
- Mean Joint Width over Sample Unit.
- Corner Breaks.
- "D" Cracking Along Joint.
- Reactive Aggregate Distress.
- Sealant Conditions.
- Incompressibles in Joint.

Table 4. Original pavement construction and design variables.

ORIGINAL PAVEMENT DESIGN VARIABLES:

General:

- Identification Number (Highway #, Milepost, Direction).
- Beginning & Ending Mile Marker (Station).
- Number of Through Lanes.
- Type of Original Pavement (JPCP, JRCP).
- Layer Descriptions, Thicknesses, Material Types.
- Date of Original Pavement Construction.
- Dates and Description of Major Pavement Improvements.

Joints and Reinforcing:

- Average Contraction Joint Spacing.
- Skewness of Joints.
- Expansion Joint Spacing.
- Transverse Contraction Joint Load Transfer System.
- Dowel Diameter.
- Type of Slab Reinforcing.
- Longitudinal Bar/Wire Diameter.
- Longitudinal Bar/Wire Spacing.

Subgrade, Shoulder and Drainage:

- Type of Subgrade Soil (Fine Grained, Coarse Grained).
- Outer Shoulder Surface Type.
- Original Subsurface Drainage Type.
- Original Subsurface Drainage Location.

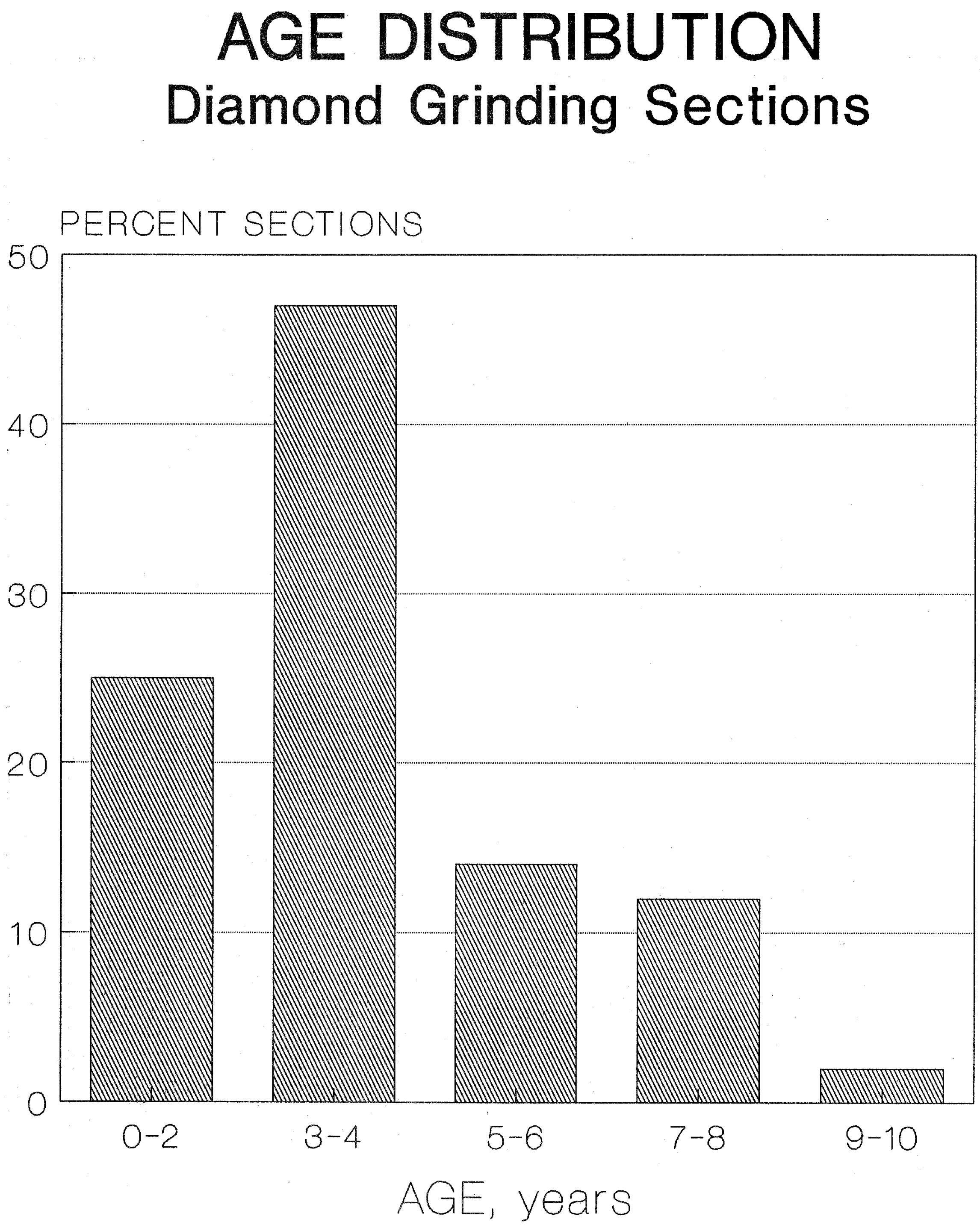


Figure 3. Age distribution for diamond grinding sections.

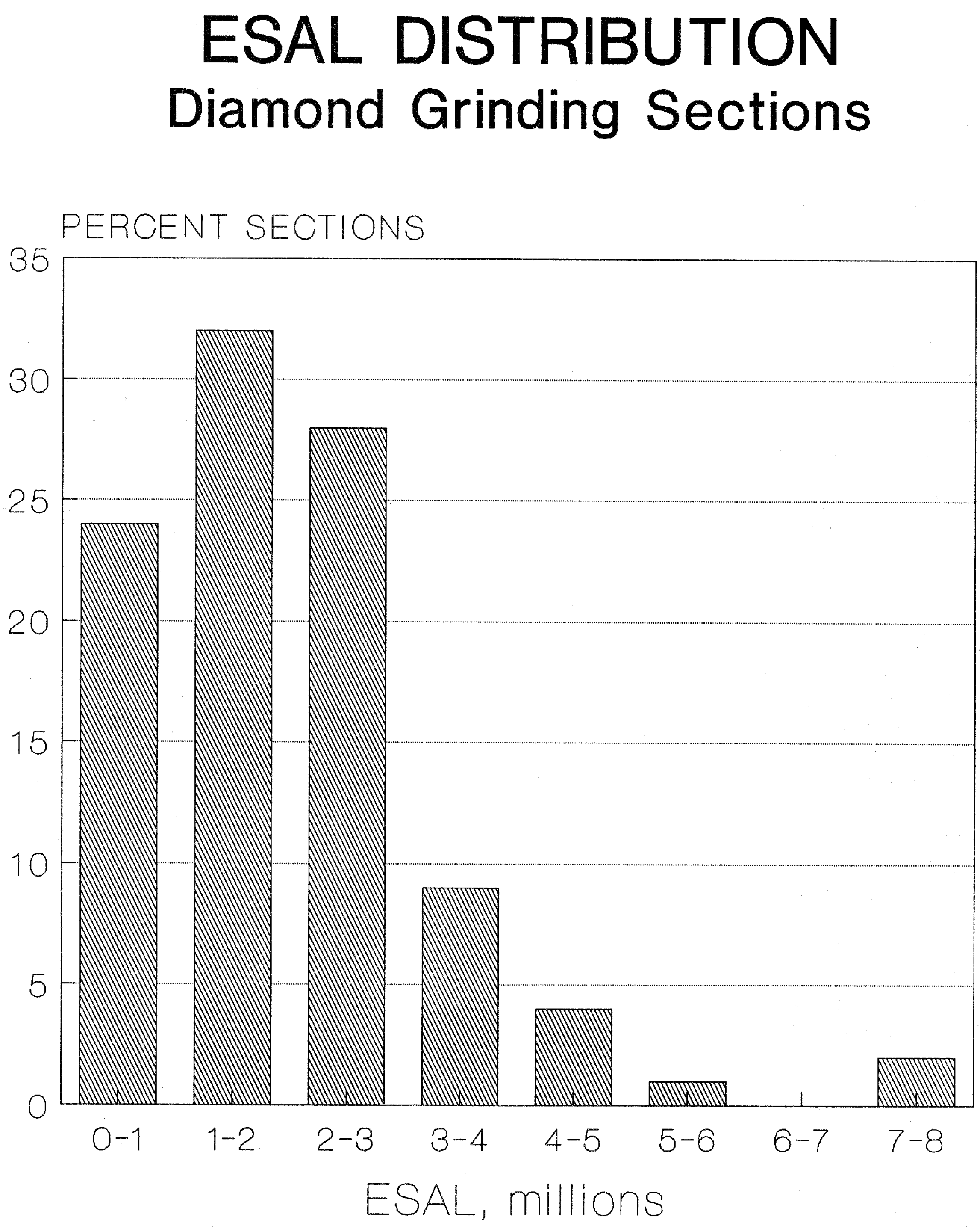


Figure 4. ESAL distribution for diamond grinding sections.

Subgrade soil type:	53 percent fine grained, 47 percent coarse
Annual precipitation:	9 to 61 in [23 to 155 cm]
Mean Freezing Index:	0 to 1750-degree days below freezing

2.2 FIELD PERFORMANCE AND EVALUATION

Diamond grinding greatly improves the rideability of the pavement through removing faulting. Diamond grinding also increases the friction resistance of the surface immediately after grinding.(7,8,10,15)

An evaluation of distresses which may impede the structural capacity, rideability and friction resistance of the ground pavements is presented below.

The distresses that have been identified which may directly affect the structural integrity of the ground pavement are transverse and longitudinal cracking, corner breaks, joint spalling, joint faulting, pumping and "D" cracking. Rideability is affected by most of the aforementioned distresses. Friction resistance is decreased by the wear and polish of the surface texture. Table 5 gives a summary of the mean and range of major distresses, normalized to a per-mile basis of the outer lane, measured for the diamond grinding sections.

The severity levels employed in describing distresses are those defined in NCHRP Project 1-19 (COPES) distress manual.(1) For example, low severity cracking describes hairline cracking, medium severity describes working cracks and high severity a badly spalled and faulted crack needing immediate repair.

2.2.1 Transverse Cracking

Transverse deteriorated cracks on jointed concrete pavements are largely caused by a combination of traffic loading fatigue damage and thermal curling stresses. In addition for JRCP, a contributing factor may be a lock up of transverse joints from corrosion or misalignment of dowels. The distribution of deteriorated transverse cracks (medium and high severity) in the truck lane for ground pavements is shown in figure 5. Forty-three percent of all uniform sections contained no deteriorated transverse cracking, while 21 percent contained over 825 ft per mile [156 m per km] of deteriorated cracks. A serious level of cracking, where pavement rehabilitation is needed, is approximately 825 ft per mile [156 m per km] (this value was determined as the average of all projects in the NCHRP 1-19 database that had a present serviceability index less than 3.0). This amount can be conceived as a working crack every 77 ft [23.5 m] for JRCP that is spalled and faulted (or a working cracking in about 50 percent of the slabs if joint spacing was 40 ft [12.2 m]) and about 12 percent cracked slabs for JPCP.

A substantial proportion of the sections had a large amount of cracking at the time of survey after grinding (21 percent, or about 1 out of 5 sections). There is no way to determine if the cracking existed at the time of grinding, and was not repaired, or whether it developed after grinding. Existing deteriorated cracks would lead to shortening the life of restoration (where the average age is 4 years and 2 million ESALs).

From this information, it is concluded that about one out of four sections had a significant amount of cracking which leads to the conclusion that these sections were probably structurally inadequate before grinding and should of been overlayed or reconstructed, instead of restored with no structural improvement.

Table 5. Summary of distress types identified for diamond grinding projects (outer traffic lane only).

Distress Type	Severity	Mean	Range
Transverse Cracking	Medium and High	459	0 to 2928 ft /mile
Longitudinal Cracking	Medium and High	91	0 to 1900 ft /mile
Corner Breaks	All	7	0 to 222 /mile
"D" Cracking	All	6 percent sections	
Pumping	Low	99 percent sections	
	Medium	1	
	High	0	
Joint Spalling	Low	96 percent sections	
	Medium	4	
	High	0	

Note: 1 mile = 1.609 km
1 ft/mile = 0.1894 m/km

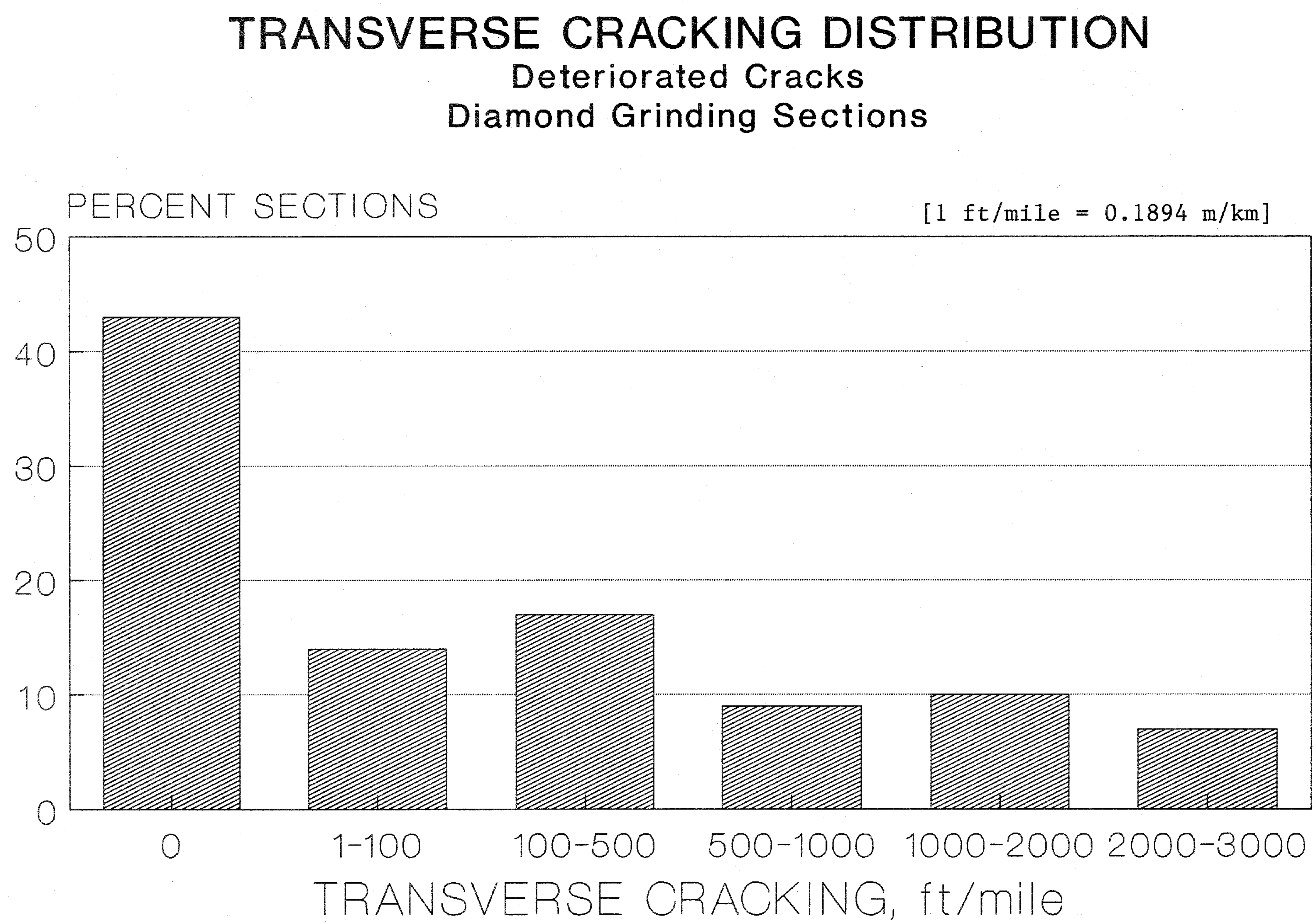


Figure 5. Transverse cracking distribution for diamond grinding sections.

2.2.2 Longitudinal Cracking

Longitudinal cracking is generally caused by late sawing, shallow saw cuts, or the use of plastic inserts that do not create an adequate weakened plane for the longitudinal joint. Figure 6 is a histogram of the longitudinal medium to high severity cracking on the diamond ground sections. Seventy-five percent had no deteriorated longitudinal cracking. Only 5 percent had more than 500 ft [152 m] of deteriorated longitudinal cracking per mile.

Three sections had over 1500 ft per mile [284 m per km] of deteriorated longitudinal cracking. It is impossible to determine whether this cracking occurred before or after grinding.

2.2.3 Corner Breaks

Corner breaks are generally due to the loss of support beneath the slab caused by erosion of the base course or subgrade. Projects that are diamond ground for faulting (which is indicative of pumping and thus loss of support) normally have some loss of support. Significant faulting can not occur without some erosion of the underlying layers of the concrete pavement, resulting in some loss of support.(6) Corner breaks are a good indicator of structural deficiency.

Corner breaks occurred on 19 percent of the sections. However, more than 25 corner breaks per mile (which is considered serious) occurred on only 6 percent of the sections. Three sections showed more than 100 corner breaks per mile [63 per km]. It is not known if the breaks occurred before or after diamond grinding. In either case, it is indicative that a number of sections were diamond ground without consideration or determination of support conditions.

2.2.4 Joint/Crack Faulting and Pumping

Faulting develops from pumping and erosion of underlying materials through the combination of four factors:

- The movement of heavy wheel loads across the joint or crack.
- The presence of free moisture in the pavement subbase and/or subgrade.
- A subbase or subgrade material that is erodible (contains many fines).
- A deficiency in load transfer across the joint.(3,7,8)

If these factors exist, the subbase and/or subgrade materials have the potential to pump beneath the approach joint with traffic loadings. Pumping generally will force water and fines from under the leave side and either deposit the fines under the approach side of the joint or force the fines out from beneath the slab through the longitudinal joint. This action is dependent on the deflection of the slab corners, and will be more severe on pavements that exhibit poor load transfer. The movement of fines will normally lift the approach side and leave a void under the leave side of the joint and lead to a differential in elevation from approach to leave side causing the faulting step-off.(3)

The distribution of transverse joint faulting for the diamond grinding sections is shown in figure 7. The average faulting in the drive lane was 0.065 in [0.165 cm], with individual sections ranging from 0.01 to 0.33 in [0.025 to 0.838 cm]. Faulting becomes detrimental for JPCP when it exceeds about 0.13 in [0.38 cm], which occurred on 7 percent of the sections.(52)

Low severity pumping (e.g., water bleeding, a few blowholes, signs of some erosion) was observed on many of the diamond grinding sections. However, only one section showed a medium level of pumping (e.g., substantial fines pumped on to the

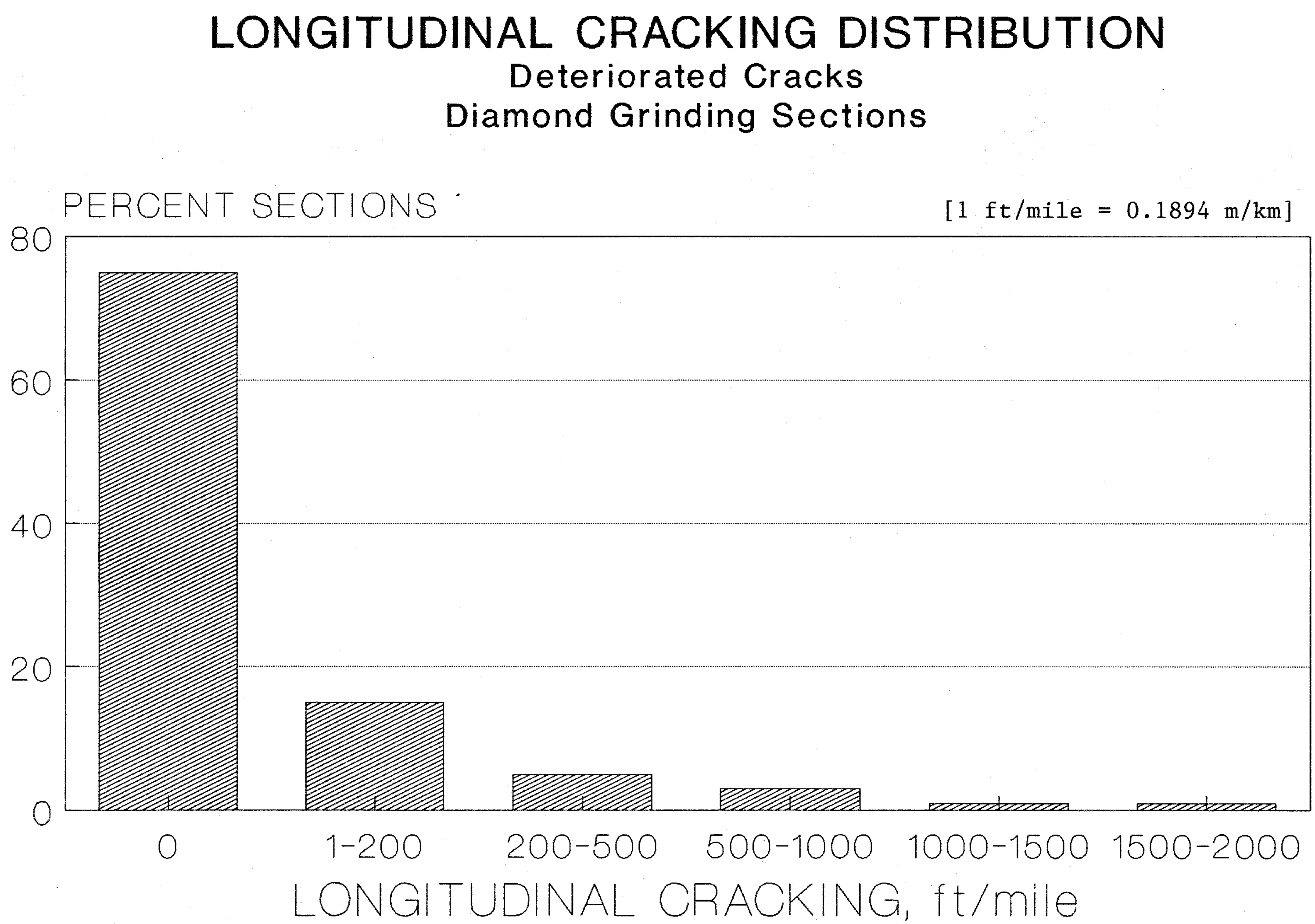


Figure 6. Longitudinal cracking distribution for diamond grinding sections.

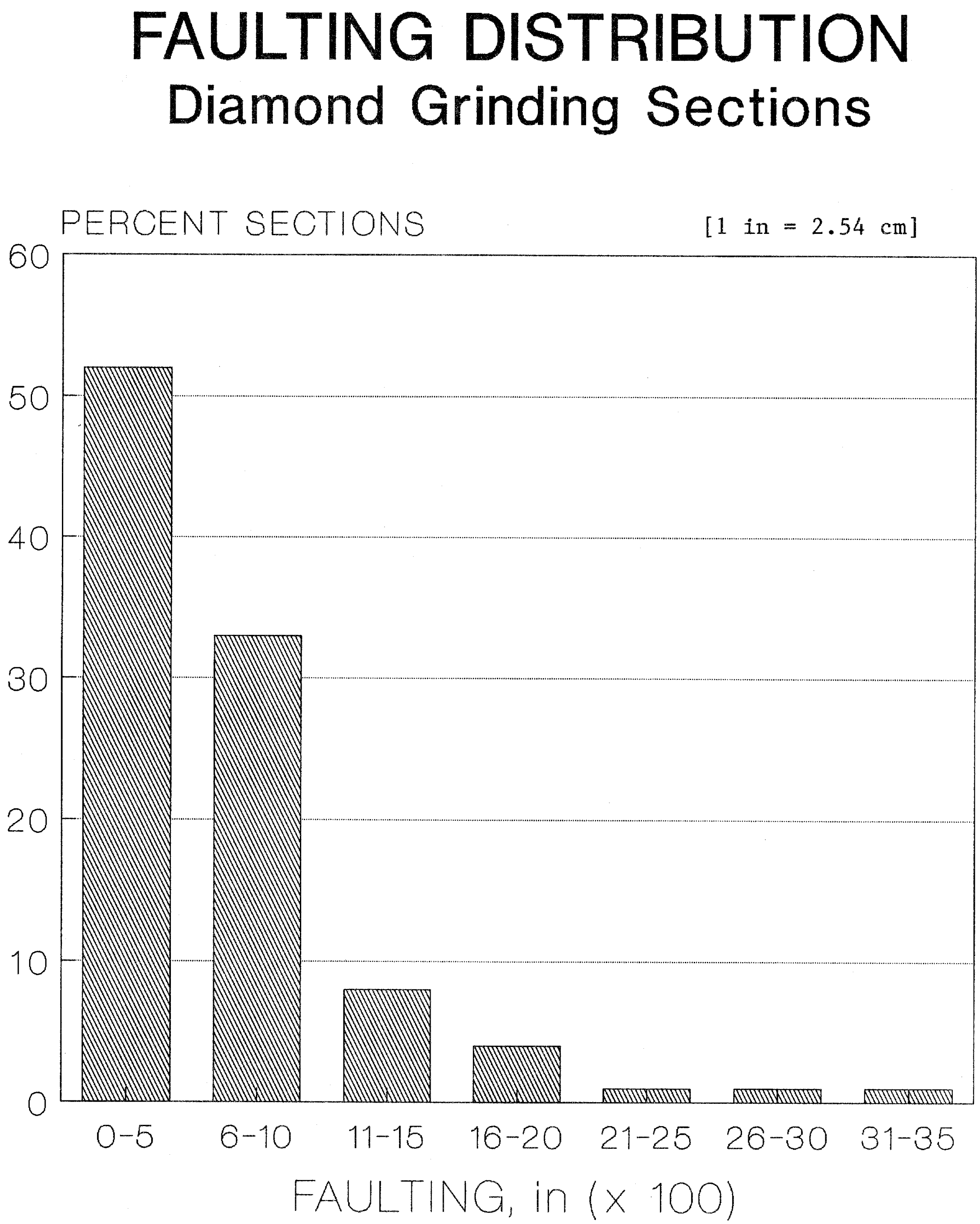


Figure 7. Faulting distribution for diamond grinding sections.

shoulder). The placement of edge drains and tied PCC shoulders on several sections probably reduced the visual signs of pumping.

2.2.5 "D" Cracking

"D" Cracking is a durability problem of the aggregates used in the concrete mix. It is caused by the freeze-thaw expansive pressures of certain coarse aggregates. The pressures developed in the concrete tend to cause fine hairline cracks near and parallel to joints and cracks which eventually spall out.

Only 6 percent of the diamond grinding sections exhibited "D" cracking. This is probably indicative of the belief that pavements that have significant "D" cracking will deteriorate rapidly, and should not be rehabilitated by restoration (particularly by grinding).

2.2.6 Wearout Of Grinding Texture

The type of texture developed by grinding provides a good friction factor immediately after grinding.(10,15) The ridges produced improve the surface macrotexture and provide an escape route for moisture under a tire.

Data was not available on friction numbers for any of the sections. On some of the sections it was evident that there was wear of the ground texture in the wheel paths. This was determined by running the hand across the texture in the wheel path and then near the center line where fewer tires pass. The rate of wear could not determined. This is a concern that warrants further detailed study because the loss of the texture could result in a loss in friction. The loss probably varies with different aggregate hardness and with the width of land area between the grooves (spacing of blades).

2.3 PERFORMANCE MODEL

2.3.1 Model Development

Faulting is a major distress type that develops after grinding on most pavements. A predictive model was developed for transverse joint faulting using nonlinear regression techniques as included in the SPSS statistical package.(16)

As a first step in analyzing the data, all independent variables that were believed to have significant influence on the faulting of ground pavements were identified. These variables were then considered in the development of the faulting model with nonlinear regression.

In addition to the regular 114 diamond grinding projects, data from doweled joint load transfer restoration sections were also added so that this type of work, done concurrently, could be considered. All of these sections were also diamond ground.

Extensive time was spent developing the final model, however, it should be considered a tentative/initial model because of the limited nature of the database. As diamond grinding is applied in more States with differing climates and designs, these initial models can be revised to include more variables and wider ranges of applicability.

2.3.2 Faulting Model

The variables that entered the faulting model included design, traffic, subgrade, climate and additional restoration work. The final model for faulting is as follows:

$$FAULT = -5.62\ (ESAL + AGE)^{0.54}\ [\ 5.85\ (1 + DRAIN + SUB)^{0.0529} - 3.8E\text{-}9\ (FI / 100)^{6.29} + 0.484\ (THICK + PCCSH)^{0.335} + 0.1554\ BASE - 7.163\ JSPACE^{0.0137} + 0.1136\ LTR\ ] / 100$$

Where:

FAULT = mean transverse joint faulting after grinding, ins. (outer traffic lane)

ESAL = accumulated 18-kip equivalent single-axle loads after grinding, millions (outer traffic lane)

AGE = time after diamond grinding, years

DRAIN = 0, if no edge drains after grinding
1, if edge drains exist after grinding

SUB = 0, if fine grained subgrade soil exists (A4 - A7)
1, if coarse grained subgrade soil exists (A1 - A3)

FI = freezing index, average F. degree days below freezing

THICK = original slab thickness, in

PCCSH = 0, if no tied concrete shoulder exists
1, if tied concrete shoulder exists

BASE = 0, if existing base is granular material
1, if existing base is stabilized granular material (asphalt, cement)

JSPACE = mean transverse joint spacing, ft

LTR = 0, if no retrofit dowels placed
1, if retrofit dowels placed

Statistics: $R^2 = 0.38$ (Significant at 0.00001 level)
Standard error = 0.027 in [0.069 cm]
n = 114 sections (diamond grinding without
without load transfer restoration (LTR),
plus 72 joints with dowel LTR and diamond grinding)

The mean and ranges of factors are as follows:

Factors	Mean	Range
Faulting	0.06	0.01 - 0.33 in
ESAL	1.94	0.22 - 7.8 million outer lane since grinding
Age	4	1 - 9 years
Slab Thick.	9.0	7.0 - 12.0 in

Factors	Mean	Range
Joint Spacing	38	15 - 100 ft
Dowel Dia.	---	0 (no dowels) - 1.25 in
PCC Shoulder	---	0 (no PCC sh.) - 1 (tied PCC sh.)
Base Type	---	0 (granular) - 1 (stabilized)
Edge Drains	---	0 (none) - 1 (yes)
Subgrade Type	---	0 (fine-grained) - 1 (coarse grained)
Freezing Index	436	0 - 1750-degree F. days below freezing
Annual Prec.	33.5	9.3 - 61.1 in
Pavement Type	JRCP = 39 sections JPCP = 75 sections	

Note: 1 in = 2.54 cm
1 ft = 0.3048 m

Several factors were identified which affect the rate of faulting of a ground pavement. Two typical or "standard" pavements were defined. Each factor was varied over a typical range and the change in faulting determined. The ratio of the higher faulting value to the lower value was computed. The results are shown below:

JRCP	Factor	Range	Ratio High Fault/Low Fault
	ESAL, millions	1 to 10	5.9 Increase
	Slab Thick., in	8 to 12	1.6 Decrease
	Joint Spacing, ft	25 to 75	1.4 Increase
	Base Type	Gran. to Stab.	1.2 Decrease
	Subgrade Soil	Fine to Gran.	1.4 Decrease
	Freezing Index	0 to 1500	1.2 Increase
	Concrete Shoulder	No to Yes	1.1 Decrease
	Edge drains	No to Yes	1.4 Decrease
	Load trans. restor.	No to Yes	1.6 Decrease

JPCP	Factor	Range	Ratio High Fault/Low Fault
	ESAL, millions	1 to 10	6.0 Increase
	Slab Thick., in	8 to 12	3.0 Decrease
	Joint Spacing, ft	8 to 20	1.7 Increase
	Base Type	Gran. to Stab.	1.2 Decrease
	Subgrade Soil	Fine to Gran.	2.0 Decrease
	Freezing Index	0 to 1500	1.3 Increase
	Concrete Shoulder	No to Yes	1.2 Decrease
	Edge drains	No to Yes	2.0 Decrease
	Load trans. restor.	No to Yes	1.6 Decrease

Note: 1 in = 2.54 cm
1 ft = 0.3048 m

These results show that the variables are affecting faulting in the logical direction, and that some of them have a much larger effect than others. The variable having the greatest effect is traffic. The design factor showing the most effect is slab thickness. The subgrade soil type also has a major effect, probably due to improved subdrainage with a coarse grained soil. One interesting factor that did not enter the equation was the presence of dowels in the original pavement. It appears that after a pavement has faulted badly enough to require faulting, the dowels are too loose to have any impact on future faulting after grinding.

The additional restoration work; including concrete shoulders, edge drains and load transfer restoration; also has a significant effect on reducing faulting.

Graphs were prepared to further illustrate the results. Figure 8 shows faulting development for the standard JRCP (table 6) over its initial life (using NCHRP 1-19 model) and after diamond grinding (year 20) where no additional restoration work was completed. The results show that the faulting after grinding is more rapid than when the pavement was new.

Figure 9 illustrates the same faulting data for the standard JRCP (table 6) after diamond grinding both with and without the use of edge drains and tied concrete shoulders. The faulting of the pavement over its initial performance period, labeled "new", is also shown for comparison. Figure 10 shows the development of faulting for the same standard JPCP (table 6) after diamond grinding both with and without the use of edge drains and tied concrete shoulders (the faulting of the pavement over its initial performance period, labeled "new", is also shown for comparison).

Figure 11 shows faulting development for the standard JRCP (table 6) after diamond grinding both with and without the placement of edge drains.

Figure 12 illustrates faulting development for the standard JPCP (table 6) after diamond grinding both with and without the placement of edge drains.

Figures 13 and 14 show faulting development for grinding alone, grinding with tied PCC shoulders, grinding with load transfer restoration (using dowels) and grinding with tied PCC shoulders and load transfer restoration.

These results clearly show that it is important to provide additional restoration work when there are deficiencies in the existing pavement such as poor subdrainage and joint load transfer. These results are for only two "standard" or typical designs, and other existing conditions could produce different results. Therefore, the designer should apply judgement when using the faulting prediction model for determining whether or not to do other restoration work.

2.4 DIAMOND GRINDING -- DESIGN AND CONSTRUCTION GUIDELINES

2.4.1 Introduction

These Guidelines cover the use of diamond impregnated blades for grinding and texturing of portland cement concrete (PCC) pavements. Diamond grinding is used to restore surface profile and retexture the pavement. These guidelines have been updated from those developed in reference 6.

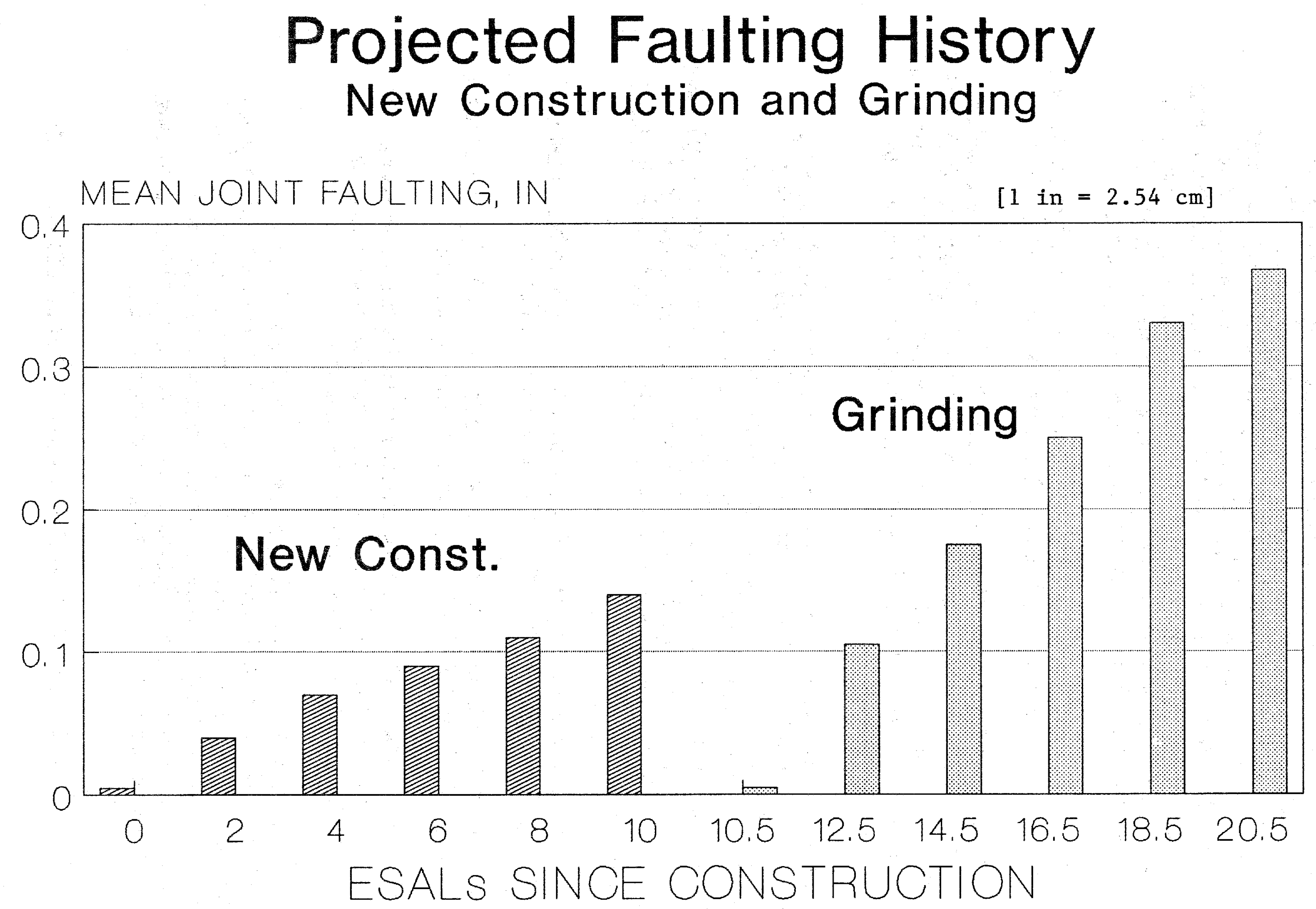


Figure 8. Estimated project faulting history for standard JRCP section for new construction and after restoration with diamond grinding.

Table 6. Typical "standard" pavement characteristics for faulting sensitivity analysis.

Factors	JRCP	JPCP
ESAL	0.5 million/year	0.5 million/year
Age (after grinding)	0 to 20 years	0 to 20 years
Edge Drains	None	None
Subgrade Soil	Fine grained	Fine grained
Freezing Index	250	0
Slab Thickness	9 in	8 in
Shoulder Type	Asphalt Concrete	Asphalt Concrete
Base Type	Granular	Stabilized
Joint Spacing	50 ft	15.5 ft
Load Transfer Restor. (dowels)	No	No

Note: Sensitivity analysis was conducted by varying one factor at a time over the range of age with corresponding change in ESAL.

1 in = 2.54 cm
1 ft = 0.3048 m

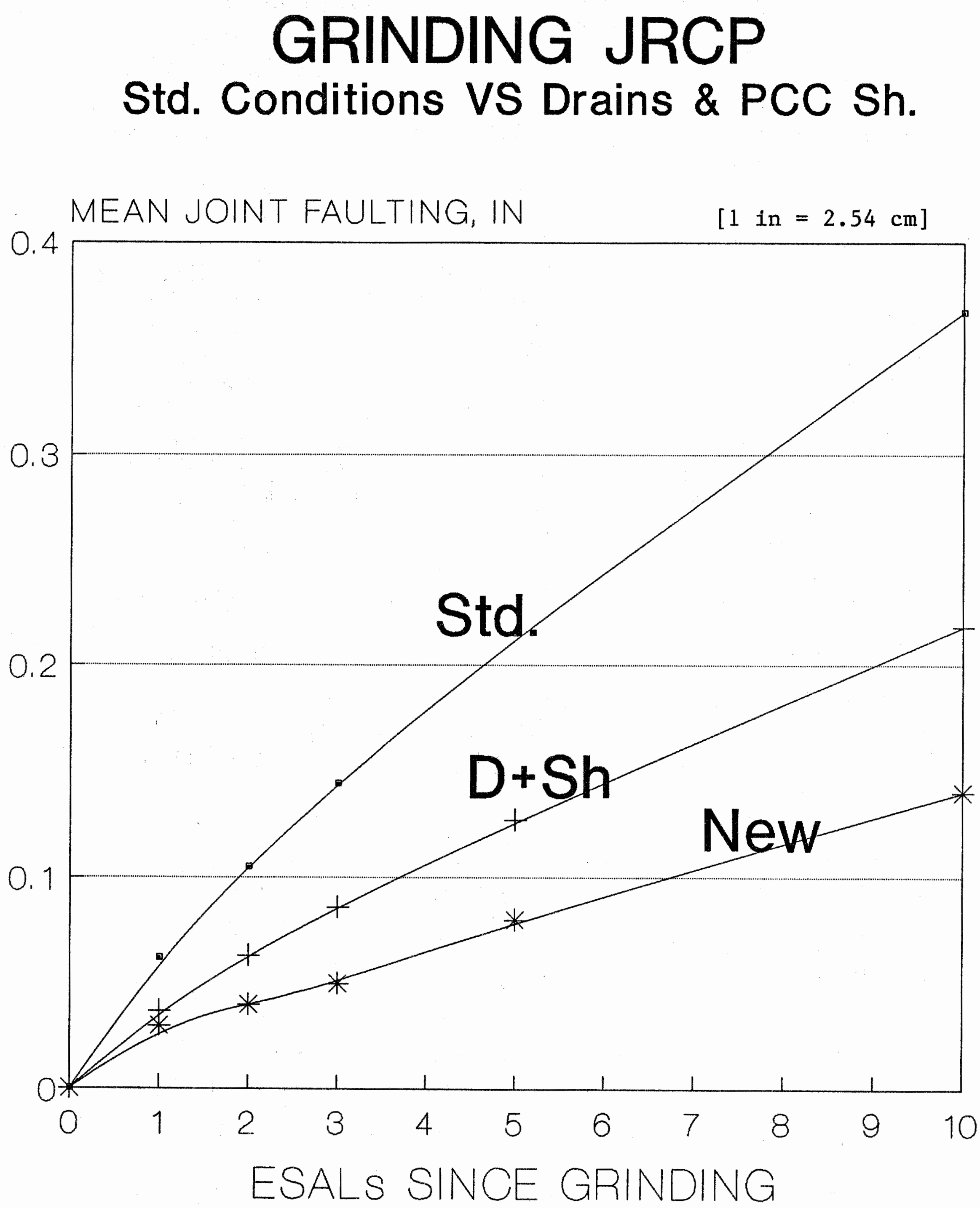


Figure 9. Faulting projections for standard JRCP and standard plus drains and PCC shoulders (Estimated faulting for new section also shown for comparison).

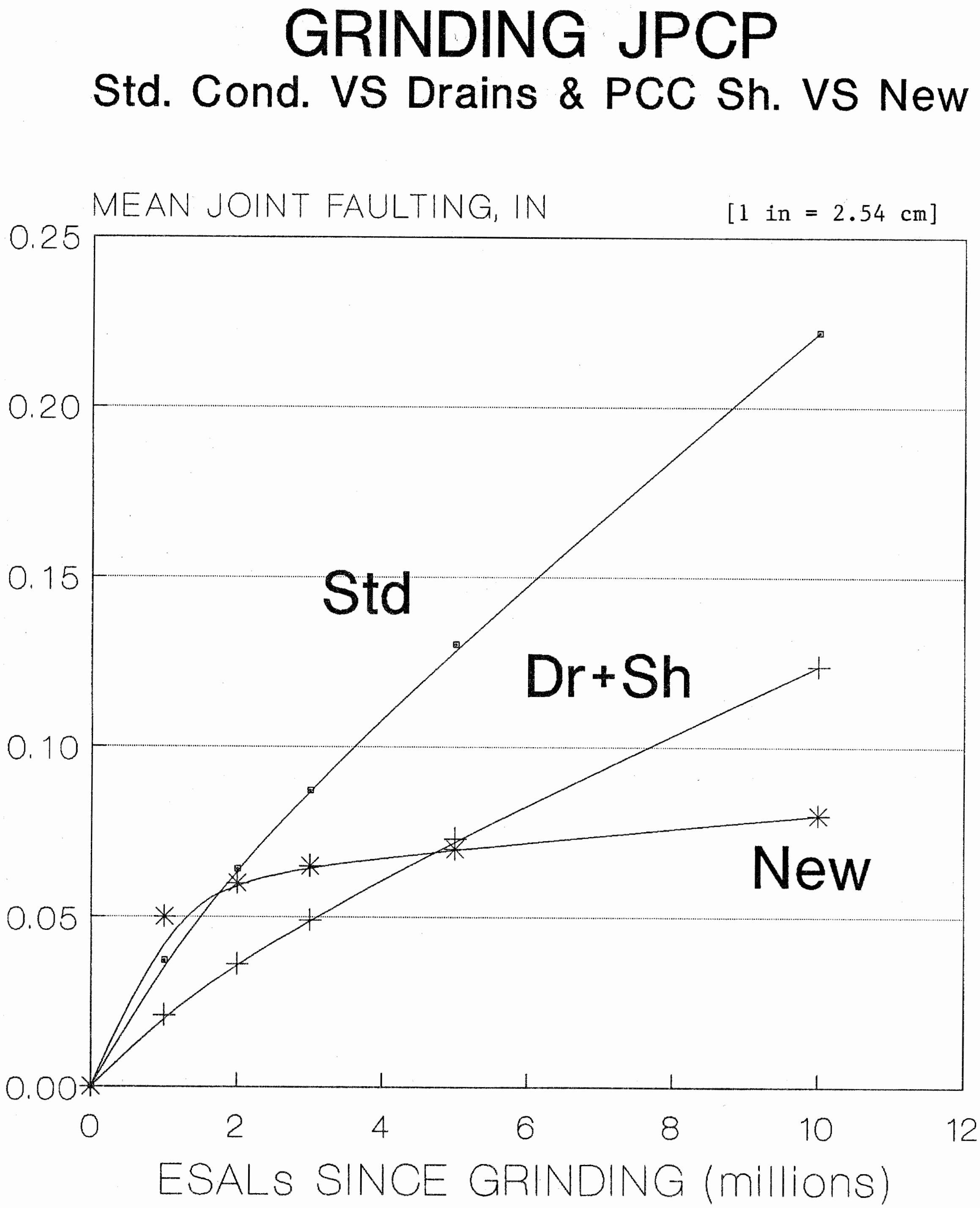


Figure 10. Faulting projections for standard JPCP and standard plus drains and PCC shoulders (Estimated faulting for new section also shown for comparison).

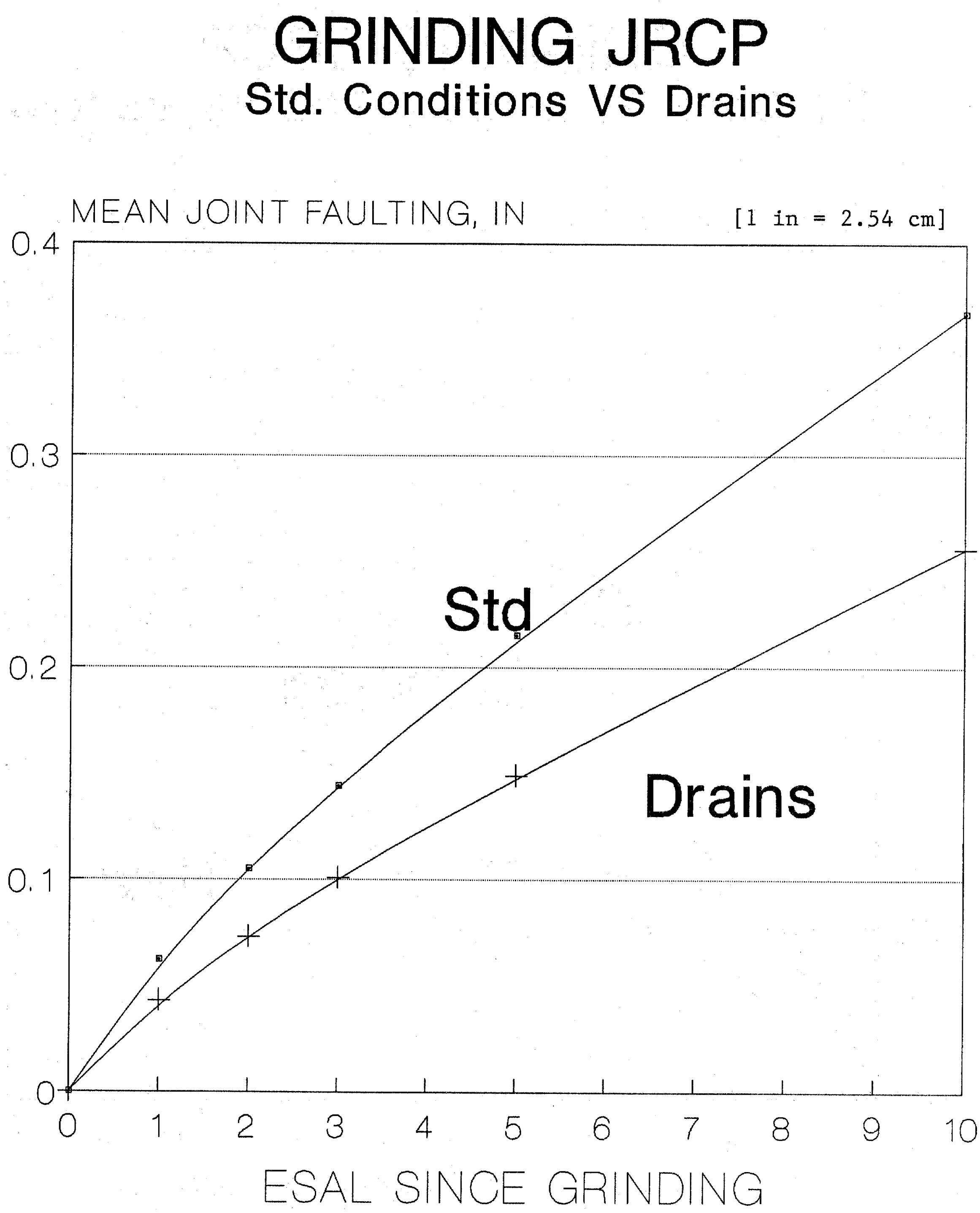


Figure 11. Faulting projections for standard JRCP and standard plus drains and PCC shoulders.

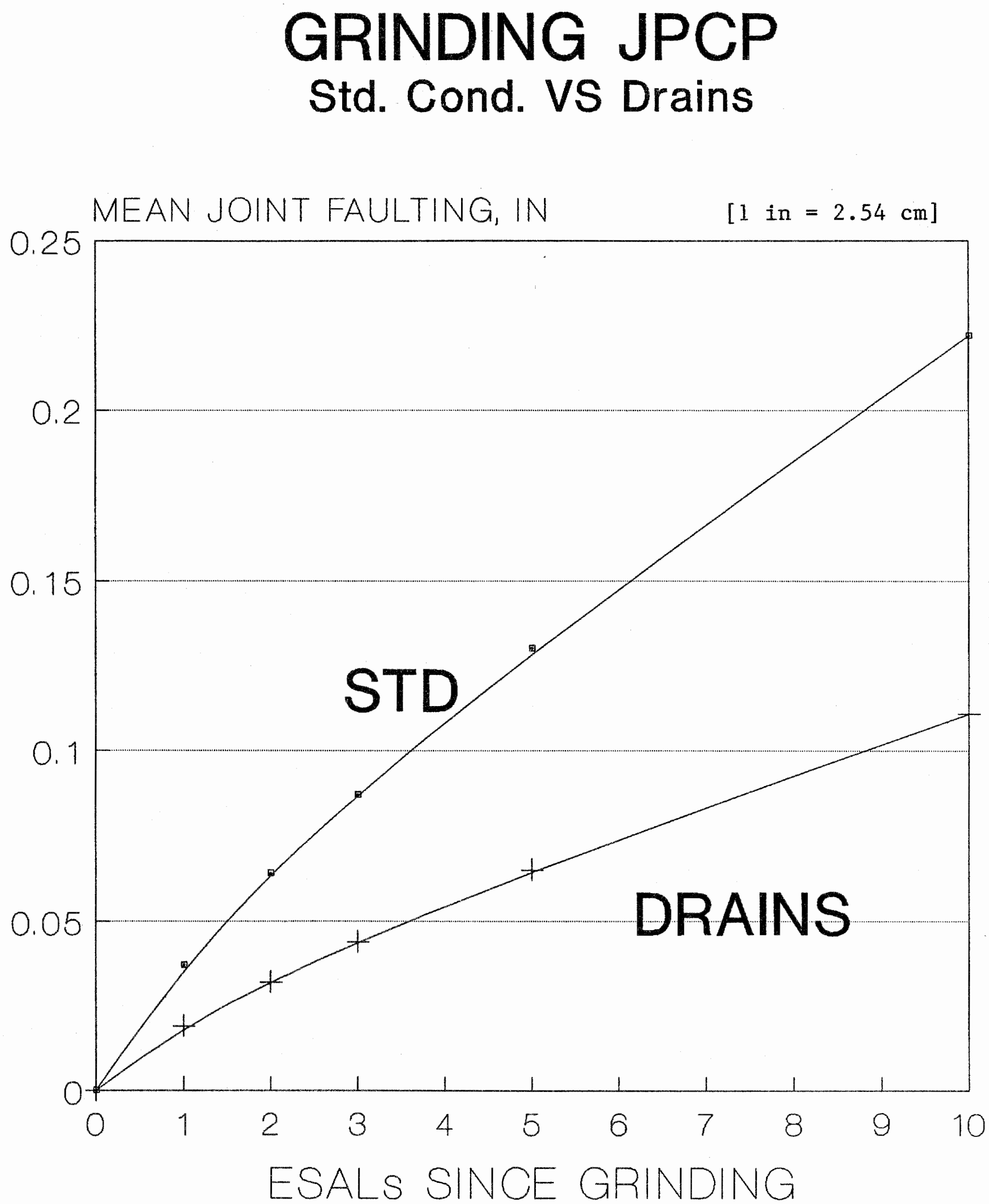


Figure 12. Faulting projections for standard JPCP and standard plus drains and PCC shoulders.

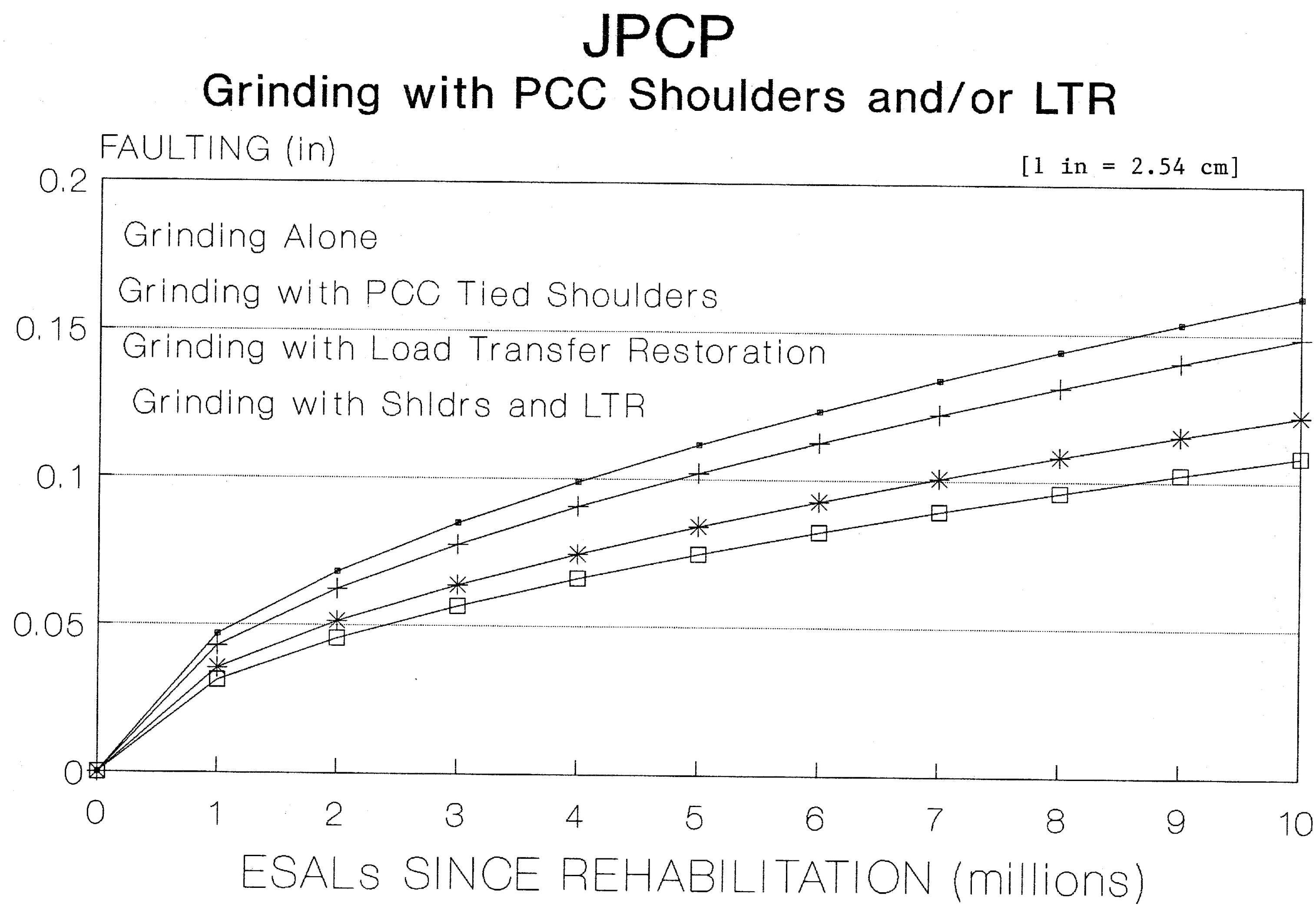


Figure 13. Illustration of predicted faulting for various restoration techniques for JPCP.

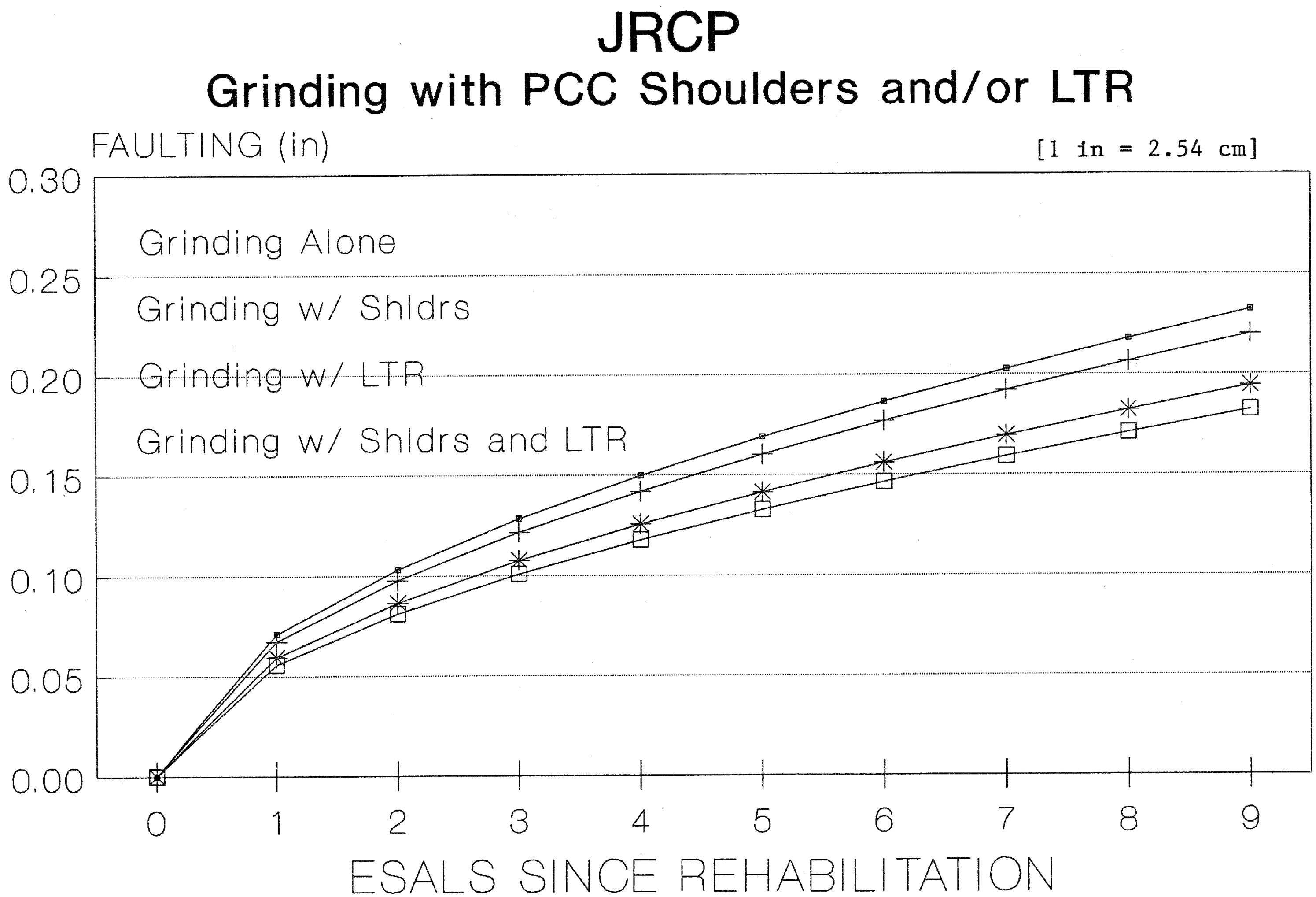


Figure 14. Illustration of predicted faulting for various restoration techniques for JRCP.

Need For Grinding

Diamond grinding is used to reprofile jointed concrete pavements which have developed any of the following conditions:

- A rough ride due to faulting or slab warping.
- Wheel ruts caused by studded tires.
- Inadequate transverse slope for drainage.
- Polishing of the surface has become excessive.

The most typical reason for grinding is excessive faulting at joints and transverse cracks in jointed PCC pavements.

Grinding should ideally be accomplished before maximum faulting exceeds 1/4 in [0.64 cm]. Georgia has developed a faulting index to describe the degree of faulting on their pavements. Each 1/32 in [0.08 cm] of faulting is adjusted to a multiple of 5. As an example, a faulting index of 15 would represent an average fault of 3/32 in [0.24 cm]. Every fourth joint is measured for faulting and an average fault per mile is determined. When pavements in Georgia reach a faulting index of 15 (3/32, or 0.094 in [0.24 cm]), the pavement will usually have some faults approaching 1/4 in [0.64 cm] and grinding is needed.

Jointed reinforced concrete pavements with long joint spacing (40+ ft [12.2+ m]) many times exhibit faulting at cracks within the panel. The mesh is broken and very little load transfer exists at these cracks due to openings caused by shrinkage due to temperature. In some cases, the doweled joints are not faulted at all, or only have minor faulting. The joints may exhibit very little horizontal movement due to dowel corrosion or other reasons. The expansion and contraction movement is therefore being accommodated by the intermediate cracks. Faulting at these cracks can be quite severe without slab breakup. Grinding can be used to remove this faulting, however, they will refault unless load transfer is restored across the crack. West Germany has been doing this with dowel bars for over 10 years. An alternative for these severely faulted cracks is full-depth repair.

Serious faulting on multilane, divided highways is usually confined to the outside or heavy traffic lane. The inside or passing lanes, in many cases, have a satisfactory profile. In these cases, grinding is needed only in the outside lane. However, if the passing lane has some faulting and considerable polishing it may be desirable to grind it also.

Rutted pavements caused by studded tires can also be reprofiled both transversely and longitudinally by diamond grinding. Transverse drainage is restored and ruts, which can fill with water and cause hydroplaning, are eliminated.

Effectiveness

The effectiveness of a grinding project depends upon its service life. The cost of grinding will be higher and the incidence of cracked or broken slabs will accelerate after grinding if the pavement is already severely deteriorated (from faulting and cracking), making initial and life-cycle costs much higher.

New-pavement smoothness, or better, can be achieved through diamond grinding. However, the tighter the specification the more the grinding work will cost.

The service life of a ground pavement depends on several factors, including:

- Rate of traffic loading (18-kip [80 kN] ESAL).
- Existing pavement design (slab thickness, joint spacing, type of base, type of subgrade soil, and subdrainage capability).
- Climate (freezing index, precipitation).
- Condition of the pavement at the time of restoration (particularly, durability of PCC ("D" cracking or reactive aggregates), amount of existing slab cracking and joint deterioration).
- Additional concurrent work to correct the problem which caused the problem.
- Performance of the existing load transfer system.

The analysis of the expected life of a grinding project is presented in section 2.4.3. The EXPEAR evaluation and rehabilitation advisory system described in volume III can be used to estimate the service life of a potential grinding project.

2.4.2 Concurrent Work

If roughness caused by faulting of the joints or cracks is evident, pumping has occurred beneath the slabs. In order to prolong the effective life of a ground pavement when pumping is evident, certain other repair and/or preventative maintenance methods must be performed at the same time. If nothing is done to reduce pumping, faulting will develop again, more rapidly.

Pumping must be reduced by any or all of the following techniques:

- Effectively sealing all joints and cracks including the longitudinal pavement centerline and edge joint.
- When pumping has advanced to medium or high severity, there is usually loss of support near the joints. This should be verified by deflection testing. If there are voids, they should be filled by subsealing to stabilize the slabs.(3,6).
- Drainage analysis may show that edge drains can also be used to reduce or eliminate pumping through rapid evacuation of water entering near the pavement edge. Recommendations for the installation of edge drains are contained in reference 3. The feasibility of installing edge drains should be carefully studied since, under certain conditions, the fines present under the pavement may be pumped out through the drainage system. The filter material that surrounds the pipe must be carefully selected to minimize the loss of fines while still permitting water flow.
- Another method of reducing the potential for pumping is to limit the amount of deflection. This can be accomplished with the installation of load transfer devices in the joints, or by using edge beams or a tied concrete shoulder. Load transfer restoration can reduce deflection by one-half and should be considered when poor load transfer exists. When used in combination with resealing and subsealing, the pumping potential will be reduced considerably.

When faulting exists, there is typically a loss of support under the leave corners of transverse joints. This can be verified through use of deflection testing and the specific joints identified.(6) If significant loss of support exists, subsealing must be applied to restore support or future problems will develop.

In addition to faulting, other problems may exist that must be corrected. Joint sealant in poor condition should be replaced, and the existing incompressibles cleaned out. Full-depth and spall repair may be required for joint and crack spalling and other localized distress.

Another important distress that impacts on the success and cost of grinding is depressions. They should be leveled by slab jacking or slab replacement prior to grinding. Trying to grind out major depressions in the pavement is not cost effective.

When diamond grinding is included in a rehabilitation project, the sequence of work is very important. Slab stabilization by subsealing, full-depth replacement, spall repair and load transfer restoration must all be completed before grinding. Resealing joints must follow the grinding operation to ensure proper sealant depth.

2.4.3 Design

<u>Condition Survey</u>

A pavement condition survey should be conducted to determine the type, severity and amount of distress present. The rehabilitation of the pavement can then be planned after an evaluation of the condition data. An important item to survey with regard to diamond grinding is the amount of cracking, pumping and faulting present. Periodic surveys will provide the information necessary to determine the increase in cracking and faulting with time in order to plan a timely rehabilitation program.

<u>Feasibility Of Diamond Grinding</u>

The feasibility of grinding from a life-cycle cost viewpoint, depends upon the following major pavement factors:

- <u>Drainage/Erosion adequacy of pavement</u>. If significant visual pumping exists (fines on shoulder, blowholes at joints), or faulting is significant (mean joint faulting greater than 0.13 in [0.38 cm]), there exists a serious subdrainage and erosion problem. Restoration will result in an increased life-cycle cost. The presence of significant faulting is a clear indication of serious pumping problems, which means that substantial additional restoration work will be required to reduce future pumping of the ground pavement.

  The amount of future faulting can be estimated using the model given in section 2.4.2, both without and with additional concurrent work such as load transfer restoration, subdrainage and tied PCC shoulders. The evaluation and rehabilitation advisory system (EXPEAR) can be used to estimate the amount of faulting for different restoration alternatives.

- <u>Structural adequacy of pavement</u>. The presence of transverse slab cracking (all severity levels for JPCP and deteriorated cracks for JRCP) and corner breaks is an indication of structural deficiency of the pavement. If historical slab cracking data is available, observating the rate of crack development would provide clear indications of structural adequacy. Another procedure is use the EXPEAR system to project the amount of future cracking for the pavement under consideration. If there is a serious structural deficiency in the existing pavement, the future slab cracking will be high, which will reduce the life of the restoration project.

- Hardness of Aggregate. There are some existing concrete pavements that have extremely hard aggregates. While almost any pavement can be ground, the cost of grinding slabs containing extremely hard aggregates is very high, and may greatly increase the cost of this restoration technique. This should be determined to help estimate proposed rehabilitation costs.

- Durability of PCC. Any pavement exhibiting medium to high severity "D" cracking or alkalai aggregate reaction should not be rehabilitated through grinding.

The EXPEAR system can estimate future performance of the ground pavement in terms of faulting, cracking, joint deterioration and present serviceability rating, based upon future traffic projections and other current restoration work. Knowing the future life and cost of the restoration, the equivalent uniform annual cost of the grinding can be determined and compared to other alternatives. These estimates are based on performance evaluations of many grinding projects throughout the U. S. having designs that were constructed in the 1960's and 1970's.

Cost Of Grinding

The cost of grinding is primarily dependent upon the amount of material to be removed and the hardness of the aggregate. On a typical project the cost of grinding for soft aggregate is in the range of $2.00 to $3.00/sq yd [$2.4 to $3.6 /sq m], for medium hardness aggregate, $3.00 to $5.00 /sq yd [$3.6 to $6.0 /sq m], and for hard aggregate $5.00 to $8.00 /sq yd [$6.0 to $9.6 /sq m] for 1984. Costs are also affected by the size of the project, labor rates, traffic control procedures (roadway closed or with traffic in adjacent lane) and the degree of smoothness specified. Due to equipment advances, the cost of grinding has remained constant for several years.

Pavement Profile

When designing a project involving diamond grinding, the existing pavement profile is useful for estimating the cost of the work. There are several methods available to measure either the actual profile or a "relative" profile of the existing pavement. Some of the equipment includes the California or Rainhart profilographs, the Mays ride Meter and the K. J. Law 690 DCN profilometer.

The profilograph or profilometer type devices make a trace of the pavement surface. The trace is normally taken in the wheel paths of the traffic lane under consideration. The traces indicate the amount of grinding necessary and the location of roughness. These charts can be used by contractors to estimate the amount of material to be removed. If rutting wear exists in the wheel paths, the amount of grinding can be underestimated. Transverse profiles are also needed.

The Mays Ride Meter and other similar types of response devices provide a relative profile (the difference between the axle and vehicle body movement). The equipment must be kept in calibration and recommended test procedures strictly followed in order to reduce the variability of this type of device.

Friction Resistance

For legal reasons, specifications do not call for a specific level of friction resistance of any type of pavement surface. The texture developed by grinding produces a good friction factor immediately after grinding.(10,15) The ridges produced improve the surface macrotexture and provide an escape route for moisture under a tire.

There is evidence that the texture produced by grinding will wear down under heavy traffic, especially for softer aggregates such as limestone.(2) Thus, it is important to maximize the land area between grooves. The ridge width can be increased by increasing the spacing between blades as described in section 2.4.4.

The use of the ASTM ribbed test tire (E501) may not provide a complete evaluation of the friction resistance of this type of texture since this ribbed tire is not sensitive to macrotexture differences.(9) The use of both the ribbed E501 tire and the smooth E524 tire should provide more complete information on the friction properties of the pavement after grinding.(9,15)

2.4.4 Construction

Equipment

The degree of joint faulting or roughness that can and should be removed in a cost-effective manner is changing with current equipment and blade developments. Equipment is available or being developed which can make grinding a more viable option for pavement with a greater degree of roughness. These developments include larger and more powerful equipment (6-ft [1.8 m] cutting width), different types of segmental cutting heads and blade development to increase the life of blades.

Procedures

Diamond grinding will result in retexturing the pavement surface to improve the friction number after grinding. Blade spacing in the cutting head can be varied to improve the life and friction factor of the texture. When grinding aggregates susceptible to polishing, the blade spacing must be wider to provide more area between the grooves. The grinding chip thickness (chip thickness of pavement broken off between blades), measured at its thickest point, should be 0.080 in [0.203 cm] minimum and have an average thickness of 0.100 in [0.254 cm]. For the harder aggregates not subject to polishing, the minimum chip thickness should be 0.065 in [0.165 cm] and an average of 0.080 in [0.203 cm].

The International Grooving and Grinding Association recommends that for hard aggregates, between 53 and 57 diamond blades per ft [174 and 187 per m] be used, and for soft aggregates, between 50 and 54 blades per ft [164 and 177 per m] be used.(12)

Water is used to cool the cutting head when diamond grinding. This slurry must be vacuumed from the surface and pumped into a tank with baffles, or deposited into the grassed slopes. Slurry can be deposited directly on grass shoulders from the grinding machine. This is the most economical solution, and the slurry is not detrimental to vegetation. Where this is impossible, in urban areas or for other reasons, a suitable means of disposal should be provided.

Much of the grinding work on interstate type facilities has been done under single lane closure with traffic carried in the adjacent lane. This type of traffic control results in increased construction costs and increased risk to construction workmen. A reduced construction zone speed limit should be strictly enforced by highway patrol personnel. These services could be a bid item under the rehabilitation contract.

In urban areas, avoiding interference with traffic flow during rush hours to minimize public inconvenience, may be necessary. In this case, the work period may be confined to off-peak traffic hours (i.e. 8 p.m. to 6 a.m.). It would be advantageous from the standpoint of costs and work period required, to close sections of the entire roadway involved and route traffic over parallel service roads or an adjacent street. Tight completion schedules can be used to expedite work when roadway closures are specified. Closing a single lane with traffic on both sides should be avoided.

2.4.5 Preparation Of Plans And Specifications

The following information would be of value to a grinding contractor and should be included in the bid documents:

- Year pavement was constructed.
- Source of both the coarse and fine aggregate used in the concrete slab.
- Transverse joint spacing and sealant used.
- Wheel rut depth if more than 1/16 in [0.16 cm].
- Pavement design: plain jointed, reinforced jointed or continuously reinforced concrete pavement. Evidence of any steel reinforcement near the surface.
- Type of traffic markers and replacement requirements. A pay item should be set up for temporary and/or permanent marking required.
- Profile of existing pavement surface.

The working time should be Stated in either working or calendar days. The hours per day should also be Stated if restrictions are imposed on the contractor's working time due to traffic volume considerations, noise restrictions, etc.

Grinding limits should be clearly defined on the plans and should show transition or stop lines at bridges and ramps. Areas to be ground should be clearly marked.

Grinding production is typically 50 machine hours per lane mile [31 per lane km], but this will vary considerably with aggregate hardness and the roughness of the pavement.

When specifying acceptance testing for smoothness, the test equipment should be listed along with the method or procedures to be followed in acceptance testing. Test methods commonly used for new pavement construction can be used for diamond grinding.

The specifications should also define who will run the acceptance tests and when these tests will be run.

Any noise limitations on equipment should be clearly defined. A level of 95 dba at 50 ft [15.2 m] is common and 86 dba at 50 ft [15.2 m] is attainable.

When grinding a pavement, isolated low areas from original construction occasionally are present. Specifications recognize this and usually require 95 percent coverage in any 3-ft by 100-ft [0.9 m by 30.5 m] test area. Isolated low spots less than 2 sq ft [0.19 m^2] in area should not require texture if lowering the cutting head is required. The maximum overlap between passes should be 2 in [5.1 cm].

If other work in addition to grinding is to be accomplished, the sequence of operation should be specified (e.g., joint resealing after grinding, subsealing and full or partial depth repair before grinding).

Various State specifications and guide specifications are available for consideration by agencies considering diamond grinding work.(5,6,13,14).

2.5 CONCLUSIONS AND RECOMMENDATIONS

1. Overall effectiveness. Diamond grinding has been successful in producing a very smooth ride and extending the service life of jointed concrete pavements. A number of sections (about 1 out of 5 projects) that were ground were apparently structurally inadequate for the traffic level and either had a large amount of cracking at the time of grinding or developed cracking after grinding. Diamond grinding does not increase the structural capacity of a pavement, and thus any pavement with substantial amounts of deteriorated cracks will continue to crack after grinding.

2. Transverse Cracking. About 21 percent of the ground sections showed a large amount of deteriorated transverse cracking (over 825 ft per lane mile [156 m per lane km]). This is for pavements having an average of 4 years life after grinding and 2 million 18-kip [80 kN] accumulated ESAL since grinding was completed. Fifty-seven percent had minor amounts of deteriorated cracks.

3. Longitudinal Cracking. About 90 percent of the ground sections showed little or no deteriorated longitudinal cracking. Only 2 percent of the sections showed a serious amount of longitudinal cracking (greater than 1000 ft per mile [189 m per lane km]).

4. Corner Breaks. About 84 percent of the ground sections showed minor corner breaks. Only 6 percent showed a serious amount of corner breaks (greater than 25 per mile [16 per km]).

5. Faulting of Transverse Contraction Joints. The rate of faulting after grinding generally is higher than for newly constructed pavements if no other restoration work is accomplished. However, this increased faulting can be largely overcome by reducing the pumping potential through concurrent work such as load transfer restoration, sealing joints, tied PCC shoulders and subdrainage. Some key factors that affect faulting were determined as follows:

   - Future Traffic -- The amount of traffic loadings after diamond grinding (as measured by accumulated 18-kip [80 kN] equivalent single axle loadings) has a large effect on the amount of faulting that develops. As traffic loading begins after grinding, faulting develops rapidly at first and then levels off, following a similar form as new pavements.(1)

   - Existing Pavement Design -- The thicker the existing slab or the presence of a stabilized base, the less the amount of future faulting. The shorter the existing joint spacing, the less the amount of future faulting.

- Drainage -- The presence of a granular subgrade will reduce the amount of faulting greatly. The placement of edge drains will reduce the amount of faulting after grinding.

- Climate -- The colder the climate where the pavement is located, the greater the amount of faulting after grinding.

- Tied Concrete Shoulder -- The placement of a tied concrete shoulder will reduce the amount of faulting after grinding.

- Load Transfer Restoration -- The placement of dowels to restore load transfer at transverse joints and working cracks will reduce the amount of future faulting after grinding.

- Prediction of Faulting After Grinding -- The predictive faulting model can be used to approximately estimate the future amount of faulting for a given pavement structure after grinding. The effect of edge drains, load transfer restoration and tied concrete shoulders can also be estimated to help determine the feasibility of diamond grinding.

6. Wearout of Grinding Texture. The surveys revealed that there was some wear of the texture in the wheel paths as compared to out of the wheel paths. The rate of wearout and the factors involved could not be determined. It is likely that the hardness of the aggregate, the level of traffic, and the original land area texture width are major factors involved. It is important to maximize the land area between grooves. This is done by increasing the spacing between blades for softer aggregates, while still providing adequate grooves for drainage.

CHAPTER 3

RESTORATION OF LOAD TRANSFER

3.0 RESEARCH APPROACH

Many jointed concrete pavements have been constructed with no mechanical load transfer devices across joints (e.g. no dowels) and significant faulting has occurred. Many others have dowels, but they have become loose and faulting has developed after being heavily trafficked. In addition, many transverse cracks have become working cracks and developed faulting and spalling due to poor load transfer. In an effort to extend the life of inservice concrete pavements which exhibit poor load transfer, highway agencies have begun to utilize various devices to restore joint or crack load transfer to an acceptable level to prevent further faulting, spalling and reduce deflections and pumping. Even if asphalt concrete overlays are placed, poor load transfer leads to rapid deterioration of transverse joint reflection cracks.

This study deals with the field performance of these various load transfer restoration devices on a nationwide basis. The effectiveness of these devices has been evaluated in terms of the amount of faulting associated with these rehabilitated joints and cracks. Load transfer was measured on one major load transfer restoration project as will be described.

The overall goal of this study is to improve the design and construction of load transfer restoration devices. The development of predictive models to forecast future faulting of jointed concrete pavements with load transfer restoration is also an objective. Field performance analysis of the devices should also lead to the improvement of current construction guidelines for these various load transfer devices.

This report includes four-load transfer devices:

- Retrofit conventional round steel dowels placed in slots.
- Double-vee shear devices marketed by Dayton Superior Corporation.
- Figure-eight devices utilized in a Georgia project, which were originally experimented with in France.(19)
- Miniature I-beam devices utilized in New York.(18)

3.1 DATABASE AND DATA COLLECTION

The load transfer restoration database incorporates both design, construction and performance variables for thirteen uniform sections. These variables are in addition to the original pavement design, traffic and climatic variables summarized in volume IV. Table 7 lists these load transfer restoration variables. Along with monitoring the performance of the device itself, some measure of joint and sealant distress was also recorded. Also, faulting measurements were taken at 369 restored joints or cracks; while device performance ratings were taken on 1,525 individual devices.

3.1.1 General Project Description

Thirteen uniform sections were located in nine States: Colorado, Georgia, Illinois, Louisiana, New York, Ohio, Oklahoma, Pennsylvania and Virginia. These uniform sections were broken down into 20 sample units that were up to 1000-ft [305 m] long, where possible (see figure 15).

Table 7. Load transfer restoration database design variables.

LOAD TRANSFER RESTORATION

DATABASE DESIGN VARIABLES

- Project Identification Number and Sample Unit.
- Load Transfer Device Type.
- Frequency of Installation.
- Lane Restored by Load Transfer.
- Number and Location of Devices along Joint/Crack.
- Diameter and Length of Retrofit Dowel Bars.
- Backfill Material and Bonding Agent for Slot or Core Hole.

DATABASE PERFORMANCE VARIABLES

Overall Distress

- Project Identification Number.
- Sample Unit Number, Length, and Present Serviceability Rating.
- Foundation of Sample Unit (cut,fill,at grade).
- Condition of Drainage Ditches and Subsurface Drainage.
- Joint/Crack Station.
- Transverse Joint Type or Crack.
- Load Transfer Device Type.
- Lane Restored by Load Transfer.
- Number of Devices along Joint/Crack.
- Device Performance:
    - No failure.
    - Material failure.
    - Device failure.
    - Debonding on approach side.
    - Debonding on leave side.
    - Debonding on both sides.

Joint Distress

- Transverse Joint Spalling on Approach and/or Leave Side.
- Corner Spalling on Approach and/or Leave Side.
- Corner Breaks on Approach and/or Leave Side.
- Pumping.
- Joint/Crack Faulting.
- Joint/Crack Width.
- Durability Cracking.
- Reactive Aggregate.

Sealant Condition

- Sealant Absent.
- Cohesion Failure.
- Adhesion Failure.
- Sealant Extrusion.
- Sealant Oxidation.
- Incompressibles in Joint.

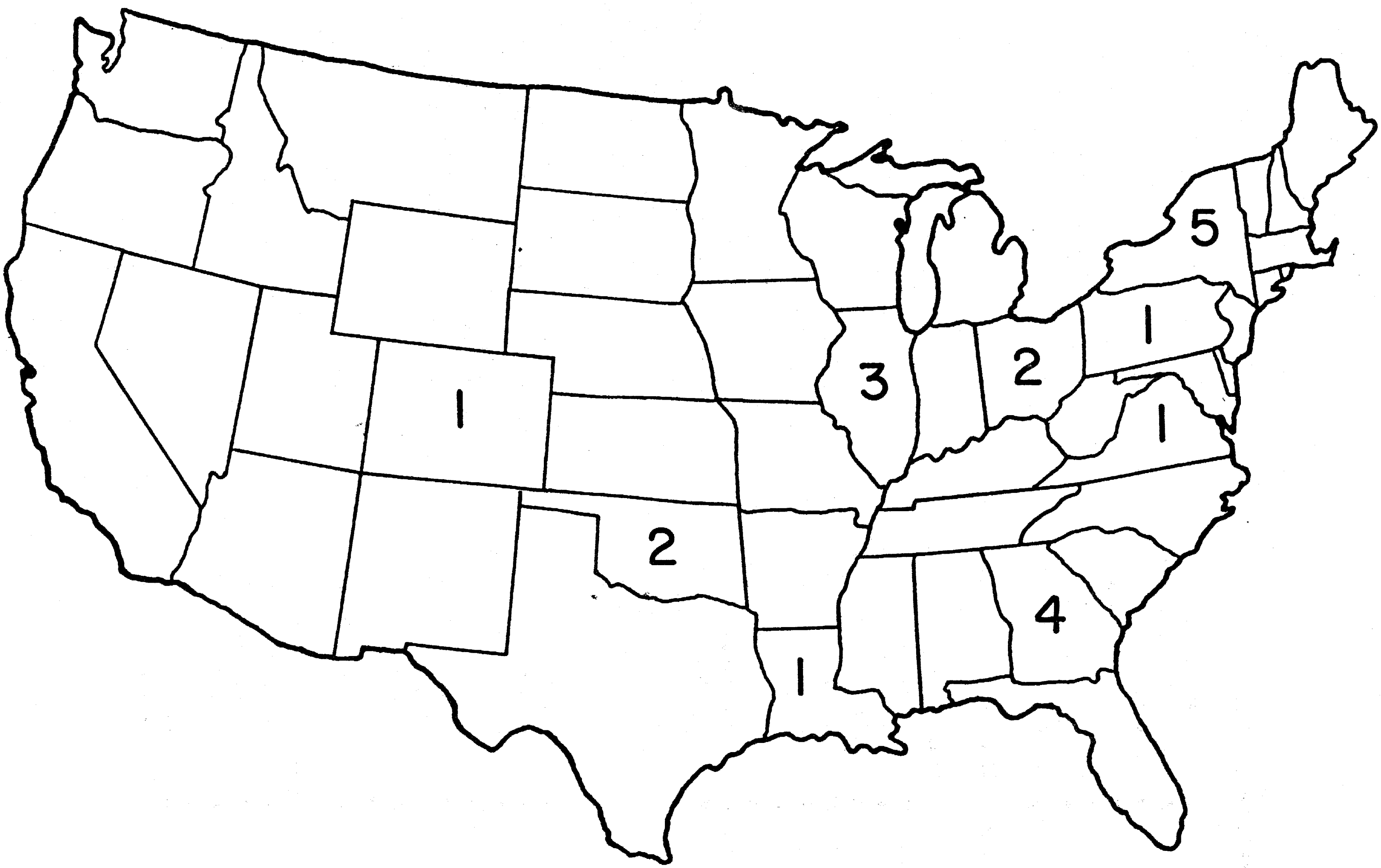


Figure 15. Location of load transfer restoration sample units by State.

3.1.2 Load-Transfer Restoration Design Variation

Load-transfer restoration was placed and evaluated at five different locations in the pavement:

- Regular contraction joints on 15- to 100-ft [4.6 to 30.5 m] joint spacings (predominant location).
- Full-depth repair approach joints.
- Full-depth repair leave joints.
- Pressure relief joints.
- Transverse cracks.

The devices were mainly placed in the outer traffic lane; however, some were installed in the inner traffic lane as well. From one to eight devices were installed at any given joint or crack. The restoration projects had been in service from 1 to 9 years at the time of survey.

3.1.3 Traffic and Climatic Variation

In terms of traffic loadings and climatic effects, the devices have withstood from 0.3 million to 5.9 million 18-kip (80 kN) equivalent single axle loads (ESALs) while in service. Annual loadings ranged from 0.3 million to 2.0 million ESALs per year. The projects were located in five of the nine climatic regions as defined by Carpenter (see figure 16).(3) The Corps of Engineers Freezing Index varied from 0 to 550.

3.1.4 Performance Variation

Faulting measurements ranged from flat to 0.36 in [0.91 cm] with the majority of the joints having less than 0.07 in [0.18 cm] of faulting at the time of survey. All of the projects which involved load transfer restoration also had diamond grinding performed in the same year. With respect to device performance, at any joint, anywhere from 0 to 8 devices were in good condition (e.g. showing no visible signs of failure) at the time of survey. Deflection load transfer associated with this variation in performance was not measured.

3.2 DATA COLLECTION

The database is comprehensive containing as many projects as were available or that could be included within available resources. This was done to provide a wide range of data to facilitate analysis of performance and the development of performance models. The projects included in the database are believed to be most of the highway pavements with load transfer restoration in existence today within the United States. These pavements were surveyed between June 1985 and July 1986.

There were five basic data sets that were deemed necessary for the development of life prediction models and for analysis aimed towards the development and improvement of design and construction procedures. These included:

- Field condition data.

- Original pavement structural design and construction and subgrade soil classification.

- Rehabilitation design factors.

- Historical traffic volumes, classifications and accumulated 18-kip [80 kN] equivalent single axle loadings.

- Environmental data.

CLIMATIC ZONE FACTORIAL

		PRECIPITATION		
		WET	WET -DRY	DRY
TEMPERATURE	FREEZE	5	0	1
	FREEZE -THAW	2	2	0
	NO FREEZE	3	0	0
	TOTAL	10	2	1

NOTE: A total of 13 uniform sections were evaluated through condition surveys.

Figure 16. Climatic zone factorial for load transfer restoration uniform sections.

The data sources and procedures used in the collection of each are described in volume IV.

3.3 FIELD PERFORMANCE AND EVALUATION

3.3.1 Field Performance

The performance of individual load-transfer restoration devices was only evaluated in terms of visual characteristics. As a result, none of the load transfer devices were rated as having a "device failure" since the devices themselves cannot be seen. Some of the devices may well have failed; however, these failures are likely manifested in the other failure modes. It is interesting to note that the retrofit dowel bars and the miniature I-beam devices have similar performance characteristics. The same can be said for the Double-vee shear and Figure-eight devices. This is probably due to the fact that both device pairs rely on similar mechanisms for load transfer restoration. It should be noted that some of these devices and their representative construction procedures have been modified and hopefully improved since these installations. For example, the Double-vee shear device construction procedure now recommends grooving of the core walls and precompression of the load transfer device itself to improve performance; whereas all of the shear devices in this study were uncompressed and ungrooved. The Florida Interstate 10 experimental study is evaluating the effectiveness of these construction modifications.

3.3.2 Retrofit Dowel Bar Performance

The performance of the retrofit round, steel dowel bars, as shown in figure 17, was measured in terms of two criteria:

- Faulting readings at 72 joints.
- Visual device evaluations of 515 devices.

The mean faulting reading of the 72 joints restored with retrofit dowel bars was 0.04 in [0.10 cm]. This faulting occurred after an average of 2.62 million ESALs had loaded the pavements over an average of 3.8 years of service. This mean faulting lies well below the failure criteria for faulting of 0.13 in [0.38 cm], the point where faulting affects rideability significantly.(52)

Of the 515 retrofit dowel bar load transfer devices inspected, 507, or better than 98 percent, of the devices were in good condition (see figure 18). The most prominent mode of failure identified was material failure (located at one percent or 5 devices), where the backfill matrix had been cracked or become loose and dislodged by traffic. Less than one percent of the joints were debonded on the approach, leave or approach and leave sides. None of the joints restored with retrofit dowel bars exhibited device failure or multiple modes of failure. Multiple modes of failure refers to the existence of two or more of the failure mechanisms listed in table 8 at any one device. The one exception to this category is the debonding at both the approach and leave sides of the same device. This was not recorded as a multiple mode of failure. Similarly, if a joint exhibited both debonding on the approach and leave sides of the same device, then this was recorded in one category as such, and not reflected under the individual failure modes of debonding approach side and debonding leave side so as not to record the failure twice.

3.3.3 Double-vee Shear Device Performance

The performance of the Double-vee shear devices (see figure 19) was measured in terms of two criteria:

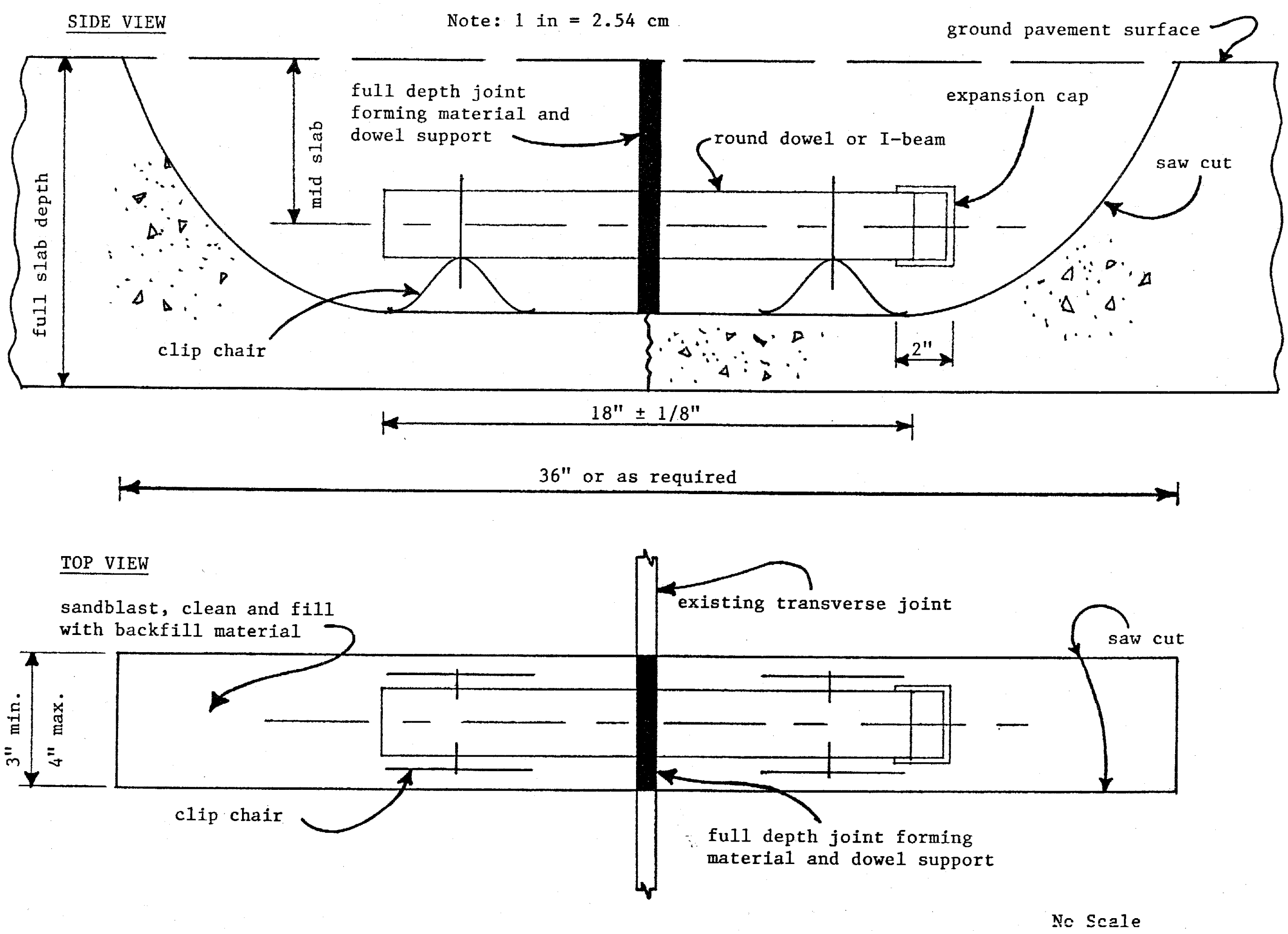


Figure 17. Diagram of retrofit dowel or I-beam device and installation.

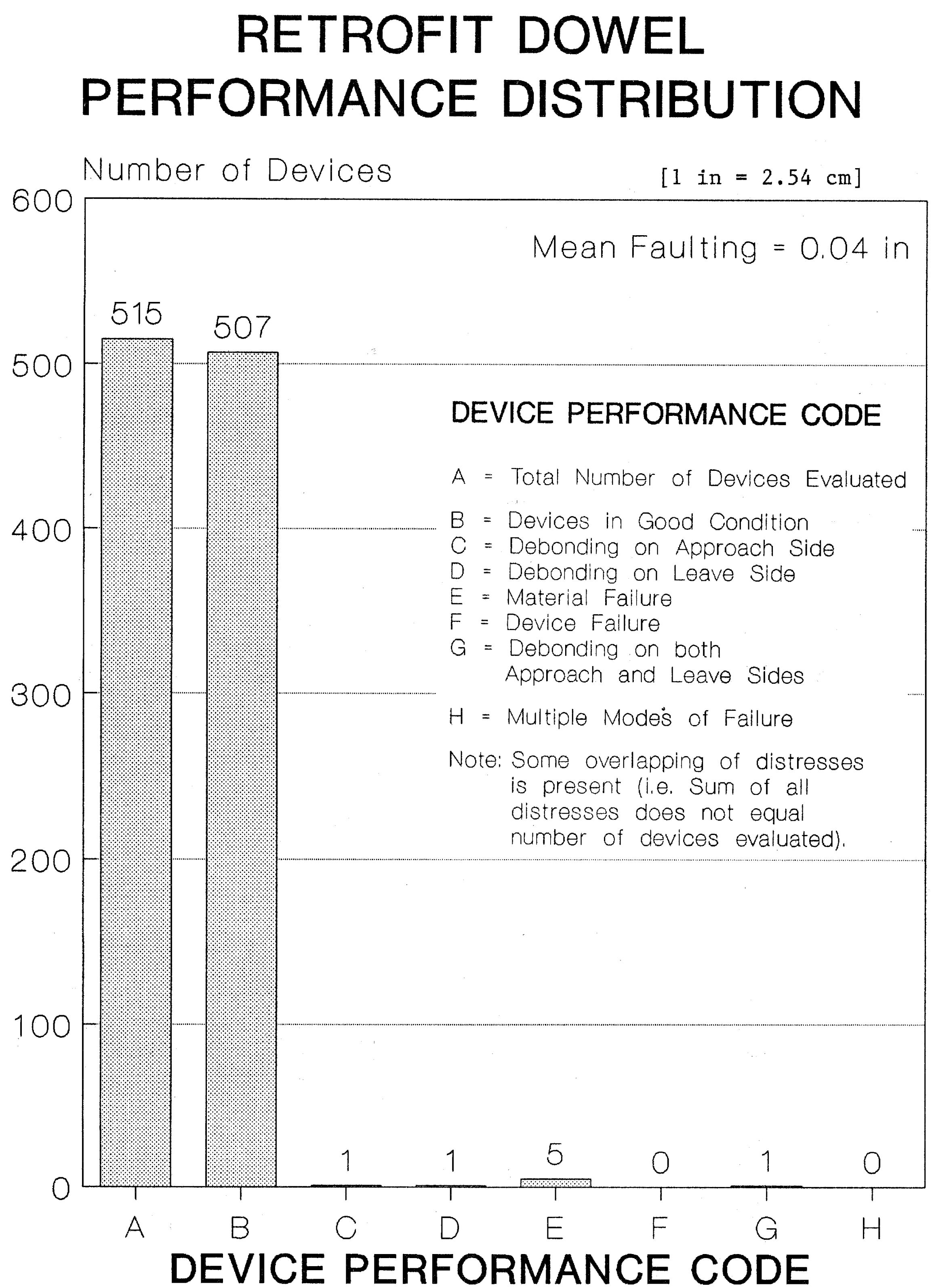


Figure 18. Distribution of retrofit dowel performance.

Table 8. Performance summary for all devices evaluated.

	Dowel Bars	Double Vees	Figure Eights	I-Beams
Number of Devices	515	810	36	164
	Percentages			
Good Condition	98	72	75	99
Debonding Approach	<1	6	8	0
Debonding Leave	<1	4	6	0
Material Failure	1	9	8	1
Device Failure	0	0	0	0
Debonding Approach and Leave	<1	13	6	0
Multiple Modes of Failure	0	4	3	0
Average Faulting, ins.	0.04	0.07	0.08	0.13

Note: 1 in = 2.54 cm

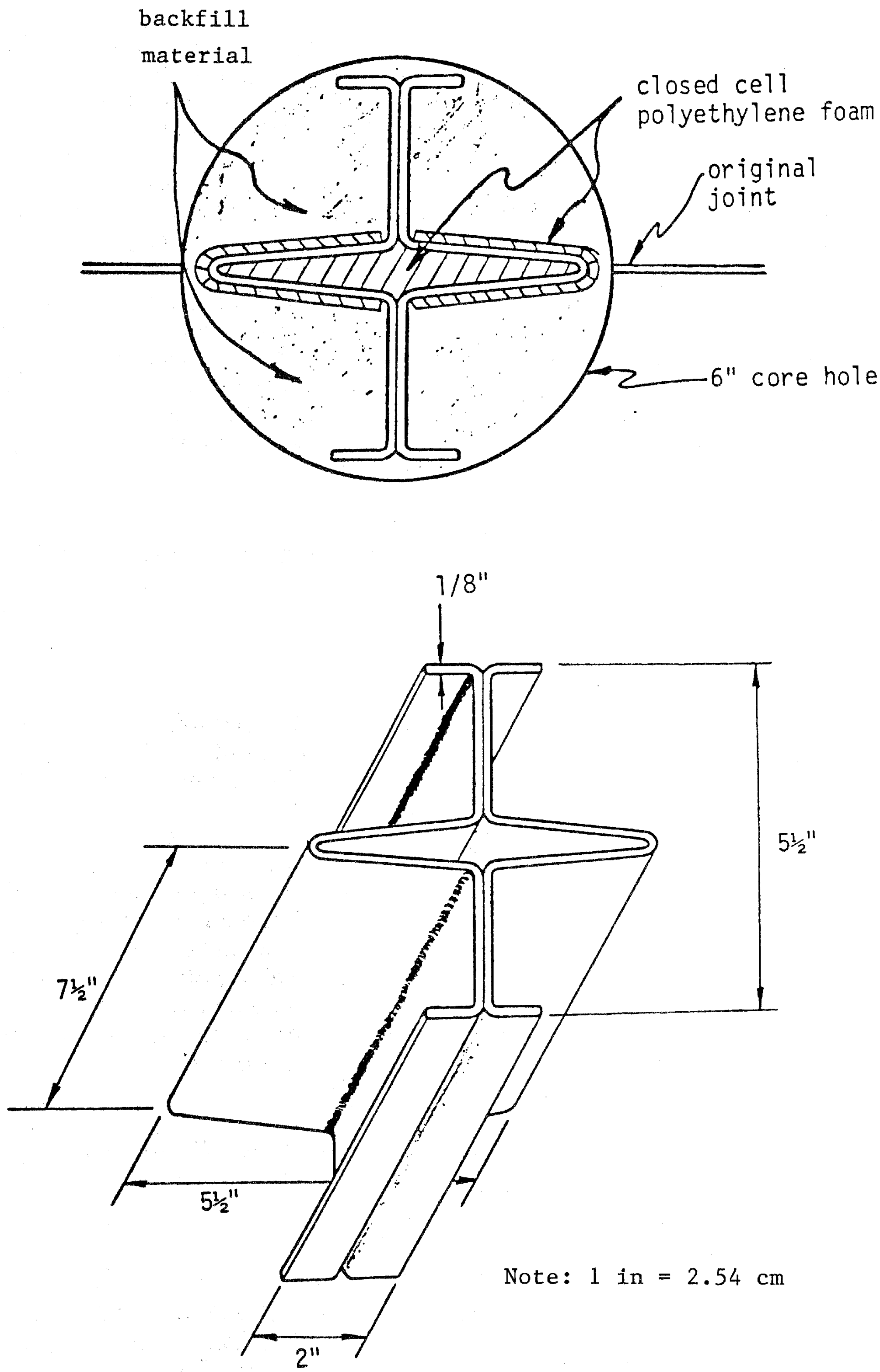


Figure 19. Diagram of double-vee shear device and installation.

- Faulting readings at 260 joints and cracks.
- Visual device evaluations of 810 devices.

The mean faulting reading of the 260 joints restored with shear devices was 0.07 in [0.18 cm]. This faulting occurred after an average of 2.55 million ESALs had loaded the pavement over 2.5 years of service, on the average. This mean faulting is approximately one-half of the failure criteria for faulting of 0.15 in [0.38 cm].

Of the 810 uncompressed, ungrooved shear load transfer devices inspected, 583, or 72 percent, of the devices were in good condition (see figure 20). The most prominent mode of failure identified was debonding on both the approach and leave sides of the same device which was found at 108, or 13 percent, of the devices. As stated previously under the discussion of retrofit dowel bars, this failure mode was recorded separately from the individual modes of debonding failure. Again, the Florida study is evaluating the use of device precompression and core wall grooving as remedies to this debonding mode of failure. None of the joints restored with shear devices exhibited device failure. Multiple modes of failure were identified at 4 percent of the devices. Multiple modes of failure refers to the existence of two or more of the failure mechanisms listed in table 8 at any one device.

3.3.4 Miniature I-beam Device Performance

The performance of the miniature I-beam devices, as shown in figure 17, was measured in terms of two criteria:

- Faulting readings at 23 joints.
- Visual device evaluations of 164 devices.

The mean faulting reading of the 23 joints restored with miniature I-beams was 0.13 in [0.33 cm]. This faulting occurred after an average of 4.01 million ESALs had loaded the pavement over 2.0 years of service, on the average. This mean faulting lies very close to the failure criteria for faulting of 0.15 in [0.38 cm].

Of the 164 I-beam load transfer devices inspected, 162, or better than 98 percent, of the devices were in good condition (see figure 21). The most prominent mode of failure identified was material failure (located at about 1 percent or 2 devices), where the backfill matrix had been cracked or become loose and dislodged by traffic. None of the devices were debonded on the approach, leave or both approach and leave sides. Also, none of the joints restored with I-beams exhibited device failure or multiple modes of failure. Multiple modes of failure refers to the existence of two or more of the failure mechanisms listed in table 8 at any one device.

3.3.5 Figure-eight Device Performance

The performance of the Figure-eight devices (see figure 22) was measured in terms of two criteria:

- Faulting readings at 8 joints.
- Visual device evaluations of 36 devices.

The mean faulting reading of the 8 joints restored with Figure-eight devices was 0.08 in [0.20 cm]. This faulting occurred after an average of 5.45 million ESALs had loaded the pavement over 9.0 years of service, on the average. This mean faulting is approximately one-half of the failure criteria for faulting of 0.15 in [0.38 cm].

Of the 36 Figure-eight load transfer devices inspected, 27, or 75 percent, of the devices were in good condition (see figure 23). The most prominent failure modes identified were debonding on the device approach side and material failure. Both of

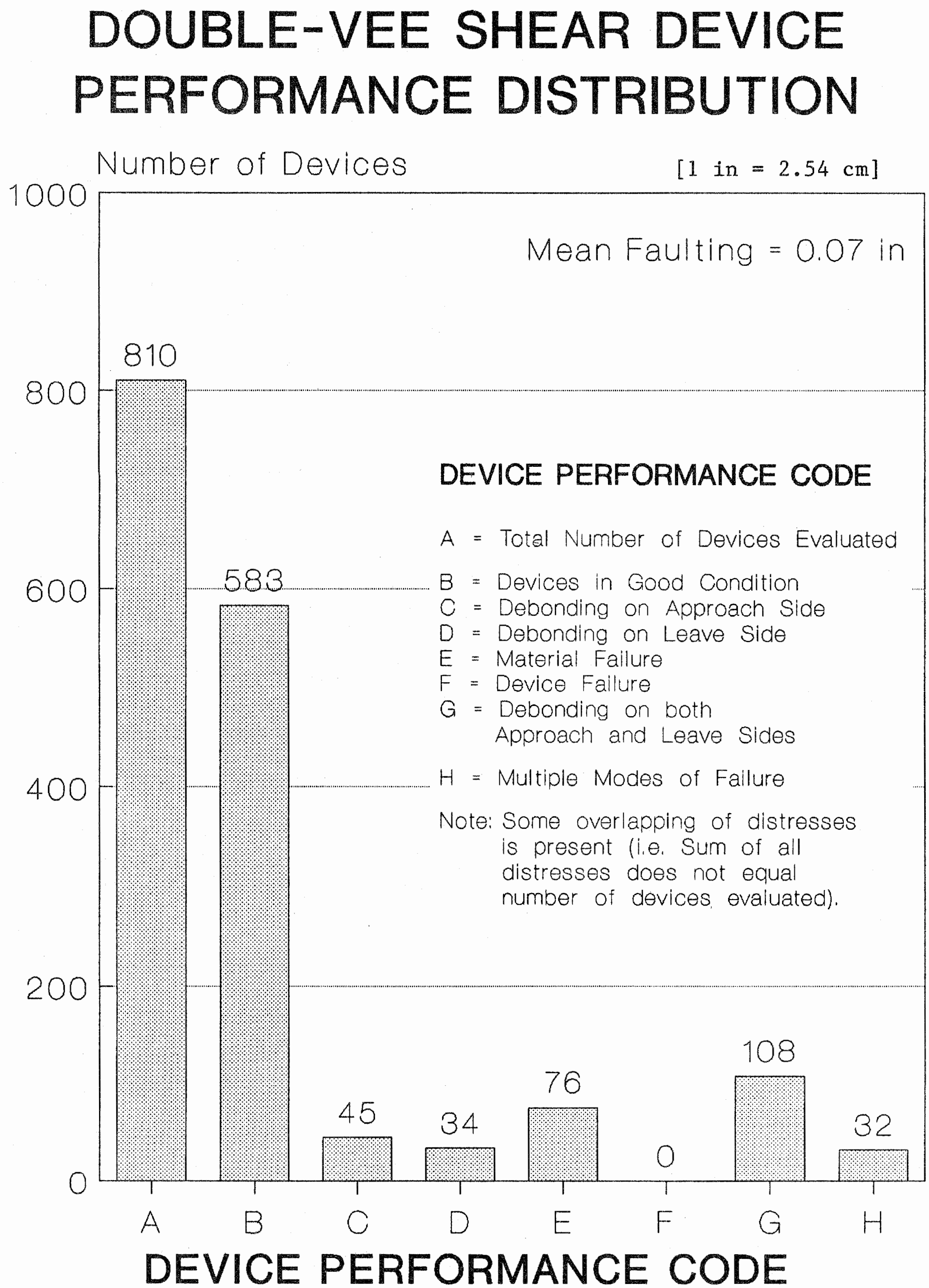


Figure 20. Distribution of double-vee shear device performance.

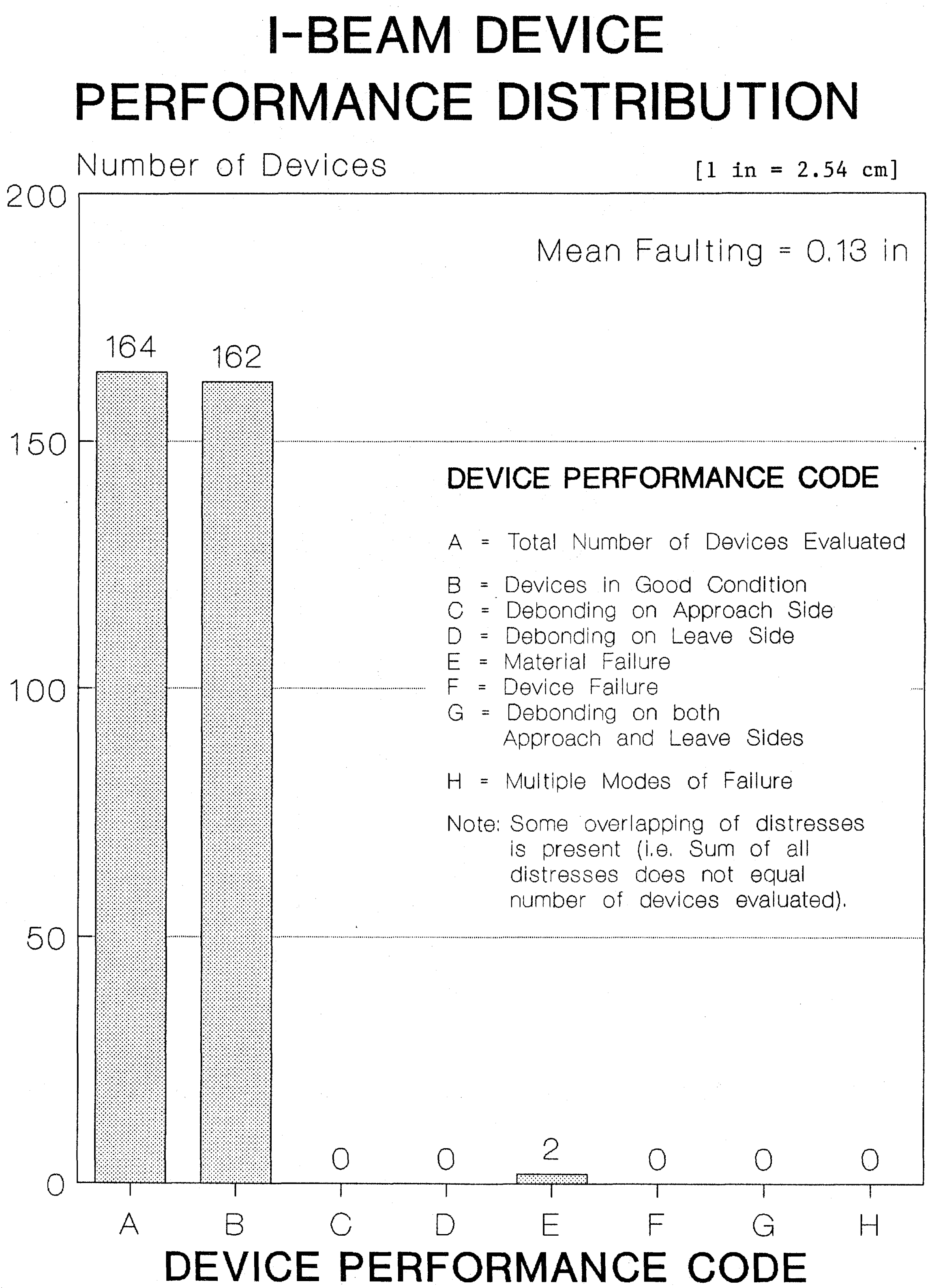


Figure 21. Distribution of I-beam device performance.

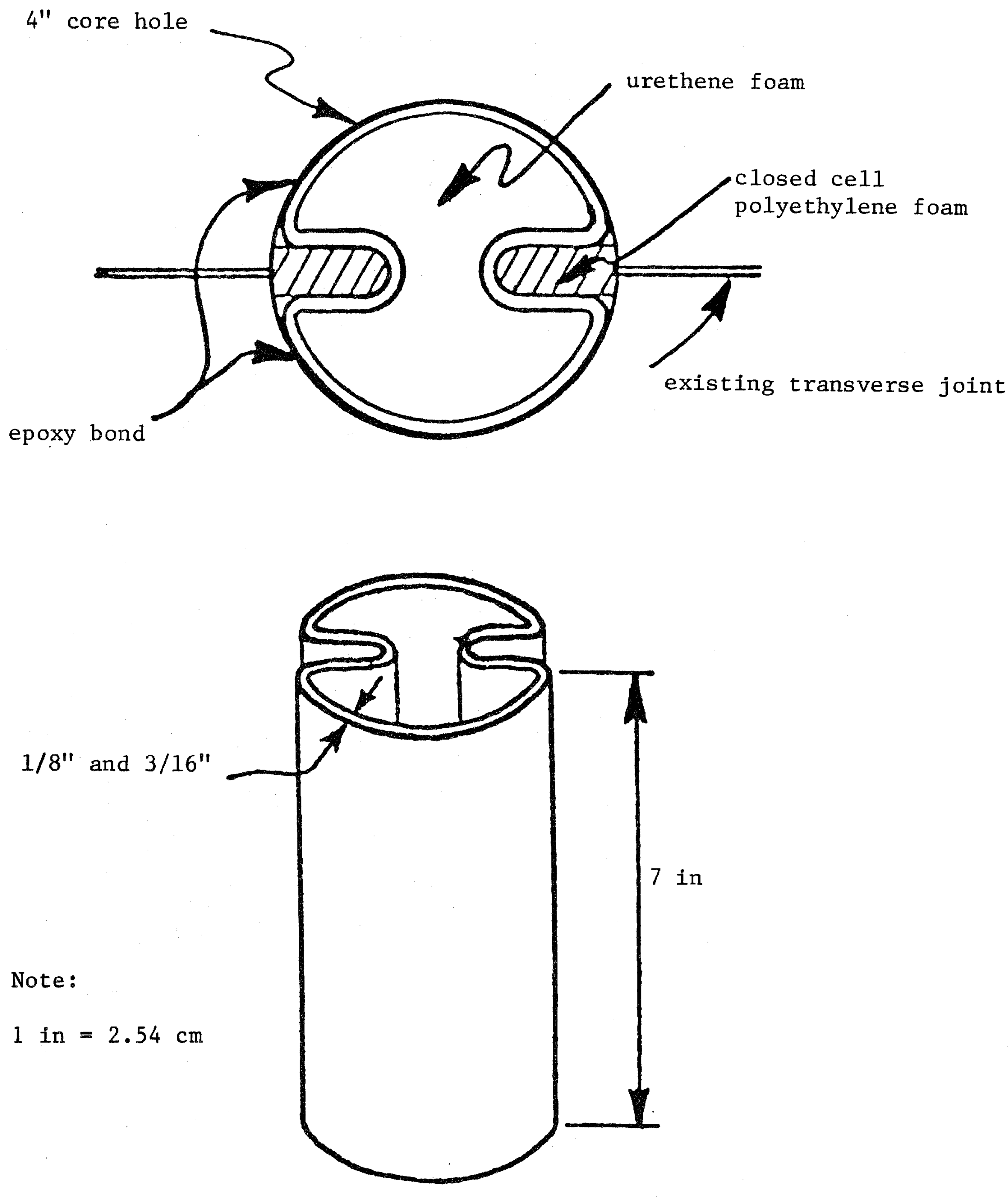


Figure 22. Diagram of figure-eight device and installation.

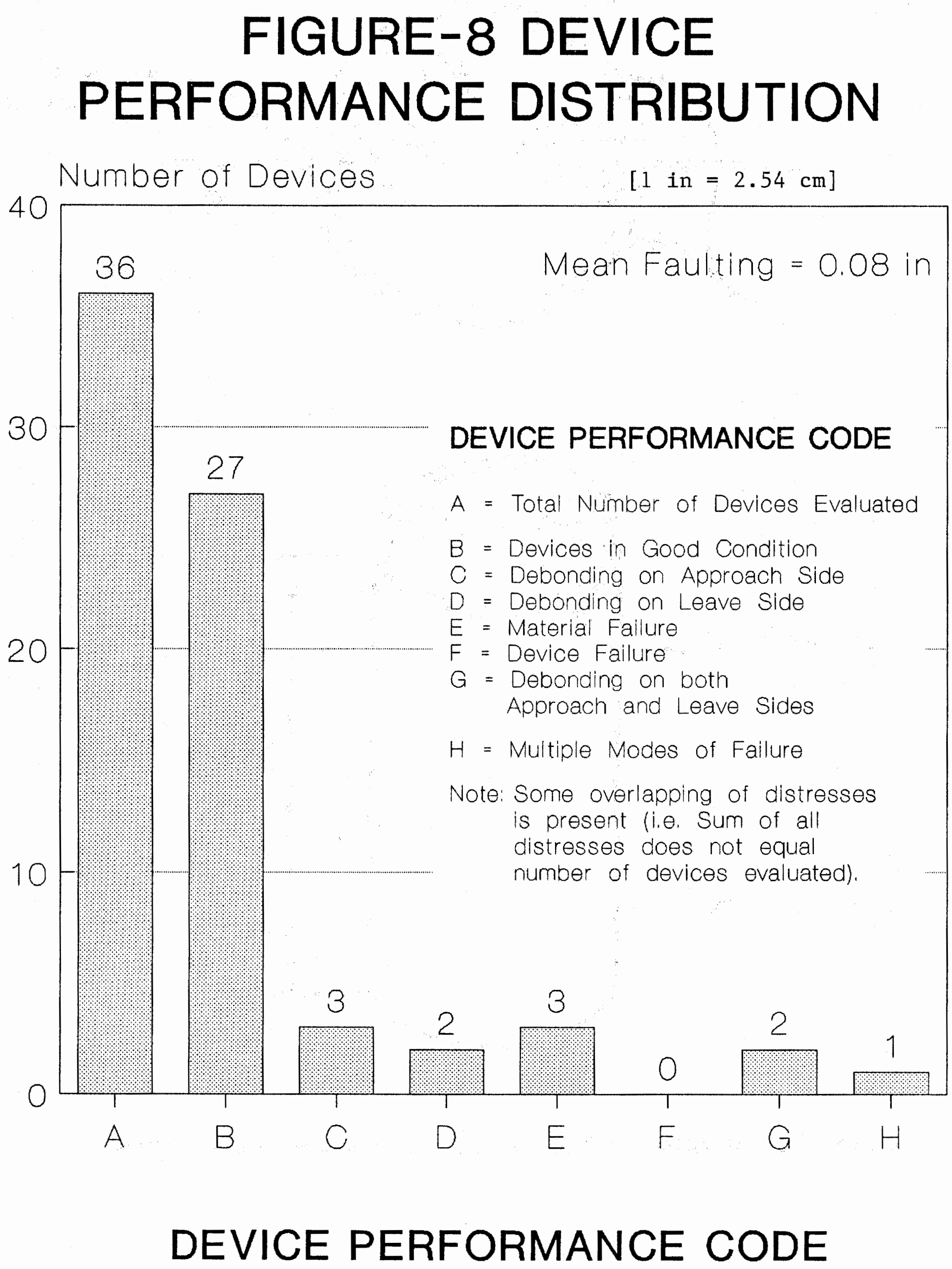


Figure 23. Distribution of figure-eight device performance.

these failure modes occurred at 8 percent of the devices. None of the joints restored with Figure-eight devices exhibited device failure. Multiple modes of failure were identified at approximately 3 percent of the devices. Multiple modes of failure refers to the existence of two or more of the failure mechanisms listed in table 8 at any one device.

3.3.6 Performance Summary

Table 8 lists the four load transfer devices evaluated in this study along with their respective modes of failure. If a device had more than one failure mode, each failure mode was recorded separately. This resulted in a cumulative percentage greater than 100 percent. The entry entitled "multiple modes of failure" was established to help determine if any of the devices have deteriorated drastically; therefore, possibly providing an indication of the extent of device failure present since the devices themselves can not be seen directly.

3.4 PERFORMANCE MODELS

3.4.1 Model Development

A predictive model for faulting after load transfer restoration was needed to determine their effectiveness and for estimating future faulting. The regression analysis of the load transfer restoration database was accomplished using the SHAZAM and SPSS (Statistics Package for the Social Sciences) statistical packages.(20,21) The initial analysis of the database variables to be included in the model was conducted by choosing those independent variables which were considered to be meaningful and with significant influence on the performance of restoring load transfer. The analysis resulted in the development of a performance model for joint and/or crack faulting.

3.4.2 Faulting Model

The faulting model for the prediction of future joint or crack faulting from the time of load transfer restoration is shown below. It should be stressed that this model was derived from a database where all of the projects had diamond grinding performed at the joints or over the entire project length in the same year as the load transfer was restored. To develop the model, all of the projects in the grinding database and the load transfer database were utilized.

Joint Faulting

$$\begin{aligned} \text{FAULT} = {} & -5.62\,(\text{ESAL}+\text{AGE})^{0.540}\,[\,5.85\,(\text{DRAIN}+\text{SUB}+1)^{0.0529} \\ & -3.8\text{x}10^{-9}\,(\text{FI}/100)^{6.29} + 0.48\,(\text{THICK}+\text{PCCSH})^{0.335} \\ & +0.1554\ \text{BASE} - 7.163\ \text{JSPACE}^{0.0137} + 0.136\ \text{DOWEL} \\ & +0.003\ \text{SHEAR} - 0.027\ \text{FIG8} - 0.316\ \text{IBEAM}\,]\,/\,100 \end{aligned}$$

where:

FAULT = The mean faulting of the restored, ground joints or cracks, inches.

ESAL = Equivalent 18-kip [80 kN] single axle loads accumulated on the restored, ground joints or cracks, millions.

AGE = Age of the restored, ground joints or cracks, years.

DRAIN = 0, if subdrainage is present currently (whether installed initially or incorporated in the rehabilitation).
1, if no subdrainage is present.

SUB = 0, if subgrade is a fine-grained soil.
1, if subgrade is a coarse-grained soil.

FI = Mean Freezing Index, degree days below freezing.

THICK = Thickness of the in-place concrete slab, inches.

PCCSH = 0, if concrete shoulders are not present.
1, if concrete shoulders are present.

BASE = 0, if granular base type.
1, if stabilized base type (asphalt, cement).

JSPACE = Contraction joint spacing, feet

DOWEL = 0, if retrofit dowels are not used to restore load transfer.
1, if retrofit dowels are used to restore load transfer.

SHEAR = 0, if Double-vee shear devices (uncompressed, ungrooved) are not used to restore load transfer.
1, if Double-vee shear devices (uncompressed, ungrooved) are used to restore load transfer.

FIG8 = 0, if Figure-eight devices are not used to restore load transfer.
1, if Figure-eight devices are used to restore load transfer.

IBEAM = 0, if I-beam devices are not used to restore load transfer.
1, if I-beam devices are used to restore load transfer.

Statistics:
R^2 = 0.30
SEE = 0.04 in [0.10 cm]
n = 114 grinding sections without load transfer restoration) plus 368 load transfer joints

Equation Range of Applicability:

ESAL The accumulated ESALs ranged from a minimum of 0.225 million in Minnesota to a maximum of 7.812 million in South Carolina, with most projects having accumulated less than 3.0 million ESALs.

AGE The range of project ages varied from a low of 1 year in Arizona, Illinois, Iowa, Louisiana, Pennsylvania, South Carolina and Virginia to a high of 9 years in Georgia and South Carolina, with most projects less than 5 years old.

FI The Freezing Index ranged from a minimum of 0 in 9 southern project States to a maximum of 1750 in Minnesota, with the majority of the projects exposed to a Freezing Index between 0 and 250 freezing degree days.

THICK The range in pavement thickness varies from a low of 7 in [17.8 cm] in Minnesota to a high of 12 in [30.5 cm] in Arizona, with most projects having a 9- or 10-in [22.9 or 25.4 cm] thick pavement.

JSPACE	The contraction joint spacing ranged from 15 ft [4.6 m] in Arizona, Arkansas, California, Minnesota and Oklahoma to 100 ft [30.5 m] in Illinois, with most projects built with a joint spacing between 15 to 30 ft [4.6 to 9.1 m].
Note:	All of the pavements incorporated into the regression analysis of load transfer restoration also had diamond grinding of the entire pavement surface or localized grinding at the restored transverse joints.

A sensitivity plot is shown in figure 24 for jointed reinforced concrete pavement (JRCP). The inputs for the pavement design variables were selected from a list of standard inputs as given in table 6, chapter 2.

Faulting for both the jointed plain and jointed reinforced pavements increases rapidly initially and then levels off as the pavements accumulated more loadings. This type of curve has been found for all types of new and restored pavements as well as full-depth repairs.(1) The figure contains five curves:

- Retrofit dowels.
- Double-vee shear devices.
- Figure-eight devices.
- Miniature I-beam devices.
- No devices (diamond grinding alone).

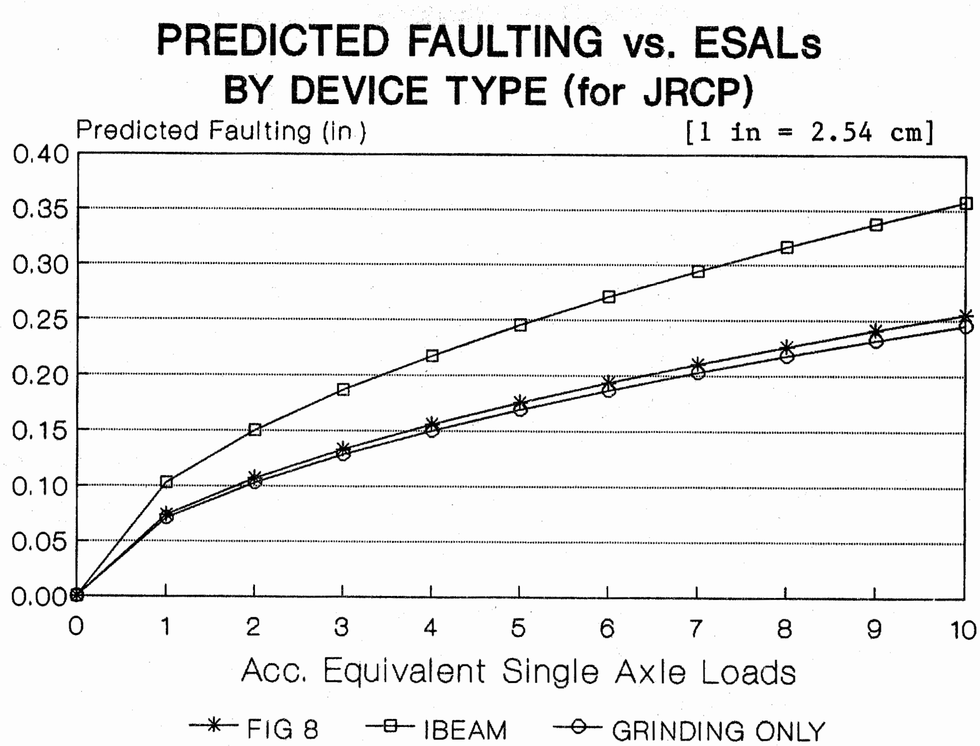
The plot shows that the retrofit dowel bars reduce faulting significantly from that obtained with grinding alone. The Double-vee shear devices and Figure-eight devices have practically no effect, while the I-beam devices appear to increase faulting. This increase, however, must not be taken literally as there is no physical reason for this result. It should only be concluded that the device has no effect on faulting according to the available data. These results are in response to the coefficients that were derived from the regression analysis. Similar results are shown in figure 25 for JPCP, but without the I-beams, since these devices were only used on JRCP. If one considers the following criteria for faulting of JPCP (0.15 in [0.38 cm]) and JRCP (0.20 in [0.51 cm]), the following allowable loadings would result from this model:

Restoration Device	JPCP Allowable	JRCP Loadings
Retrofit Dowels	16.0	10.0
Diamond Grinding Alone	8.8	6.9

(Loadings in millions of 18-kip [80 kN] ESALs)

The extension of life obtained with retrofit dowels is significant (almost double). Diamond grinding addresses only the symptoms of pavement deterioration (excessive faulting) without addressing the source of the deterioration which may require load transfer restoration, subdrainage, etc. If diamond grinding is used in this temporary repair strategy, it has been shown that faulting will develop at a rate greater than the initial new pavement faulting pattern (see volume II, chapter 2). The use of load transfer restoration appears to be an effective means to extend the life of a restoration project.

It is important to note that the Double-vee devices included in this study did not include grooving of the core walls or precompression of the devices. These two



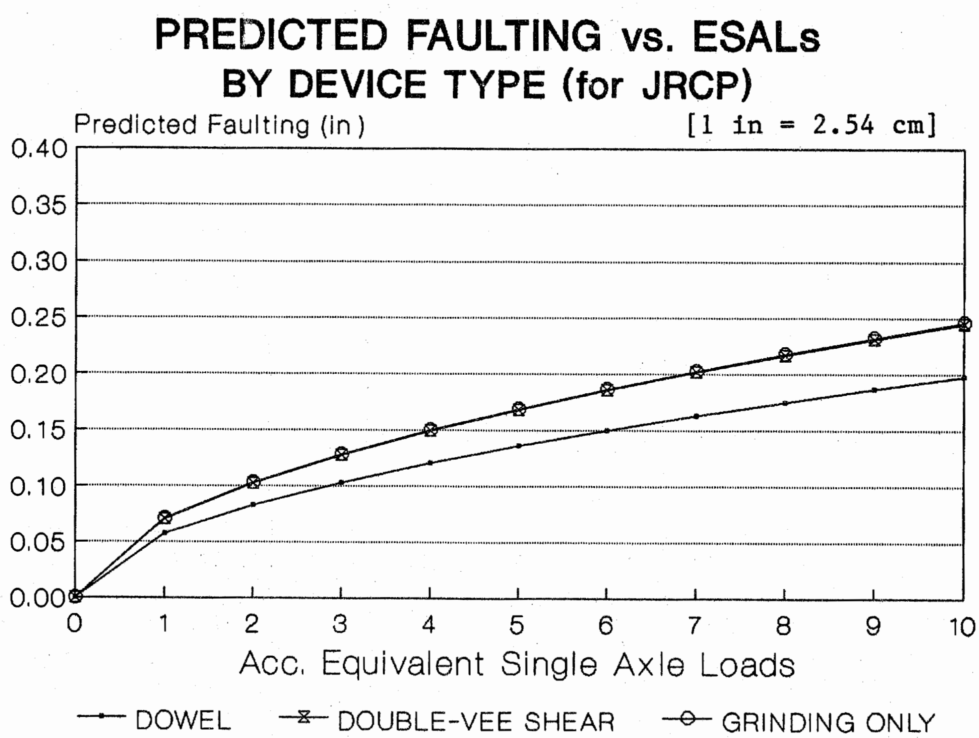
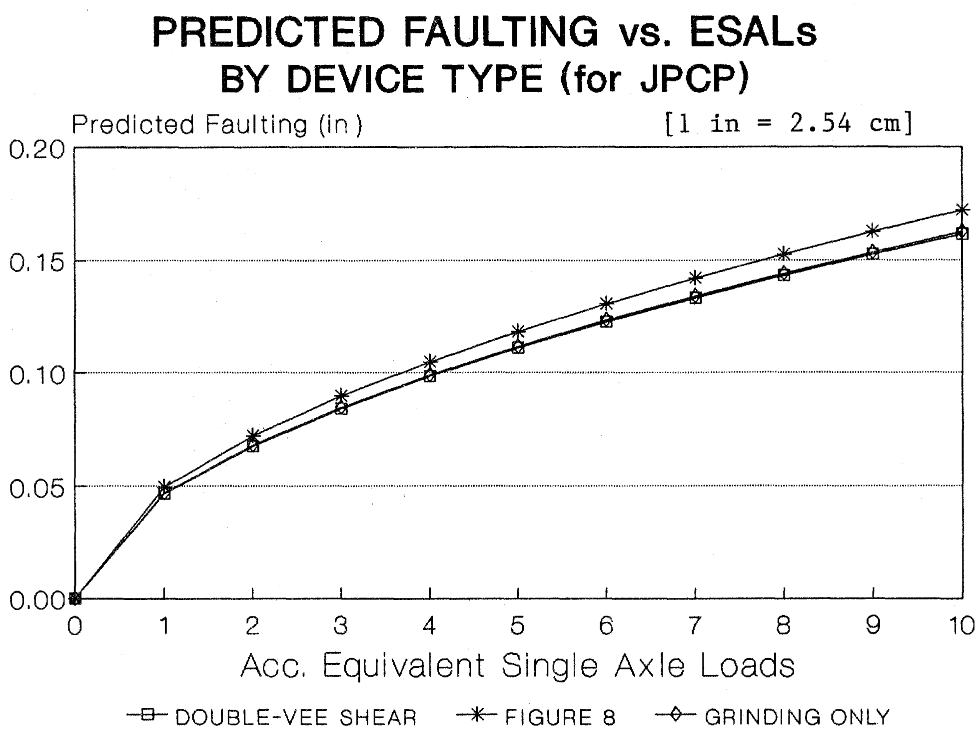


Figure 24. Sensitivity plot depicting model-predicted faulting vs. accumulated 18-kip [80kN] ESALs for JRCP.



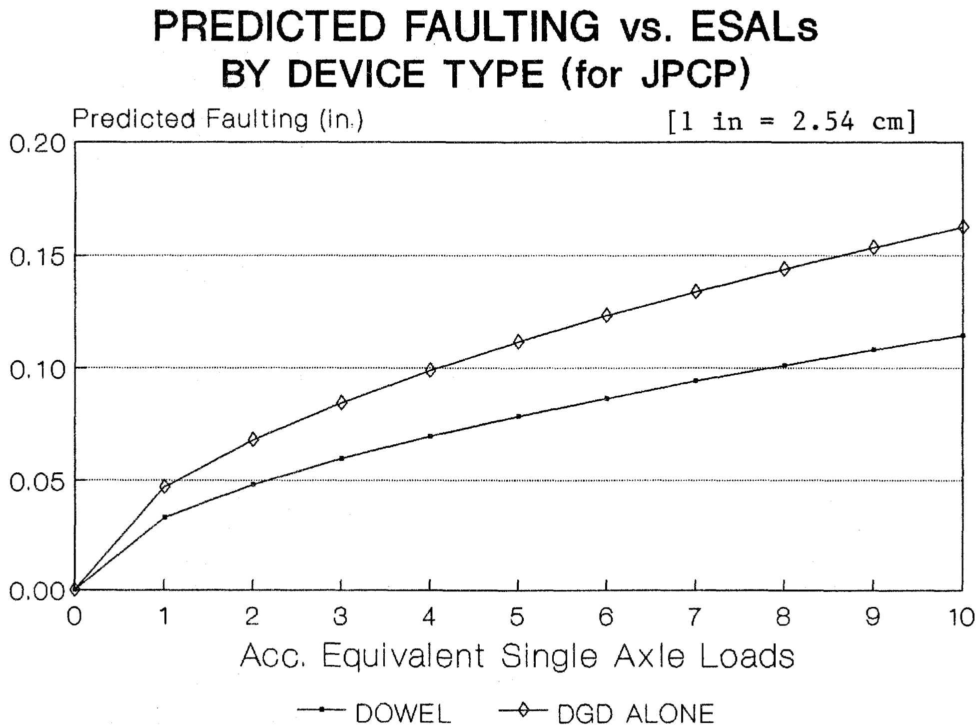


Figure 25. Sensitivity plot depicting model-predicted faulting vs. accumulated 18-kip [80 kN] ESALs for JPCP.

modifications may or may not have a significant effect on their performance and are currently under study as described in section 3.5.

3.5 CURRENT RESEARCH IN LOAD TRANSFER RESTORATION

Two of the devices which showed the most promise during past research are the retrofit dowels and Double-vee shear devices. As expected, each successive experimental project incorporated slight modifications to the design and/or installation procedures in order to improve the field performance of these load transfer restoration devices. To further test and improve these devices, a statistically designed load transfer restoration experimental project was installed on Interstate 10 near Tallahassee, Florida. The project included 8 different retrofit dowel configurations (1152 dowels in all) and 6 different precompressed shear device (432 devices in all) configurations. Each of these different configurations was installed at nine consecutive transverse contraction joints. The 14 identical sets of different configurations were constructed in both the eastbound and westbound outer traffic lanes for replication purposes. Figures 26 through 28 and tables 9 and 10 illustrate the 14 different configurations, their layout plans and the relative installation positions. These devices were installed in the fall of 1986.

The monitoring of this project includes preconstruction and postconstruction:

- Faulting measurements.
- Falling Weight Deflectometer load transfer measurements.
- Visual performance rating of the devices themselves.

The preliminary results from the first set of monitoring data taken after 4 months of service indicates that all of the retrofit dowels are performing well, with predominantly good load transfer and very few visually apparent distresses. Twenty-six, or 6 percent, of the precompressed shear devices have some form of visual distress. Eight of the 432 devices placed (or 2 percent) display serious defects such as cracked or repaired backfill matrix, replaced devices or debonding between the backfill matrix and the adjacent concrete slab. The 18 remaining distressed shear devices only exhibit a minor matrix flaw described as "flaking." This "flaking" may be the result of difficulties encountered during the joint reservoir resawing operation.

Both devices have increased joint load transfer greatly and have reduced both corner deflection and joint faulting over that of adjacent control joints. Monitoring of performance is continuing on this project.

3.6 DESIGN AND CONSTRUCTION GUIDELINES -- RESTORATION OF JOINT LOAD TRANSFER

3.6.1 Introduction

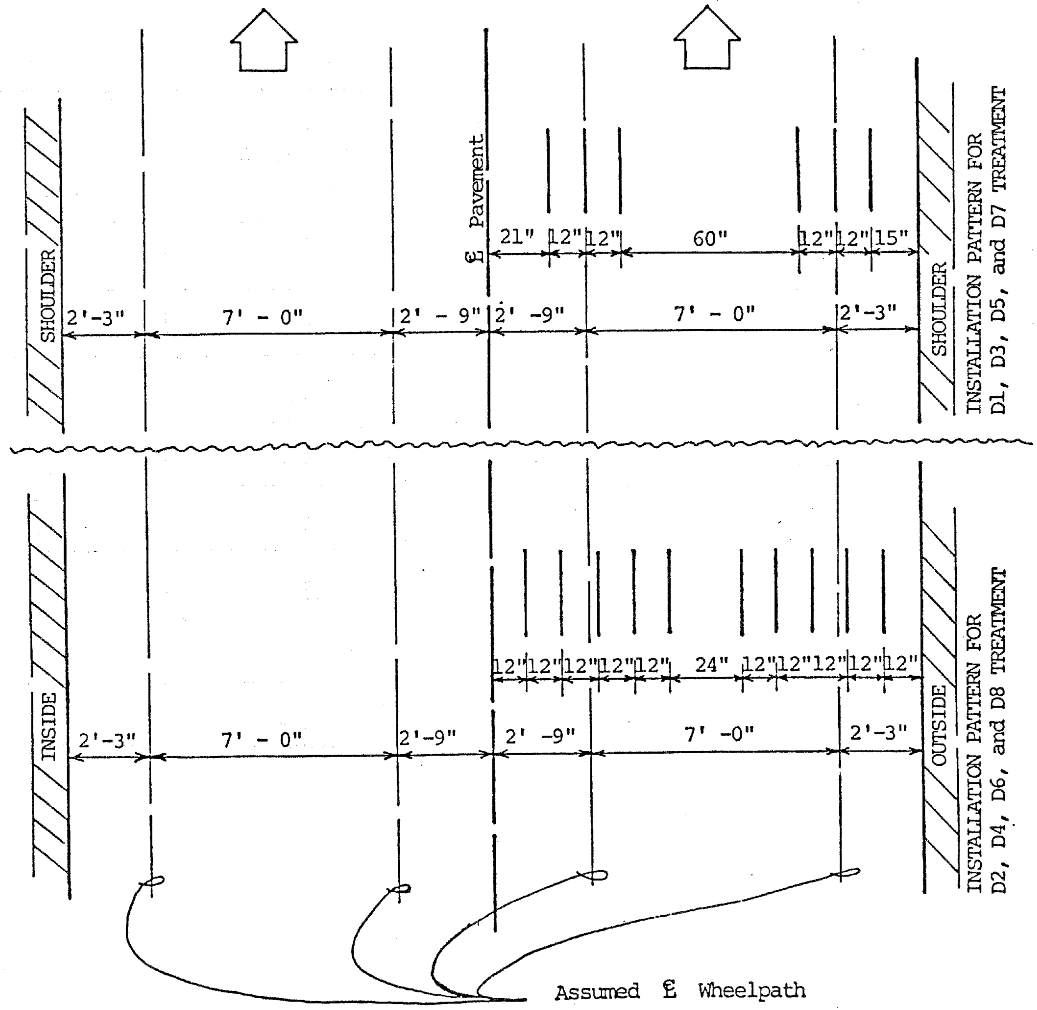
These guidelines were originally prepared under NCHRP Project 1-21 and published in NCHRP Report No. 281, Transportation Research Board, 1985.(32) Further updates resulted from the research conducted for the "Determination of Rehabilitation Methods for Rigid Pavements" study conducted for the FHWA, which is described in this final report.

The ability of a joint or crack to transfer load is a major factor in its structural performance. Load transfer efficiency across a joint or crack is defined as the ratio of deflection of the unloaded side to the deflection of the loaded side. If perfect load transfer exists the ratio will be 1.00 (or 100 percent), and if no load transfer exists (such as a free edge) the ratio will be 0.00 (or 0 percent). Joints

INSTALLATION PATTERN

FOR

RETROFIT DOWEL LOAD TRANSFER DEVICES



Assumptions:
(1) ℄ of dual wheel average 27 in from outside pavement edge.
(2) 8 ft-6 in out to out dual wheel distance (7 ft-0 in center to center).
(3) 12-in dowel spacing is desirable.

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

Figure 26. Installation pattern for retrofit dowels in Florida.

Table 9. Description of treatment configurations for retrofit dowels in Florida.

TREATMENT DESCRIPTION

FOR

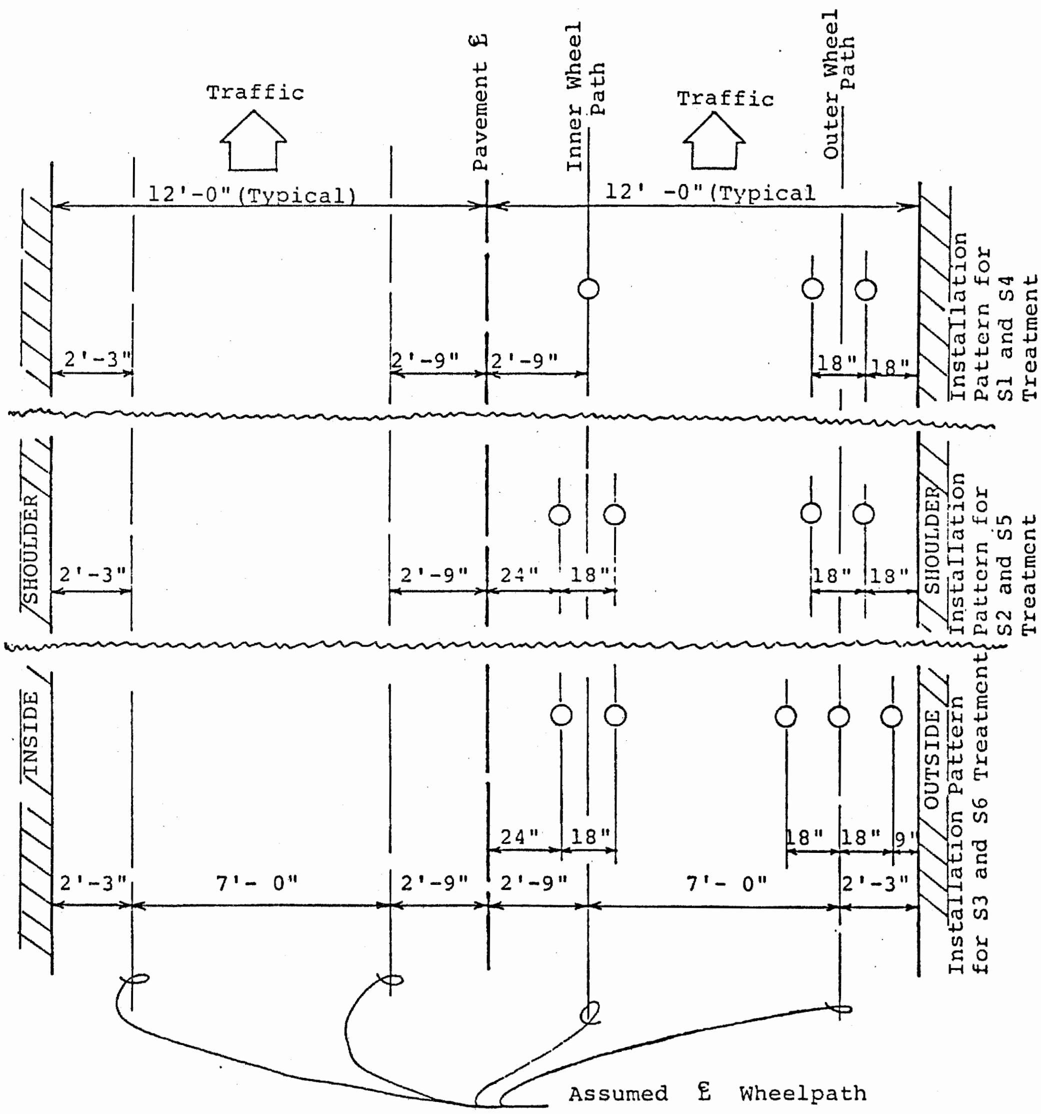
RETROFIT DOWEL LOAD TRANSFER DEVICES

(Epoxy Coated)

Treatment Designation	No. of Dowels Per Wheelpath	Dowel Diameter (in)	Dowel Length (in)
D1	3	1	14
D2	5	1	14
D3	3	1½	14
D4	5	1½	14
D5	3	1	18
D6	5	1	18
D7	3	1½	18
D8	5	1½	18

Note: 1 in = 2.54 cm

Installation Pattern For Retrofit Load Transfer Shear Devices



Assumptions:
(1) ℄ of dual wheel average 27 in from outside pavement edge.
(2) 8 ft-6 in out to out dual wheel distance (7 ft-0 in center to center).

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

Figure 27. Installation pattern for double-vee shear devices in Florida.

Table 10. Description of treatment configurations for double-vee shear devices in Florida.

TREATMENT DESCRIPTION
FOR
RETROFIT LOAD TRANSFER SHEAR DEVICES

Treatment Designation	No. of Devices per wheelpath		* Grooving N=not required y=required
	Inner	Outer	
S1	1	2	N
S2	2	2	N
S3	2	3	N
S4	1	2	Y
S5	2	2	Y
S6	2	3	Y

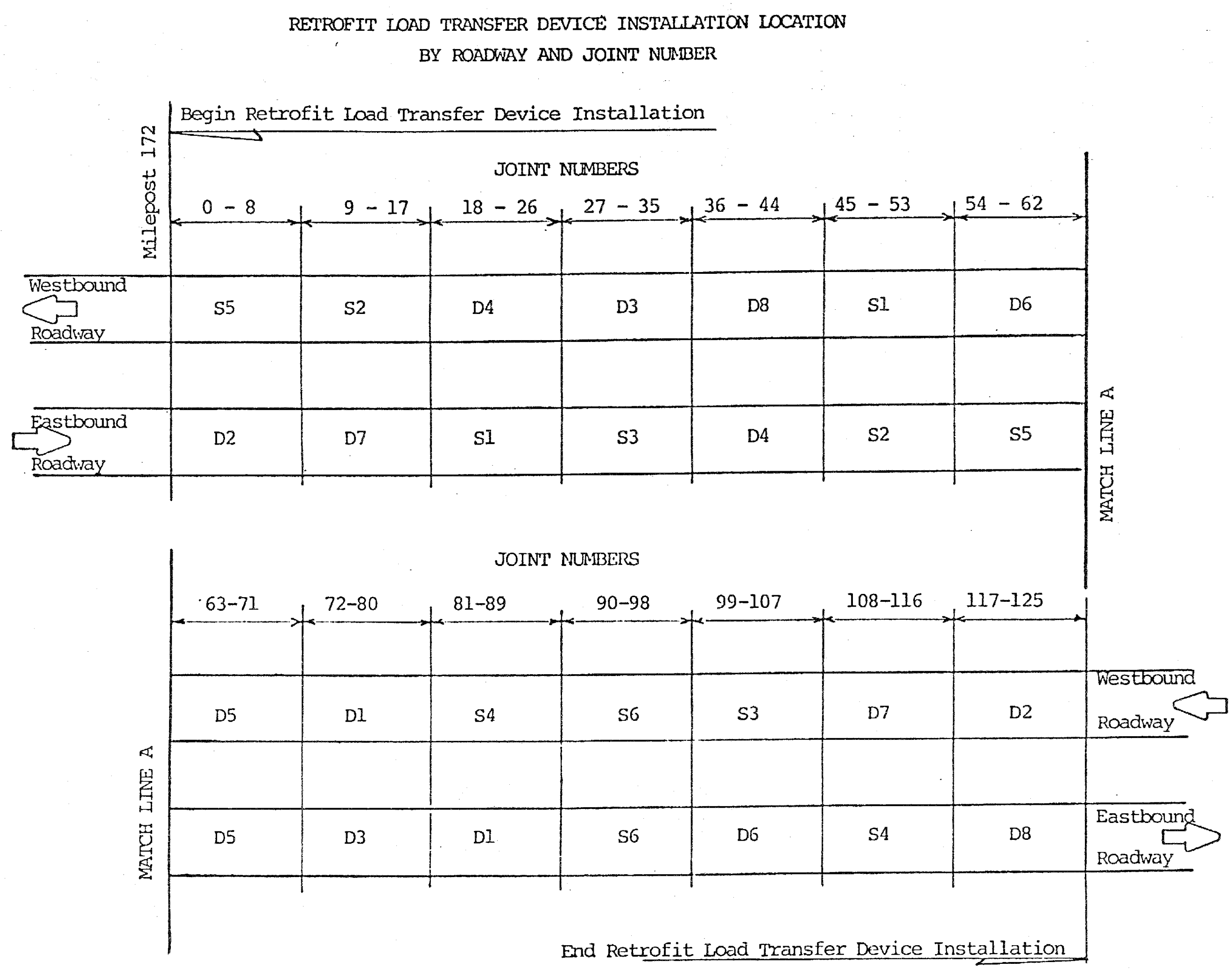


Figure 28. Layout plan of treatment configurations and locations by joint number in Florida project.

that are doweled normally have good load transfer (70 - 100 percent). However, repeated heavy loads can cause an elongation of the dowel socket cross sections and looseness of the dowel. This leads to a loss of load transfer and faulted and spalled joints.

Many jointed plain concrete pavements have been constructed without dowels at transverse joints. The load transfer measured at these joints is typically low, except on warm afternoons when joints close tightly. Transverse cracks in both jointed plain and reinforced concrete pavements (where steel has ruptured) can also have poor load transfer.

When load transfer is restored from 0 to 100 percent, maximum deflection and stress in the slab is <u>reduced by one half</u>. This effect greatly reduces the potential for pumping, faulting, spalling and cracking and thus would extend the life of the pavement.

<u>Need For Load Transfer Restoration</u>

Restoration of load transfer across a transverse joint or crack can be used to retard further deterioration. Poor load transfer leads to joint or crack deterioration, including pumping, faulting, corner breaks and spalling. Overlays placed over joints or cracks that have poor load transfer will soon develop reflective cracks that will spall and deteriorate into potholes.

Load transfer restoration is recommended on all transverse faulted joints or cracks that exhibit poor deflection load transfer of approximately 0 to 50 percent when measured during early morning times or in cooler weather. Heavy load deflection devices should be used for the measurement so as to resemble regular traffic loads. These recommendations are for jointed concrete pavements with or without placement of asphalt overlays.(32)

<u>Effectiveness and Limitations</u>

Two of the most promising methods of restoring load transfer to existing joints and cracks are <u>retrofit dowels and Double-vee shear devices</u>.

Short-term experience with load transfer restoration has indicated that dowels and shear devices can be effective in transferring loads across joints and cracks and reducing faulting.(32,17,35) Test results from the NCHRP Project 1-21 field demonstration projects and from the Georgia and Florida tests show an immediate increase in load transfer to 90 - 100 percent and a reduction in deflections ranging from 50 to 75 percent.

Long-term effectiveness can only be estimated from the Georgia project which is currently 9 years old (5.45 million ESALs accumulated on the restored, ground joints). Ninety-nine percent of the 61 retrofit dowel restored joints, 87 percent of the 44 uncompressed, ungrooved Double-vee shear device restored joints and 88 percent of the eight Figure-eight device restored joints measured less than or equal to 0.10 in [0.25 cm] of joint faulting over the last 9 years of service. However, a substantial number of failures of shear devices have occurred by bond loss between the device and the core wall.(32,17) Shear devices which have been effective in transferring load and have performed well under full scale field load testing are the Double-vee device and the plate and stud connector. Both of these devices are proprietary.

The Double-vee device has been tested in laboratory fatigue tests and in field installations. Fatigue tests have shown that load transfer failure occurred first with the device itself, failing in flexure only after several million repetitions. However, field tests have shown considerable failure of the bond between the polymer concrete and the core wall of the existing concrete. These failures were believed to be partially caused by loss of the liquid portion of the polymer concrete, which drained out through the bottom of the core hole due to improper sealing. Also, improvements have been made in the installation of the Double-vee devices by cutting grooves into the core walls and by precompressing the device in the core hole. However, an improved bonding material is still greatly needed.(32)

The following conclusions are from the Georgia installations:

"The results of the sections with Double-vee devices is variable and is largely influenced by the performance of the various patching materials used with these devices. The Double-vee devices are performing well where leaching of the polymer concrete did not take place, where Portland Cement concrete was used and with some of the rapid set materials. The Double-vee devices are performing marginal to poor where problems with leaching and material quality of the polymer concrete occurred during the 1981 construction season." (17)

The only equipment needed for the installation of shear devices is a coring rig with a 6-in [15.2 cm]-diameter diamond core bit, which is normally readily available to all pavement contractors, and a special precompression tool and groove coring bit, both provided by the manufacturer (Dayton Superior Corporation).

Dowels cut in slots are an effective technique to restore load transfer across joints or cracks. Dowel installation has been evaluated under an FHWA contract in Georgia and is currently being studied in Florida.(17,35) Results for Georgia show the dowels to have performed very well after 9 years of heavy traffic, although a few failures have occurred. The patching material was not as critical as for the shear type devices.(17,35) The equipment needed to install dowels is a diamond saw to cut the slots, and air hammers. Equipment manufacturers are currently working on developing more efficient means of cutting the slot and removing the concrete.

Measurements show that the horizontal joint movement is not excessively restricted by either the Double-vee or dowel devices.(17)

The successful installation of load transfer devices requires sound concrete adjacent to the joint or crack. If the concrete is deteriorated near joints or cracks, a full-depth repair should be placed rather than load transfer restoration.

3.6.2 Concurrent Work

Before any load transfer devices are installed it is necessary to determine the cause of the joint/crack distress. Attempts should be made to correct these deficiencies prior to load transfer restoration.

Heavily distressed slabs ("D" cracked, corner breaks, transverse, longitudinal, and diagonal cracking) may require portions, or all of the slab to be replaced. In which case the load transfer can be restored through the patch design.

Additional work to be done prior to load transfer restoration may include subsealing to restore support to the slabs (this is essential if loss of support exists), full-depth repair and partial-depth repair. Work that can be done after load transfer restoration includes grinding, joint and crack sealing and installation of subdrainage.

Joints or cracks having high deflections must be subsealed <u>before</u> load transfer devices are installed.

3.6.3 Design

Identification of Joints/Cracks Requiring Load Transfer Restoration

Joints and cracks requiring improved load transfer must first be identified. Load transfer should be measured during cooler temperatures (e.g., ambient temperatures less than 80 ^{O}F [26.7 ^{O}C]) and during early morning times. A heavy load device such as the Falling Weight Deflectometer, Road Rater or a weight truck with two Benkleman Beams should be used.

The deflection load transfer should be measured in the outer wheel path and is defined as follows:

Load Transfer = [Unloaded slab defl. / Loaded slab defl.] X 100

Any joint or crack having a measured load transfer of less than 50 percent during cool temperatures should be considered for restoration. The deflection measurements should be taken as close as possible to the joint/crack, or if measured by a sensor in the center of the load plate and 12 in [30.5 cm] across the joint they should be corrected for normal slab bending as measured in the center of the slab.(32)

It is recommended that any transverse joint or crack with load transfer less than 50 percent (measured at pavement surface temperatures less than 80 ^{O}F [27 ^{O}C]) should have load transfer devices installed.

If deflection measurements are impossible, an indicator of poor load transfer is faulting of the joint or crack. Any joint greater than 0.10 in [0.25 cm] of faulting or more will likely have poor load transfer.

Design Requirements

Gulden and Brown conclude that the following factors must be met for a load transfer restoration system to provide long-term performance:

"● The patching material and device must have sufficient strength to carry the required load.

● Sufficient bond must be achieved between the device and the patching material to carry the required load.

● Sufficient bond must be achieved between the patching material and the existing concrete to carry the required load.

● The device must be able to accommodate movement due to thermal movement of the concrete slabs.

● The bond between the device and the patching material must be sufficient to withstand the forces due to thermal expansion of the concrete slabs.

● The patching materials must have little or no shrinkage during curing. Shrinkage of the patching material can cause weakening or failure of the bond with the existing concrete.

- The patching material must develop strength rapidly so that traffic can be allowed on the slabs in a reasonable length of time (3 to 4 hours)".(17)

Results from tests conducted in Georgia, Florida and other States show that the retrofit dowel bars can meet the above requirements. These dowels, when properly constructed, were found to greatly improve the existing load transfer (and reduce deflection) and to permit horizontal movement (or opening and closing) of the joints.(32,17)

Dowel Devices

The number, diameter and spacing of dowel devices must be determined. An analysis was conducted by Tayabji and Colley that determined that stresses and deflections for six dowels spaced nonuniformly in a joint (three in each wheel path) were similar to stresses and deflections obtained for a joint with twelve uniformly spaced dowels.(22) Thus, placing the retrofit dowels in the wheel paths should provide similar performance and be more cost effective.

The number, spacing and diameter of the dowels will determine the amount of future faulting of the transverse joints. Several different retrofit dowel load transfer restoration designs were evaluated under this study. The following table shows these design variations and pertinent pavement factors:

Devices in Wheelpath Outer	Devices in Wheelpath Inner	Dowel Spacing (in)	Mean Fault (in)	Dowel Diameter (in)	Accumulated ESALs (millions)	Joint Spacing (ft)
4	4	15	0.04	1.25	5.45	30.0
3	3	12	0.09	1.25	1.49	15.0
3	2	18	0.03	1.25	5.45	30.0
4	0	18	0.01	1.25	5.45	30.0

NOTE: Faulting values pertain to the outer lane only, measured 1 ft in from the lane edge.
1 in = 2.54 cm; 1 ft = 0.3048 m.

Results from NCHRP Project 1-19 showed the significant impact dowel diameter has on faulting. Larger diameter dowels slow down the development of faulting in new pavements. The larger dowels also showed less loss of load transfer in the Illinois Interstate 70 full-depth repair study (see chapter 5). Figures 29 and 30 compare joint faulting of new JPC and JRC pavements of various dowel diameters with similar rehabilitated pavements (either diamond grinding alone or diamond grinding along with retrofit dowel load transfer restoration). These figures show that retrofit dowels reduce faulting; however, not to the same level as new construction dowelled pavements. This probably occurs because the aggregate interlock is much less for an older pavement than for new construction.

The development of a mechanistic-emperical retrofit dowel design procedure is currently under investigation using the results from the Florida test site in addition to data from other States. The best recommendations that can be provided at this time are as follows:

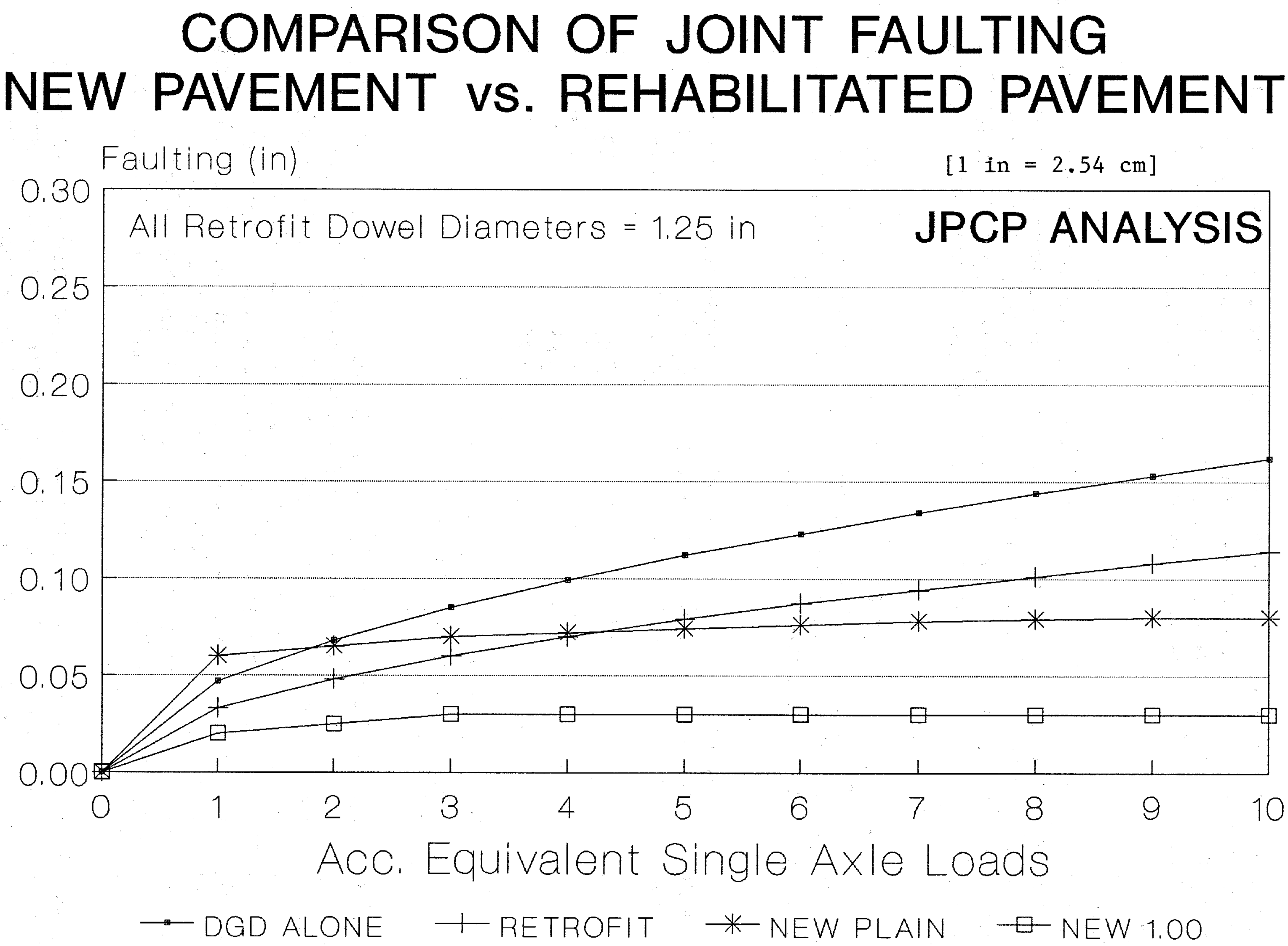


Figure 29. Comparison of JPCP joint faulting: new pavement vs. rehabilitated pavement.

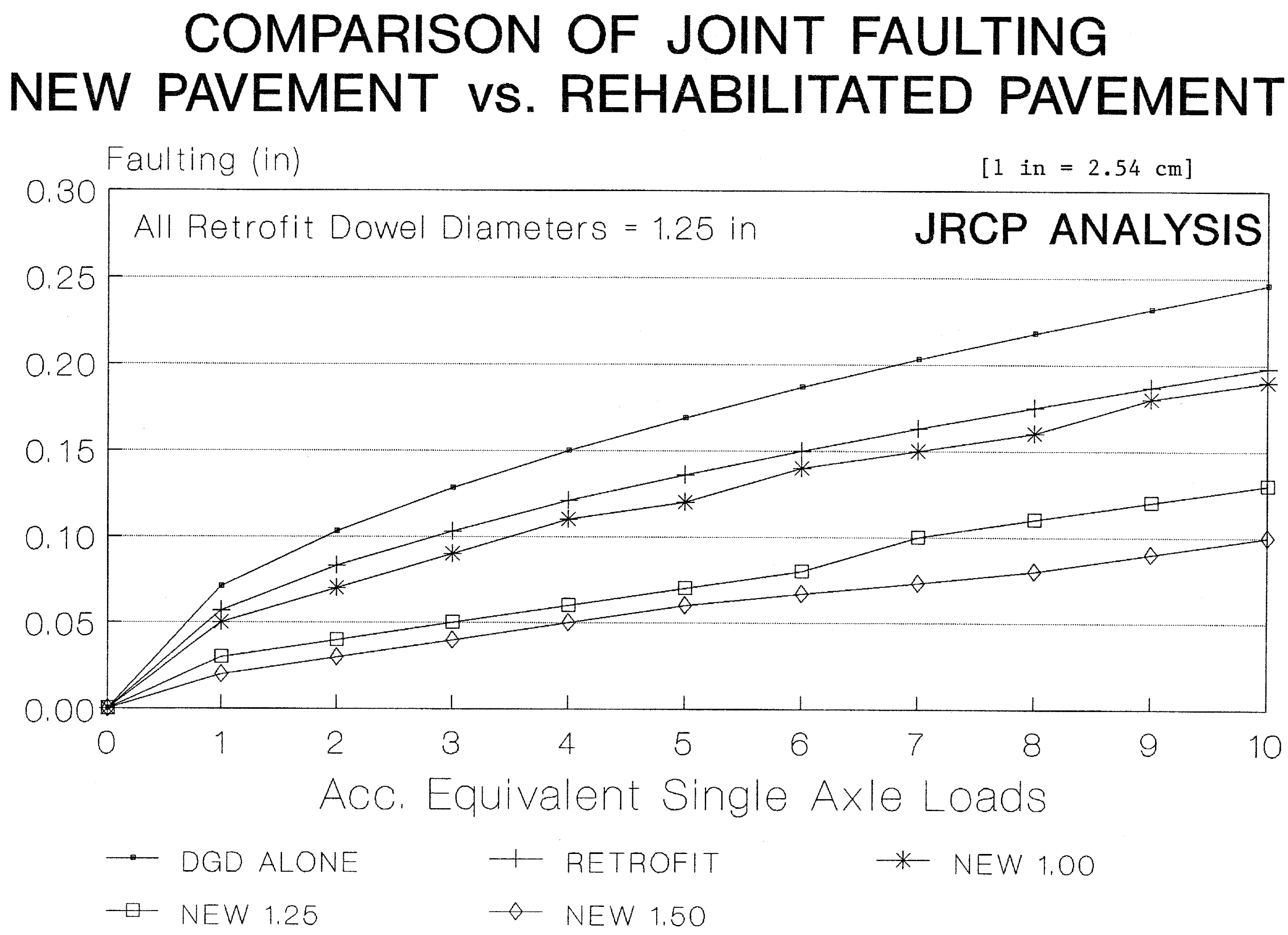


Figure 30. Comparison of JRCP joint faulting: new pavement vs. rehabilitated pavement.

1. Use dowel bars with diameters of at least 1.25 in [3.2 cm], and preferably 1.50 in [3.8 cm]. Heavier trafficked pavements having 0.5 million ESALs per year in the outer lane should use the 1.50-in [3.8 cm]-diameter bars.

2. Use 3 to 4 dowels placed in each wheelpath at 12-in [30.5 cm] spacings.

3. The outermost dowel in the outer wheelpath should be located 12 in [30.5 cm] from the outer lane edge.

4. Care must be taken to avoid any existing dowels in the pavement.

A recommended layout is shown in figure 31 for retrofit dowel design.

3.6.4 Construction

<u>Materials</u>

Plans should include details and sketches of the load transfer device itself. Details of the dowel device and supporting chair are shown in figure 17.

The patch material used with load transfer devices is a critical factor in performance, particularly with shear devices. Sufficient bond must be established between the device and patching material as well as between the existing concrete and the patching material to carry the applied loads and movement from thermal changes. Patching material must also develop strength rapidly to accommodate traffic and thermal stresses soon after placement.

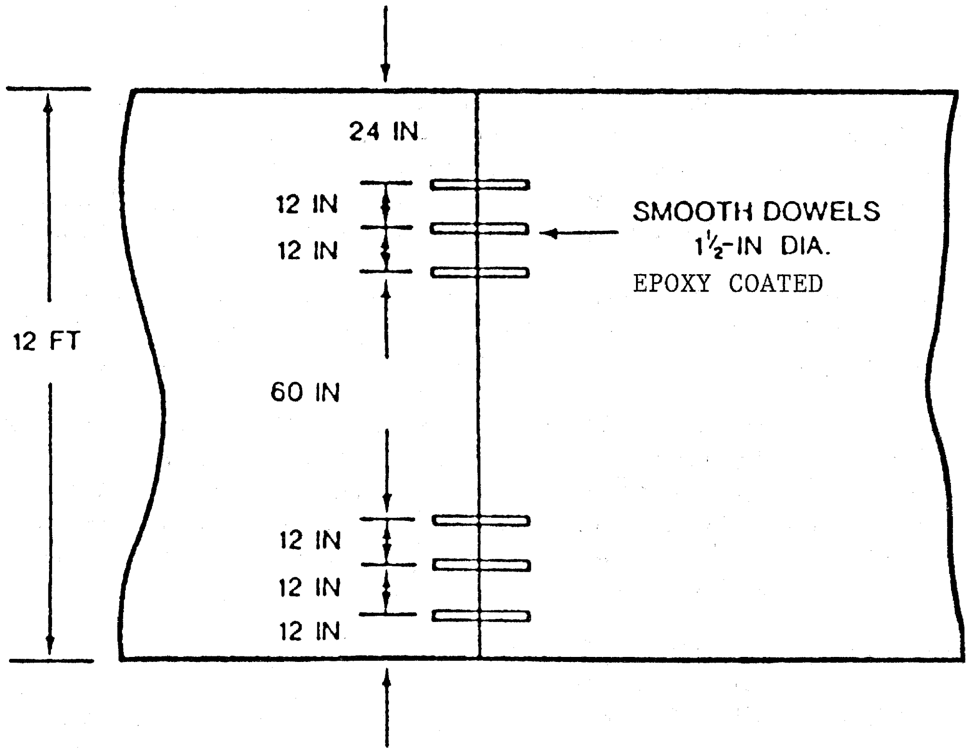
A thorough laboratory evaluation must be made of any patching material to be utilized for the load transfer devices. Gulden and Brown conclude that "working time, bond strength, rapid early strength gain and shrinkage are prime factors which must be evaluated prior to choosing a patching material".(35)

Polymer concretes and high early strength portland cement concrete have been used in most installations to date. Polymer concrete material properties, fine aggregate gradation, and mix designs should be specified by the agency. A high early strength concrete mixture used in conjunction with an epoxy applied to the existing slab was used successfully in Georgia.(17) Aggregate gradation should meet ASTM C33 "Standard Specification for Concrete Aggregates" fine aggregate requirements. This allows the polymer concrete to easily fill this space. The mix design should allow the fine aggregate to be easily and completely coated.

The high early strength portland cement concrete mixture utilized successfully in Georgia is as follows:

One bag cement - Type III
125 lb [56.7 kg] sand
220 lb [99.9 kg] stone - 3/8-in [0.95 cm] top sized pea gravel
5 gallons [18.9 liters] water
1 1/2 lb [0.68 kg] calcium chloride
Expansion agent 4.5 oz [127.6 g] (35)

The expansion agent was aluminum powder mixed with a filler in a ratio of one part powder to 50 parts of filler. Both inert flyash and pumicite was used as a filler. Four hour compressive strengths ranged from 1250 to 1650 psi [8.6 to 11.4 MPa].



Note: For very heavy traffic, 4 dowels may be necessary in each wheelpath.

1 in = 2.54 cm
1 ft = 0.3048 m

Figure 31. Recommended retrofit dowel design for heavy traffic. (6)

The Florida test section successfully used a heavy duty patch material (Trade name HD-50 and manufactured by Dayton Superior Corporation) for both the retrofit dowels and the Double-vee shear devices. Additionally, a 3/8-in top sized pea gravel extender was used for the dowels.

<u>Dowel Device Procedures</u>

When using dowels installed in slots, expansion caps should be specified. Coated dowels should be 14 to 18 in [35.6 to 45.7 cm] long and of sufficient diameter to reduce faulting to an acceptable level as described under section 3.6.3.

Slots for dowels should be first cut with multiple blade saws (e.g. a ganged sawing assembly will allow for a more uniform and efficient sawing operation). The "fins" have a life expectancy of about 1 week, depending on width, before they break down and the open slot becomes a hazard to traffic.(35)

Light weight pneumatic hammers are then used to remove the concrete with minimal damage to the surrounding concrete. Sandblasting of the slots followed by airblasting to provide for final cleaning should be performed. It is important to check the nozzle leading from the compressor with a clean rag for contaminants, such as oil, so that the oil is not emitted from the compressor thereby coating the surface of the slots.

Slots should be cut so that the dowels are allowed to rest horizontally and perpendicular to the joint or crack at mid-depth of the slab. Each dowel should be placed on a support chair to allow the patch material to surround the dowel.

Dowels must be provided with filler board or styrofoam material at mid-length to prevent the intrusion of patch material into the existing joint/crack, and to form the joint in the kerf. To account for varying joint/crack widths over the project, multiple thin sheets of filler can be used. To keep joints/cracks free of material it is important to have a tight fitting filler which matches the existing contraction joint width. Details of the dowel placement are shown in figure 17.

<u>Procedures for Opening to Traffic</u>

The lane may be opened to traffic after several hours of hardening, depending on materials tests and the agency's experience with patching material and slab temperature.

3.6.5 Preparation of Plans and Specifications

The plans must indicate the joints and cracks and spacings where load transfer devices are to be placed. The agency should determine which joints/cracks need load transfer restoration by measurement of the deflection transfer as discussed in section 3.6.3.

A detailed engineering drawing of the device to be used must be provided.

3.7 CONCLUSIONS AND RECOMMENDATIONS

1. This research study revealed that the retrofit dowel bars provided the best results in reducing faulting of all load transfer devices. The Double-vee shear device (without precompression or grooving of the core walls), the Figure-eight shear device and the retrofit miniature I-beam device did not reduce faulting to any greater degree than joints where only diamond grinding was performed. All of the projects considered here had diamond grinding conducted as part of their rehabilitation strategies. The initial faulting, therefore, was zero in all cases

and direct comparison between the devices could be made. Device faulting performance is summarized below:

Device Type	Mean Fault (in)	Mean ESAL (millions)	Mean Age (yrs)
Retrofit Dowels	0.04 [0.10 cm]	2.6	3.8
Double-vee Devices	0.07 [0.18 cm]	2.6	2.5
Figure-eight Devices	0.08 [0.20 cm]	5.5	9.0
I-beam Devices	0.13 [0.33 cm]	4.0	2.0

The results of this analysis reflect a wide range of both project and rehabilitation design, inservice life, traffic loading and climatic variables.

2. The faulting analysis between load transfer restored and control joints clearly showed the benefit of some types of load transfer restoration as a rehabilitation technique, restricting the development of joint and/or crack faulting. As expected, load transfer efficiency from the Florida test site was greatly increased and deflections reduced through the use of the load transfer restoration devices.

3. The most promising method of restoring load transfer to existing transverse joints and cracks is <u>Retrofit Dowels</u>. Results from test sites in Georgia and Florida, as well as from field tests, show that retrofit dowels can reliably meet the requirements to reduce faulting. These dowels, when properly installed, were found to greatly improve the existing load transfer (and reduce deflections) and to permit horizontal joint movement (or opening and closing).

4. The retrofit dowels were more effective and reliable than the other load transfer devices. However, the contractor in Florida indicated that, as expected, the dowels were more difficult to properly install than the Double-vee shear devices (even when the shear devices required core wall grooving and precompression). Equipment manufacturers are currently developing more efficient means of cutting the slots and removing the concrete "fins".

5. The device performance evaluation indicated that the critical factor for any of the devices was the performance of the backfill material. Backfill material failure was either the most prominent or second most prominent failure mode for any of the four load transfer devices evaluated. This was even evident on the retrofit dowel bars and miniature I-beams where less than 2 percent of the devices exhibited any failure modes.

6. The successful performance of load transfer restoration is controlled, as are so many other rehabilitation techniques, by the ability to identify and address the source of the deterioration. These distress mechanisms must be addressed and any deficiencies corrected prior to load transfer restoration. Typical rehabilitation work associated with the need for load transfer restoration can require (1) localized subsealing to provide uniform slab support in order to compensate for a pumped subbase, (2) retrofit subdrainage to provide a positive means for infiltrated free water to more rapidly leave the pavement structure, (3) diamond grinding of the restored joints or entire pavement to reestablish a smooth riding surface and (4) joint resealing. Diamond grinding and joint resealing are done after the load transfer devices have been installed.

7. The recommended retrofit dowel bar design is given in section 3.6, Design and Construction Guidelines.

CHAPTER 4

EDGE SUPPORT

4.0 RESEARCH APPROACH

The outer traffic lane edge and corner have long been identified as critical locations for high stresses and deflections. The outer edge develops high stresses and usually becomes the critical fatigue damage point where transverse cracks initiate and work across the traffic lane. The outer corner develops high deflections that result in pumping and subsequently faulting, loss of support and corner breaks/diagonal cracks.

Tied concrete shoulders and/or widened lanes have been shown to reduce the corner deflection and the edge stress produced by edge wheel loading. It is theorized that this reduction in deflections and stresses will result in a life extension to the mainline pavement. In addition, the expected benefits also may include a more reliable longitudinal lane-shoulder joint for effective joint reservoir construction and sealing which reduces the amount of water that can enter the pavement structure and deteriorate the underlying structural or supporting layers. Another benefit is a long lasting low maintenance shoulder pavement. These benefits, if true, are significant enough to warrant the consideration of edge support as a rehabilitation alternative for rigid pavements <u>of sound concrete</u>.

While there has been some field evidence that tied PCC shoulders are beneficial for new designs, there has not been field evidence that retrofit PCC shoulders have the same effect. The major concern is that the tie between the lane and shoulder is adequate to provide substantial load transfer. It load transfer is lost over time, the PCC shoulder will not have much of a significant effect on the traffic lane. The shoulder may separate greatly, eliminating the possibility of sealing the joint.

This research study will attempt to ascertain the benefits of edge support of the mainline pavement in terms of amount of reduction in joint faulting. A preliminary analysis of the database indicated that many of the edge support projects were actually tied concrete shoulders on a concrete overlay project or tied concrete shoulders in conjunction with diamond grinding of the mainline pavement. This resulted in very few sections where edge support was the only form of rehabilitation. To accommodate this reduction in sample size, the effect of edge support on joint faulting was incorporated into the diamond grinding joint faulting model, which did show a beneficial effect.

4.1 DATABASE AND DATA COLLECTION

The edge support database incorporates both design, construction and performance variables for 13 uniform sections. These variables are in addition to the pavement design, traffic and climatic variables summarized in volume IV. Table 11 lists these edge support variables. The monitoring data collection for the tied concrete shoulders is similar to the data collection associated with any of the traffic lanes, except for the addition of a measure of the dropoff at the lane-shoulder joint.

4.1.1 General Project Description

Thirteen uniform sections were located in nine States: Arkansas, Colorado, Illinois, Michigan, Minnesota, Ohio, Pennsylvania, South Carolina and Wyoming. These uniform sections were broken down into 22 sample units that were up to 1000 ft [305 m] long, where possible (see figure 32).

Table 11. Edge support database design and monitoring variables.

EDGE SUPPORT

DATABASE DESIGN VARIABLES

- Project Identification Number.
- Sample Unit.
- Type of Edge Support System.
- Matching of Shoulder and Pavement Joints.
- Lane/Shoulder Tie System.
- Tie Bar Diameter, Length, and Spacing.
- Shoulder Width and Thickness.
- Shoulder Thickness Tapering, if any.
- Thickness of Undercut, if any.
- Lane/Shoulder Joint Type.
- Lane/Shoulder Joint Forming Method.

DATABASE PERFORMANCE VARIABLES

Overall Distress

- Project Identification Number.
- Sample Unit Number, Length, and Present Serviceability Rating.
- Foundation of Sample Unit (cut,fill,at grade).
- Condition of Drainage Ditches and Subsurface Drainage.
- Number of Transverse Joints on the Mainline Pavement.
- Number of Transverse Joints on the Shoulder.

Pavement Distress (Inner Lane, Outer Lane, and Shoulders)

- Transverse Cracking.
- Transverse "D" Cracking.
- Longitudinal Cracking.
- Longitudinal "D" Cracking.
- Longitudinal Joint Spalling.
- Scaling, Crazing, Map Cracking.
- Centerline Joint Cracking (Outer Lane Only).
- Lane/Shoulder Drop-off (Shoulder Only).

Joint Distress (Inner Lane, Outer Lane, and Shoulders)

- Transverse Joint Spalling on Approach and/or Leave Side.
- Corner Spalling on Approach and/or Leave Side.
- Corner Breaks on Approach and/or Leave Side.
- Pumping.
- Joint/Crack Faulting.
- Joint/Crack Width.
- Durability Cracking.
- Reactive Aggregate.

Sealant Condition (Inner Lane, Outer Lane, and Shoulders)

- Sealant Absent.
- Cohesion Failure.
- Adhesion Failure.
- Sealant Extrusion.
- Sealant Oxidation.
- Incompressibles in Joint.

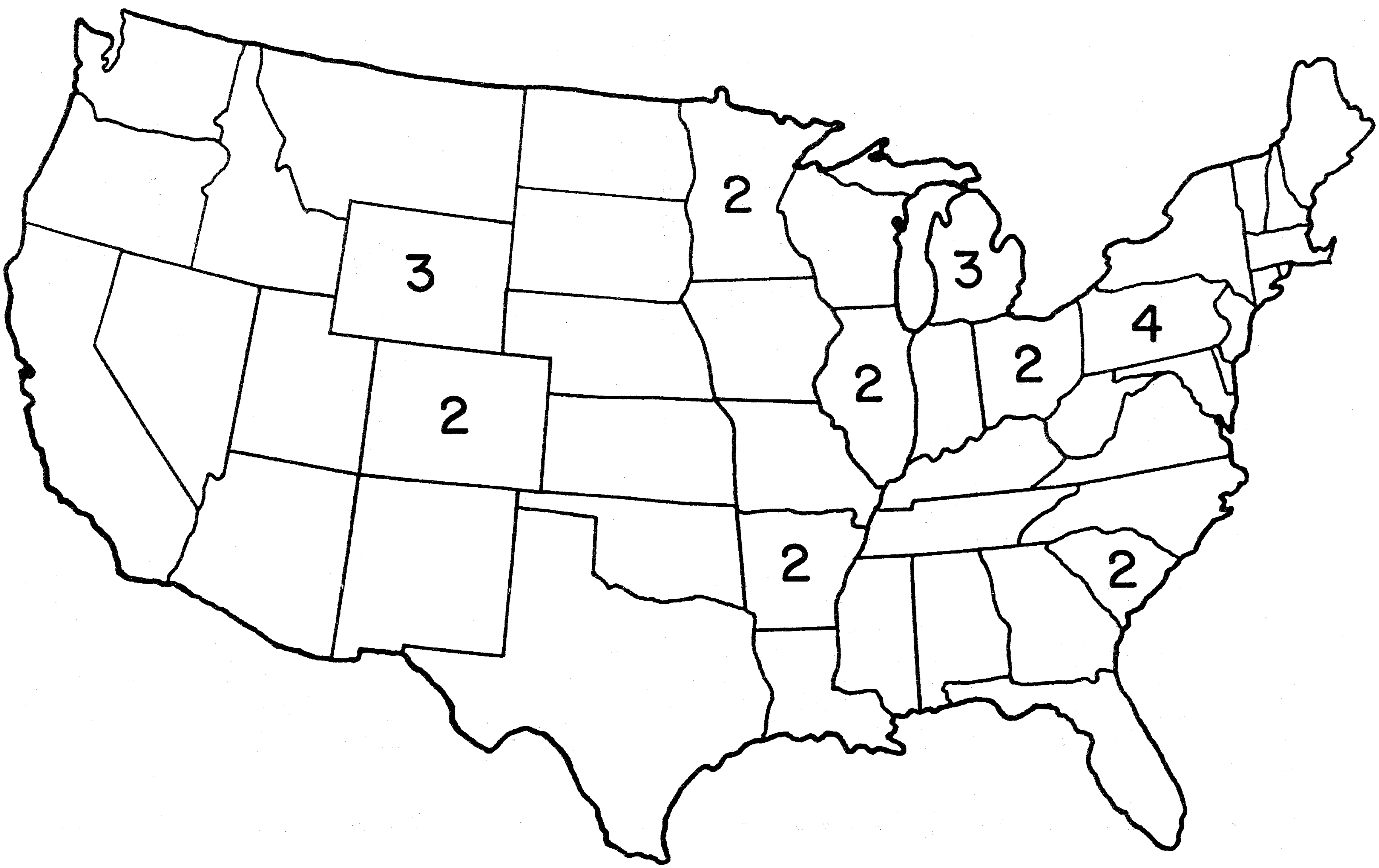


Figure 32. Location of edge support sample units by States.

4.1.2 Edge Support Design Variation

Edge support was incorporated into three main categories:

- Edge beam (narrow strip of PCC about 2 to 3 ft [0.6 to 0.9 m] wide tied to the existing traffic lane) (two uniform sections).
- Tied retrofit PCC shoulder.
- Tied concrete shoulders in conjunction with the construction of a new concrete overlay.

The edge support designs evaluated varied significantly in all aspects. This variability is illustrated in table 12. These projects have been in service from 1 to 21 years at the time of survey.

4.1.3 Traffic and Climatic Variation

In terms of traffic loadings and climatic effects, the pavements associated with edge support projects have withstood from 0.4 million to 2.8 million 18-kip [80 kN] Equivalent Single-Axle Loads (ESALs) for the outer traffic lane while in service. Annual loadings ranged from 0.1 million to 2.7 million ESALs per year in the outer traffic lane. The projects were located in five of the nine climatic regions as defined by Carpenter (see figure 33).(3) The Corps of Engineers Freezing Index varied form 0 to 1750 degree days.

4.2 DATA COLLECTION

The database is comprehensive containing as many projects as was available or that could be included within available resources. These pavements were surveyed between July 1985 and August 1986.

There were five basic data sets that were deemed necessary for the development of life prediction models and for analysis aimed towards the development and improvement of design and construction procedures. These included:

- Field condition data.
- Original pavement structural design and construction and subgrade soil classification.
- Rehabilitation design factors.
- Historical traffic volumes, classifications and accumulated 18-kip [80 kN] equivalent single axle loadings.
- Environmental data.

The data sources and procedures used in the collection of each are described in volume IV.

4.3 PERFORMANCE MODELS

The development of a predictive model to assess the effect of edge support on transverse joint faulting solely on the basis of edge support design and monitoring variables, as well as project design variables, was not possible due to limited number of uniform sections. However, the effect of edge support on joint faulting

Table 12. Design variability by edge support project site.

PROJECT SITE	JOINT SPACING PAVEMENT (feet)	JOINT SPACING SHOULDER (feet)	EDGE SUPPORT TYPE	LANE-SHOULDER TIE SYSTEM	TIE BAR DIAMETER (in)	TIE BAR SPACING (in)	TIE BAR LENGTH (in)	SHOULDER WIDTH (feet)	SHOULDER THICKNESS (in)
TIED RETROFIT EDGE BEAM EDGE SUPPORT									
MINNESOTA, U.S. 10	15.00	15.00	EDGE BEAM	DEFORMED REBAR	0.625	24.0	18.0	2.0	6.00
ARKANSAS, I-30	15.00	15.00	EDGE BEAM	DEFORMED REBAR	0.500	30.0	30.0	3.0	9.00
TIED RETROFIT CONCRETE SHOULDER EDGE SUPPORT									
ILLINOIS, RTE. 116	100.00	100.00	TIED JPCP	DEFORMED REBAR	0.625	30.0	24.0	8.0	6.00
WYOMING, I-80	20.00	20.00	TIED JPCP	DEFORMED REBAR	0.500	24.0	24.0	10.0	8.00
SOUTH CAROLINA, I-20	25.00	25.00	TIED JPCP	DEFORMED REBAR	0.500	30.0	30.0	10.0	9.00
TIED RETROFIT CONCRETE SHOULDER EDGE SUPPORT AS PART OF A CONCRETE OVERLAY									
COLORADO, I-25 (MP 247)	13.50	13.50	TIED JPCP	DEFORMED REBAR	0.500	30.0	30.0	10.0	7.75
COLORADO, I-25 (MP 253)	13.50	13.50	TIED JPCP	DEFORMED REBAR	0.500	30.0	30.0	10.0	6.25
OHIO, I-70	60.00	20.00	TIED JPCP	DEFORMED REBAR	0.625	60.0	30.0	10.0	10.00
WYOMING, I-25	20.00	20.00	TIED JPCP	DEFORMED REBAR	0.500	24.0	24.0	10.0	12.00
PENNSYLVANIA, I-376	30.75	15.00	TIED JPCP	HOOK BOLTS	0.625	30.0	30.0	10.0	10.00
MICHIGAN, U.S. 23	41.00	41.00	TIED JRCP	DEFORMED REBAR	0.625	55.0	24.0	8.0	7.00

** Joint Spacing in Colorado is random: 12-15-13-14, Avg. = 13.5 ft.

** Joints in Colorado, Pennsylvania and Wyoming are skewed 6:1.

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

CLIMATIC ZONE FACTORIAL

TEMPERATURE	PRECIPITATION: WET	WET -DRY	DRY
FREEZE	6	1	4
FREEZE -THAW	1	0	0
NO FREEZE	1	0	0
TOTAL	8	1	4

NOTE: A total of 13 uniform sections were evaluated through condition surveys.

Figure 33. Climatic zone factorial for edge support uniform sections.

was incorporated into the diamond grinding predictive model which is given in chapter 2 (i.e. the edge support database was combined with the diamond grinding database). The benefit associated with the installation of tied concrete shoulders is shown in figures 34 and 35, taken from the diamond grinding chapter, which illustrate the effect that edge support has on joint faulting. Grinding, grinding with load transfer restoration, grinding with shoulders, grinding with subdrains and shoulders and new pavement faulting curves are shown for comparison purposes.

4.4 EDGE SUPPORT CASE STUDIES

A total of thirteen uniform sections were surveyed at eleven project sites. These projects can be broken down into the three categories previously listed. The pertinent original pavement design, overlay design (if applicable), edge support system design, traffic and climatic variables are listed in table 13 through 15. The field performance and evaluation case studies which follow depict the pavement distresses observed at the time of survey. The distresses and severity levels identified in the condition surveys are as defined in reference 1.

The design information was retrieved from as-built plans and rehabilitation special provisions provided by the respective state's departments of transportation. The traffic data was calculated with the aid of State-provided historical traffic records and the use of FHWA W-4 tables. The environmental data was retrieved from publicly available brochures entitled, "Monthly Normals of Temperature, Precipitation, and Heating and Cooling Degree Days, 1951-80" (National Oceanic and Atmospheric Administration), for the weather recording station nearest to the individual project sites.

The following case studies are included to provide specific descriptions of the projects from which the database was developed.

The projects are described as categorized below:

<u>TIED RETROFIT EDGE BEAM EDGE SUPPORT</u>

Minnesota, U.S. Route 10
Arkansas, Interstate 30

<u>TIED RETROFIT CONCRETE SHOULDER EDGE SUPPORT</u>

Illinois, Route 116
Wyoming, Interstate 80
South Carolina, Interstate 20

<u>TIED RETROFIT CONCRETE SHOULDERS AS PART OF A CONCRETE OVERLAY</u>

Colorado, Interstate 25 (MP 247)
Colorado, Interstate 25 (MP 253)
Ohio, Interstate 70
Wyoming, Interstate 25
Pennsylvania, Interstate 376
Michigan, U.S. Route 23

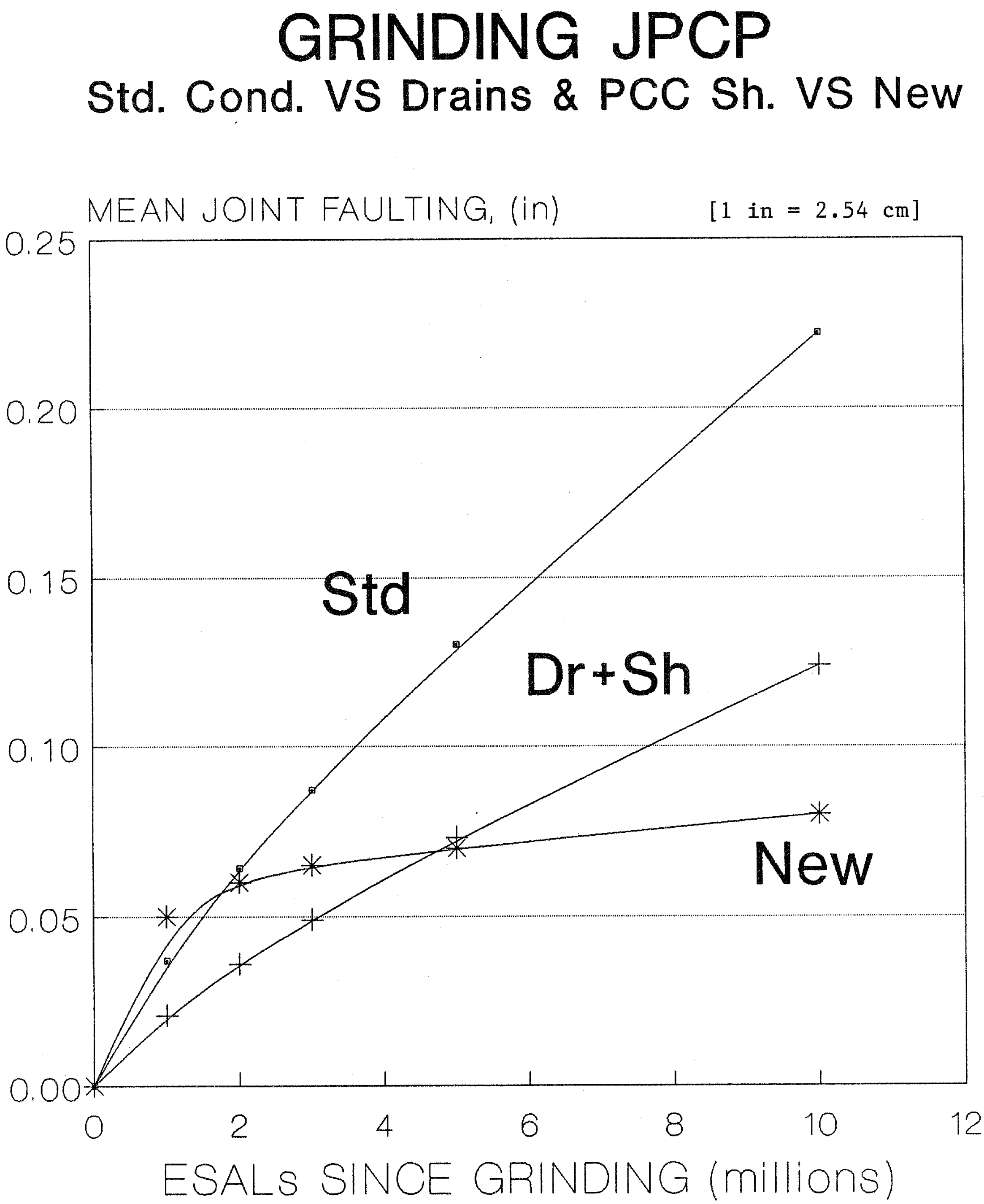


Figure 34. Faulting projections for standard JPCP and standard plus drains and PCC shoulders (Estimated faulting for new section also shown for comparison).

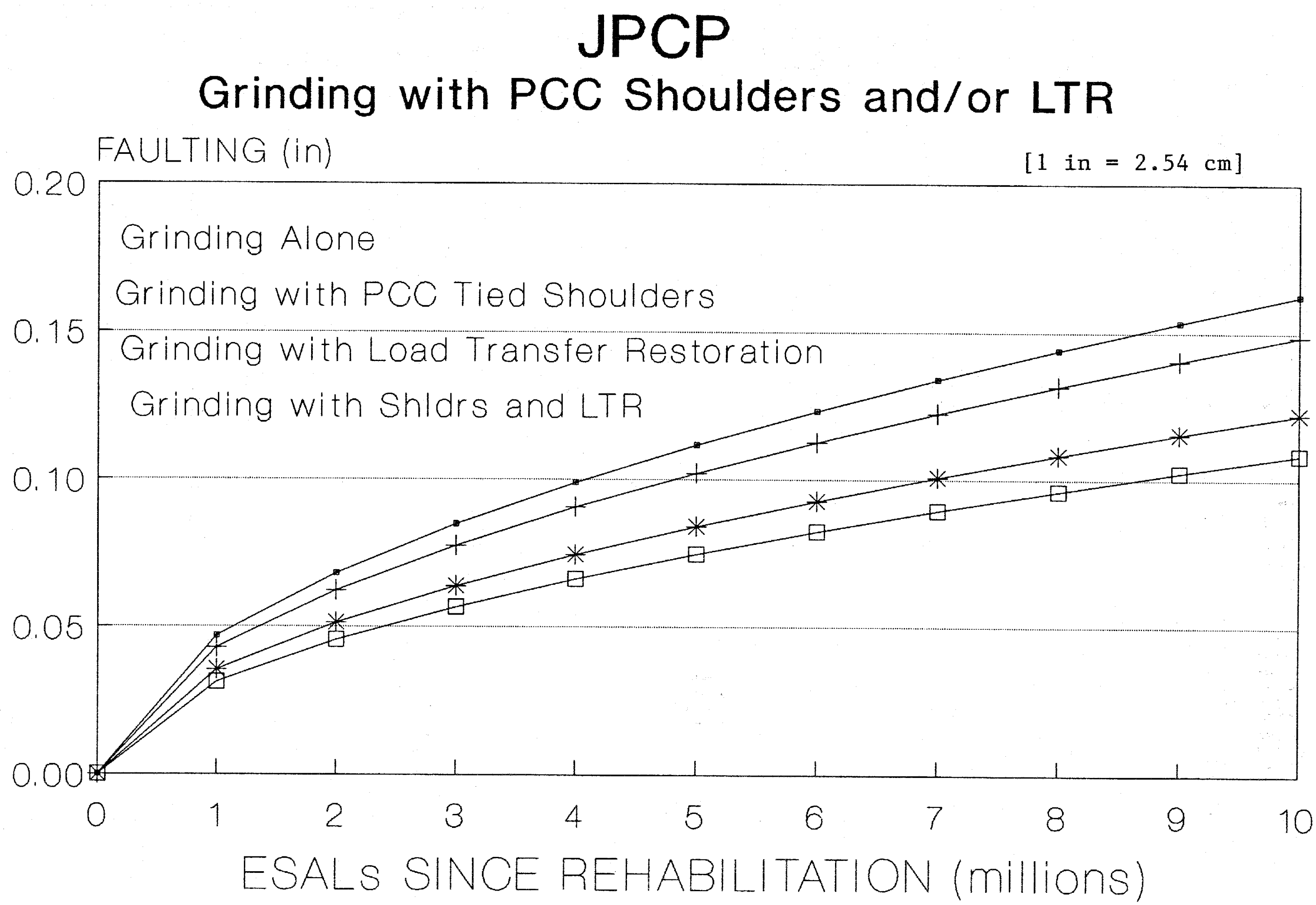


Figure 35. Illustration of predicted faulting for various restoration techniques for JPCP.

Table 13. Edge beam project variability.

PROJECTS INCORPORATING TIED RETROFIT EDGE BEAM EDGE SUPPORT		
PROJECTS	MINNESOTA US 10 near ELK RIVER, MN (MP 204)	ARKANSAS I-30 near BENTON, AR (MP 104)
PAVEMENT DESIGN		
slab	JPCP	JPCP
year constructed	1946	1966
thickness, in.	8	9
joint spacing, ft.	15	15
skewed joints?	NO	NO
load transfer	AGG. INTERLOCK	1-IN. DOWEL EVERY THIRD JOINT
subdrainage?	NONE	NONE
subgrade type	FINE-GRAINED	COARSE-GRAINED
other rehab	NONE	SUBSEAL, FDR, GRINDING DRAINS, JOINT SEALING
TRAFFIC		
current ADT	8800	19200
current % Trucks	23	46
cumulative ESALs on REHAB		
Outer	0.557	2.404
Inner	0.067	0.558
CLIMATE		
climatic zone	WET-DRY/FREEZE	WET/FREEZE-THAW
Freezing Index	1750	0
annual precip, in.	28	52
EDGE SUPPORT DESIGN		
year constructed	1983	1984
joint spacing, ft.	15	15
skewed joints?	NO	NO
matched joints?	YES	YES
lane/beam tie system	DEFORMED REBAR	DEFORMED REBAR
tie bar diameter, in.	0.625	0.5
tie bar length, in.	18	30
tie bar spacing, in.	24	30
beam width, ft.	2	3
beam thickness, in.	6	9
lane/beam joint type	BUTT	BUTT
lane/beam joint seal	SAWED & SEALED	SAWED & SEALED

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

Table 14. Full-width PCC shoulder project variability.

PROJECTS INCORPORATING TIED RETROFIT CONCRETE SHOULDER EDGE SUPPORT			
PROJECTS	ILLINOIS RT. 116 near PEORIA, IL (MP 1)	WYOMING I-80 near LARAMIE, WY (MP 315)	SOUTH CAROLINA I-20 near AUGUSTA, GA (MP 0)
PAVEMENT DESIGN			
slab	JRCP	JPCP	JPCP
year constructed	early 1960'S	1966	1967
thickness, in.	10	8	9
joint spacing, ft.	100	20	25
skewed joints?	NO	SKEWED 6:1	NO
load transfer	1.25-IN. DOWELS	AGG. INTERLOCK	AGG. INTERLOCK
subdrainage?	NONE	NONE	NONE
subgrade type	COARSE-GRAINED	COARSE-GRAINED	COARSE-GRAINED
other rehab	NONE	SUBSEAL, FDR, GRINDING PDR, CRACK&JOINT SEAL	SUBSEAL, FDR, GRINDING PDR, JOINT SEALING
TRAFFIC			
current ADT	6800	8000	18700
current % Trucks	17	25	14
cumulative ESALs on REHAB			
Outer, millions	2.759	1.321	0.361
Inner, millions	0.287	0.161	0.064
CLIMATE			
climatic zone	WET/FREEZE	DRY/FREEZE	WET/NO FREEZE
Freezing Index	500	500	0
annual precip, in.	35	10	43
EDGE SUPPORT DESIGN			
slab	JPCP	JPCP	JPCP
year constructed	1965	1983	1984
joint spacing, ft.	100	20	25
skewed joints?	NO	SKEWED 6:1	NO
matched joints?	YES	YES	YES
lane/shoulder tie system	DEFORMED REBAR	DEFORMED REBAR	DEFORMED REBAR
tie bar diameter, in.	0.625	0.5	0.5
tie bar length, in.	24	24	30
tie bar spacing, in.	30	24	30
shoulder width, ft.	8	10	10
shoulder thickness, in.	6	8	9
lane/shoulder joint type	BUTT	BUTT	BUTT
lane/shoulder joint seal	SAWED & SEALED	FORMED & SEALED	FORMED & SEALED

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

Table 15. Edge support project variability as part of a concrete overlay.

PROJECTS INCORPORATING TIED RETROFIT CONCRETE SHOULDER EDGE SUPPORT AS PART OF A CONCRETE OVERLAY

PROJECTS	COLORADO I-25 N. of DENVER, CO (MP 247)	COLORADO I-25 N. of DENVER, CO (MP 253)	OHIO I-70 near SPRINGFIELD, OH (MP 62)	WYOMING I-25 near DOUGLAS, WY (MP 141)	PENNSYLVANIA I-376 near PITTSBURGH, PA (MP 4)	MICHIGAN US 23 near DUNDEE, MI (MP 10)
ORIGINAL PAVEMENT DESIGN						
slab	JPCP	JPCP	JRCP	JPCP	JRCP	JRCP
year constructed	1964	1964	1968	1968	1946	1959
thickness, in.	8	8	9	9	10	9
joint spacing, ft.	20	20	60	20	90	99
skewed joints?	NO	NO	NO	SKEWED 6:1	NO	NO
load transfer	AGG. INTERLOCK	AGG. INTERLOCK	1.25-IN. DOWELS	AGG. INTERLOCK	1.25-IN. DOWELS	1.25-IN. DOWELS
subdrainage?	NONE	NONE	LONG. EDGE DRAINS	NONE	LONG. EDGE DRAINS	NONE
subgrade type	COARSE-GRAINED	COARSE-GRAINED	FINE-GRAINED	COARSE-GRAINED	FINE-GRAINED	FINE-GRAINED
other rehab	UNBONDED PCC OL	UNBONDED PCC OL	GRINDING (1982) UNBONDED PCC OL	BONDED PCC OL	UNBONDED PCC OL	EDGE DRAINS UNBONDED PCC OL
OVERLAY PAVEMENT DESIGN						
slab	UNBONDED JPCP	UNBONDED JPCP	UNBONDED JPCP	BONDED JPCP	UNBONDED JPCP	UNBONDED JRCP
year constructed	1985	1985	1984	1984	1983	1984
thickness, in.	7.8	6.8	10	3	8	7
joint spacing, ft.	12-15-13-14	12-15-13-14	60	20	30.75	41
skewed joints?	SKEWED 6:1	SKEWED 6:1	NO	SKEWED 6:1	SKEWED 6:1	NO
load transfer	AGG. INTERLOCK	AGG. INTERLOCK	1.625-IN. DOWELS	AGG. INTERLOCK	1.25-IN. DOWELS	1.25-IN. DOWELS
TRAFFIC						
current ADT	25100	25100	28800	4300	67800	21500
current % Trucks	16	16	26	27	8	17
cumulative ESALs on REHAB						
Outer, millions	0.614	0.614	2.66	0.564	1.78	0.909
Inner, millions	0.161	0.161	0.743	0.034	1.131	0.2
CLIMATE						
climatic zone	DRY/FREEZE	DRY/FREEZE	WET/FREEZE	DRY/FREEZE	WET/FREEZE	WET/FREEZE
Freezing Index	250	250	100	700	150	500
annual precip, in.	12	12	37	13	36	33
EDGE SUPPORT DESIGN						
slab	JPCP	JPCP	JPCP	JPCP	JPCP	JRCP
year constructed	1985	1985	1984	1984	1983	1984
joint spacing, ft.	12-15-13-14	12-15-13-14	20	20	15	41
skewed joints?	SKEWED 6:1	SKEWED 6:1	NO	SKEWED 6:1	SKEWED 6:1	NO
matched joints?	YES	YES	NO	YES	NO	YES
lane/shoulder tie system	DEFORMED REBAR	DEFORMED REBAR	DEFORMED REBAR	DEFORMED REBAR	HOOK BOLTS	DEFORMED REBAR
tie bar diameter, in.	0.5	0.5	0.625	0.5	0.625	0.625
tie bar length, in.	30	30	30	24	30	24
tie bar spacing, in.	30	30	60	24	30	55
shoulder width, ft.	10	10	10	10	10	8
shoulder thickness, in.	7.75	6.25	10	12	10	7
lane/shoulder joint type	BUTT	BUTT	BUTT	BUTT	KEYED	BUTT
lane/shoulder joint seal	FORMED & SEALED	FORMED & SEALED	SAWED & SEALED	SAWED & SEALED	SAWED & SEALED	SAWED & SEALED

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

TIED RETROFIT EDGE BEAM EDGE SUPPORT

MN U.S. 10 Elk River

The 8-in [20.3 cm], 15-ft [4.6 m] undowelled JPCP was built in 1946. A 2-ft [0.6 m]-wide, 6-in [15.2 cm] thick, 15-ft [4.6 m] edge beam edge support system was installed in 1983.

The project was in service for 2 years at the time of survey. Two sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in July, 1985.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	296	507	365
	MEDIUM	1236	475	5
	HIGH	0	0	0
Longitudinal Cracking, ft/mile	LOW	253	0	0
	MEDIUM	766	21	0
	HIGH	0	0	0
Transverse Joint Faulting, in		0.07	0.06	0.03

The extent of cracking of the edge beam shoulder after only 2 years is substantial. The narrow width of 2 ft [0.6 m] and thickness of 6 in [15.2 cm] may not be adequate for long-term structural support. The fact that faulting is still occurring in the outer traffic lane indicates that the edge beam has not prevented pumping.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at 0.05 in [0.13 cm], indicating a good tie. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and the edge beam shoulder. Low-severity pumping (slight water erosion) was identified for the outer lane, inner lane and outer shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse "D"-Cracking
Corner Breaks
Reactive Aggregate
Longitudinal "D"-Cracking
Centerline Cracking
Map Cracking

AR I-30 Benton

The 9-in [22.9 cm] thick, 15-ft [4.6 m] JPCP, built in 1966, had 1-in [2.5 cm] dowels at every third joint for load transfer. A 3-ft [0.9 m] wide, 9-in [22.9 cm]-thick, 15-ft [4.6 m] edge beam edge support system was installed in 1984.

The project was in service for 2 years at the time of survey. Two sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in June, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	148	0	0
	MEDIUM	11	0	0
	HIGH	0	0	0
Longitudinal Cracking, ft/mile	LOW	0	0	0
	MEDIUM	69	0	0
	HIGH	0	0	0
Transverse Joint Faulting, in		0.09	0.03	0.02

The lack of cracking of the edge beam indicates its structural adequacy over the heavy truck loadings of two million ESALs in the outer lane. A 3-ft [0.9 m] beam with a 9-in [22.9 cm] thickness appears to be adequate.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at -0.06 in [-0.15 cm], which indicates a good tie. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and edge beam shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse "D"-Cracking
Corner Breaks
Reactive Aggregate
Longitudinal "D"-Cracking
Centerline Cracking
Map Cracking

TIED RETROFIT CONCRETE SHOULDER EDGE SUPPORT

IL Rte. 116 Peoria

The 10-in [25.4 cm] thick, 100-ft [30.5 m] JRCP, built in the early 1960s, had 1.25-in [3.2 cm] dowels for load transfer. An 8-ft [2.4 m]-wide, 6-in [15.2 cm]-thick, 100-ft [30.5 m] undowelled tied concrete shoulder edge support system was installed within 1 to 2 years after original construction along the outer lane only.

The project was in service for 21 years at the time of survey. Two sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in June, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	1811	1748	866
	MEDIUM	317	412	454
	HIGH	32	42	0

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Longitudinal Cracking, ft/mile	LOW	0	42	5
	MEDIUM	0	0	0
	HIGH	0	0	0
Transverse "D"-Cracking, ft/mile	LOW	665	481	164
	MEDIUM	729	1114	159
	HIGH	0	0	0
Transverse "D"-Cracking, % of joints	ALL	100	100	88
Longitudinal "D"-Cracking, ft/mile	LOW	206	0	27
	MEDIUM	0	0	53
	HIGH	0	0	0
Corner Breaks, number/mile	ALL	0	5	11
Transverse Joint Faulting, in		0.05	0.11	0.09

Two items are of importance. The faulting and transverse cracking is greater in the inner lane than the outer lane. This may indicate the beneficial effect of the tied PCC shoulder. Also, there was a lot of cracking on the 100-ft [30.5 m] shoulder, which indicates the problem associated with long joint spacings.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at 0.26 in [0.66 cm], indicating a slight drop-off of the shoulder. Overall transverse joint spalling and corner spalling was evaluated as 100 percent medium-severity for the traffic lanes and 63 percent medium and 37 percent low for the PCC shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder. Also, one sample unit of this project did exhibit medium-severity centerline cracking for the entire length of the 1000-ft [305 m] sample unit.

WY I-80 Laramie

The 8-in [20.3 cm]-thick, 20-ft [6.1 m] undowelled JPCP was built in 1966. The transverse joints were skewed 6 to 1. A 10-ft [3.0 m]-wide, 8-in [20.3 cm]-thick, 20-ft [6.1 m] undowelled tied concrete shoulder edge support system was installed in 1983 with 6 to 1 skewed joints.

The project was in service for 3 years at the time of survey. Two sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in August, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	64	243	180
	MEDIUM	253	137	53
	HIGH	0	0	0

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Longitudinal Cracking, ft/mile	LOW	317	328	0
	MEDIUM	317	317	0
	HIGH	0	0	0
Corner Breaks, number/mile	ALL	48	5	5
Transverse Joint Faulting, in		0.02	0.01	0.01

The cracking data indicates some structural deterioration is occurring; however, it is not known how much existed directly after restoration and how much has occurred since. Shoulder cracking may be due to the long joint spacing of 20 ft [6.1 m] for the thin 8-in [20.3 cm] slab (from curling and warping stresses). Faulting is minimal which may reflect a beneficial effect of the tied PCC shoulder.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at -0.03 in [-0.08 cm], indicating an excellent tie. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and the outer shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

- Transverse "D"-Cracking
- Reactive Aggregate
- Centerline Cracking
- Longitudinal "D"-Cracking
- Map Cracking

SC I-20 Augusta, GA

The 9-in [22.9 cm]-thick, 25-ft [7.6 m] undowelled JPCP was built in 1967. A 10-ft [3.0 m]-wide, 9-in [22.9 cm]-thick, 25-ft [7.6 m] undowelled tied concrete shoulder edge support system was installed in 1984.

The project was in service for 2 years at the time of survey. Two sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in January, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Joint Faulting, in		0.04	0.03	0.03

The fact that no cracking has occurred shows that restoration was clearly appropriate for this project. Faulting is developing, however, and may become substantial as loadings accumulate. This long, undowelled joint spacing in a wet climate with no subdrainage is responsible for the development of faulting. The tied concrete shoulder does not appear to prevent faulting under these conditions.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at 0.01 in [0.025 cm]. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer traffic lane and the PCC shoulder, whereas the inner lane was 87 percent low-severity and 13 percent medium-severity. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse Cracking	Longitudinal Cracking
Transverse "D"-Cracking	Longitudinal "D"-Cracking
Corner Breaks	Centerline Cracking
Reactive Aggregate	Map Cracking

TIED RETROFIT CONCRETE SHOULDERS AS PART OF A CONCRETE OVERLAY

CO I-25 Denver, Milepost 247

The 8-in [20.3 cm]-thick, 20-ft [6.1 m] undowelled JPCP was built in 1964. A 7.8-in [19.8 cm]-thick jointed plain concrete unbonded overlay was installed in 1985. The transverse overlay joints are skewed 6 to 1 and the random joint spacing pattern is 12-15-13-14 ft [3.7-4.6-4.0-4.3 m] (avg. = 13.5 ft [4.1 m]). A 10-ft [3.0 m]-wide, 7.75-in [19.7 cm]-thick undowelled tied concrete shoulder edge support system was installed when the overlay was placed. The shoulder joints have the same random spacing and skewness as the overlay joints.

The project was in service for 1 year at the time of survey. One sample unit was identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in August, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	0	0	254
	MEDIUM	0	0	0
	HIGH	0	0	0
Transverse Joint Faulting, in		0.01	0.01	0.00

The cause of the transverse cracking on the shoulders is unknown. It could be construction-related such as late sawing of the joints. Essentially no faulting has occurred.

In addition to the distresses presented above, overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and the PCC shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder. The mean lane/shoulder drop-off for this project could not be measured since the cross slope of the shoulders was different than that for the mainline pavement.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Longitudinal Cracking
Longitudinal "D"-Cracking
Centerline Cracking
Map Cracking
Transverse"D"-Cracking
Corner Breaks
Reactive Aggregate

CO I-25 Denver, Milepost 253

The 8-in [20.3 cm]-thick, 20-ft [6.1 m] undowelled JPCP was built in 1964. A 6.8-in [17.3 cm]-thick jointed plain concrete unbonded overlay was installed in 1985. The transverse overlay joints are skewed 6 to 1 and the random joint spacing pattern is 12-15-13-14 ft [3.7-4.6-4.0-4.3 m] (avg. = 13.5 ft [4.1 m]). A 10-ft [3.0 m]-wide, 6.25-in [15.9 cm] thick undowelled tied concrete shoulder edge support system was installed when the overlay was placed. The shoulder joints have the same random spacing and skewness as the overlay joints.

The project was in service for 1 year at the time of survey. One sample unit was identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in August, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Joint Faulting, in		0.01	0.00	0.00

No cracking and essentially no faulting has occurred. In addition, overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and the PCC shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder. The mean lane/shoulder drop-off for this project could not be measured since the cross slope of the shoulders was different than that for the mainline pavement.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse Cracking
Transverse"D"-Cracking
Corner Breaks
Reactive Aggregate
Longitudinal Cracking
Longitudinal "D"-Cracking
Centerline Cracking
Map Cracking

OH I-70 Springfield

The 9-in [22.9 cm]-thick, 60-ft [18.3 m] JRCP was built in 1968 and utilized 1.25-in [3.2 cm] dowels for load transfer. A 10-in [25.4 cm]-thick, 60-ft 18.3 m] jointed plain concrete unbonded overlay was installed in 1984, which used 1.625-in [4.1 cm] dowels for load transfer. A 10-ft [3.0 m] wide, 10-in [25.4 cm]-thick undowelled tied concrete shoulder edge support system was installed when the overlay was placed. The shoulder joints are not matched with a joint spacing of 20 ft [6.1 m].

The project was in service for 1 year at the time of survey. Two sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in August, 1985.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	565	95	27
	MEDIUM	0	0	0
	HIGH	0	0	0
Transverse Joint Faulting, in		0.01	NA	0.00

Some low-severity cracking has occurred in the traffic lanes because of the long joint spacing. The shoulder is performing well. Practically no faulting has occurred on this heavily trafficked highway.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at -0.07 in [-0.18 cm], which indicates an excellent tie. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and PCC shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

- Longitudinal Cracking
- Longitudinal "D"-Cracking
- Centerline Cracking
- Map Cracking
- Transverse "D"-Cracking
- Corner Breaks
- Reactive Aggregate

WY I-25 Douglas

The 9-in [22.9 cm]-thick, 20-ft [6.1 m] undowelled JPCP was built in 1968. A 3-in [7.6 cm]-thick, 20-ft [6.1 m] jointed plain concrete bonded overlay was installed in 1984. A 10-ft [3.0 m]-wide, 12-in [30.5 cm] thick undowelled tied concrete shoulder edge support system was installed when the overlay was placed. The original pavement, overlay and shoulder joints are all skewed 6 to 1.

The project was in service for 2 years at the time of survey. One sample unit was identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in June, 1986.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	887	0	0
	MEDIUM	0	0	0
	HIGH	0	0	0

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Longitudinal Cracking, ft/mile	LOW	2535	211	0
	MEDIUM	0	0	0
	HIGH	0	0	0
Transverse Joint Faulting, in		0.01	0.01	0.00

A substantial amount of low-severity cracking has occurred in the outer traffic lane. This could in part be reflection cracking. No cracking exists on the 12-in [30.5 cm]-thick shoulder. Faulting is also negligible. The longitudinal cracking in the outer lane appears to be the result of late sawing of the centerline joint.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at -0.24 in [-0.61 cm], which indicates some tie problems or differential construction problems. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and concrete shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder. Map cracking was identified covering a full-lane width for a distance of 390 ft [118.9 m]; this, therefore, corresponds to approximately 2059 ft [627.6 m] of map cracking per mile for a full-lane width.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse "D"-Cracking
Corner Breaks
Reactive Aggregate
Longitudinal "D"-Cracking
Centerline Cracking

PA I-376 Pittsburgh

The 10-in [25.4 cm]-thick, 90-ft [27.4 m] JRCP was built in 1946 and utilized 1.25-in [3.2 cm] dowels for load transfer. An 8-in [20.3 cm]-thick, 30.75-ft [9.4 m] jointed plain concrete unbonded overlay was installed in 1983, which used 1.25-in [3.2 cm] dowels for load transfer. A 10-ft [3.0 m] wide, 10-in [25.4 cm]-thick undowelled tied concrete shoulder edge support system was installed when the overlay was placed. The shoulder joints are not matched with a joint spacing of 15 ft [4.6 m]. Also, the overlay and shoulder joints are skewed 6 to 1.

The project was in service for 2 years at the time of survey. Four sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in August, 1985. Distress quantities for the inner lane were not measured since the high traffic volume and the presence of a concrete raised median posed too great of a safety hazard for the condition survey crew.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Cracking, ft/mile	LOW	618		40
	MEDIUM	0		159
	HIGH	0		0

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Longitudinal Cracking, ft/mile	LOW	0		27
	MEDIUM	0		0
	HIGH	0		0
Transverse Joint Faulting, in		0.04		0.03

Some cracking has occurred in the shoulder. This could be due to construction joint sawing problems since the shoulder appears to be of adequate structure and joint spacing. Some traffic may be using the shoulder as indicated by the 0.03 in of faulting.

In addition to the distresses presented above, mean lane/shoulder drop-off was measured at 0.07 in [0.18 cm], which indicates an excellent tie. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer traffic lane and the PCC shoulder. Low-severity pumping was identified for the outer traffic lane and the PCC shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse "D"-Cracking	Longitudinal "D"-Cracking
Corner Breaks	Centerline Cracking
Reactive Aggregate	Map Cracking

MI U.S. 23 Dundee

The 9-in [22.9 cm] thick, 99-ft [30.2 m] JRCP was built in 1959 and utilized 1.25-in [3.2 cm] dowels for load transfer. An 7-in [17.8 cm] thick, 41-ft [12.5 m] jointed reinforced concrete unbonded overlay was installed in 1984, which used 1.25-in [3.2 cm] dowels for load transfer. An 8-ft [2.4 m] wide, 7-in [17.8 cm] thick undowelled tied concrete shoulder edge support system was installed when the overlay was placed. Also, the shoulder joints are spaced every 41 ft [12.5 m].

The project was in service for 1 year at the time of survey. Three sample units were identified and evaluated for pavement condition distresses. The following list is a summary of the pavement condition as determined by the condition survey crew in July, 1985.

DISTRESS	SEVERITY LEVELS	OUTER LANE	INNER LANE	SHOULDER
Transverse Joint Faulting, in		0.01	0.01	0.03

No cracking and essentially no faulting has occurred. In addition, mean lane/shoulder drop-off was measured at -0.02 in [-0.05 cm], which indicates an excellent tie. Overall transverse joint spalling and corner spalling was evaluated as 100 percent low-severity for the outer and inner traffic lanes and the PCC shoulder. Low-severity pumping was identified for the outer lane, inner lane and outer shoulder.

The following distress were not observed on the outer lane, inner lane or shoulder of this project:

Transverse Cracking	Longitudinal Cracking
Transverse "D"-Cracking	Longitudinal "D"-Cracking
Corner Breaks	Centerline Cracking
Reactive Aggregate	Map Cracking

4.5 DESIGN AND CONSTRUCTION GUIDELINES -- EDGE SUPPORT

4.5.1 Introduction

These guidelines were originally prepared under NCHRP Project 1-21 published in NCHRP Report No. 281, Transportation Research Board, 1985. Further updates resulted from the research conducted under this study, which is described in this report.

Need For Edge Support

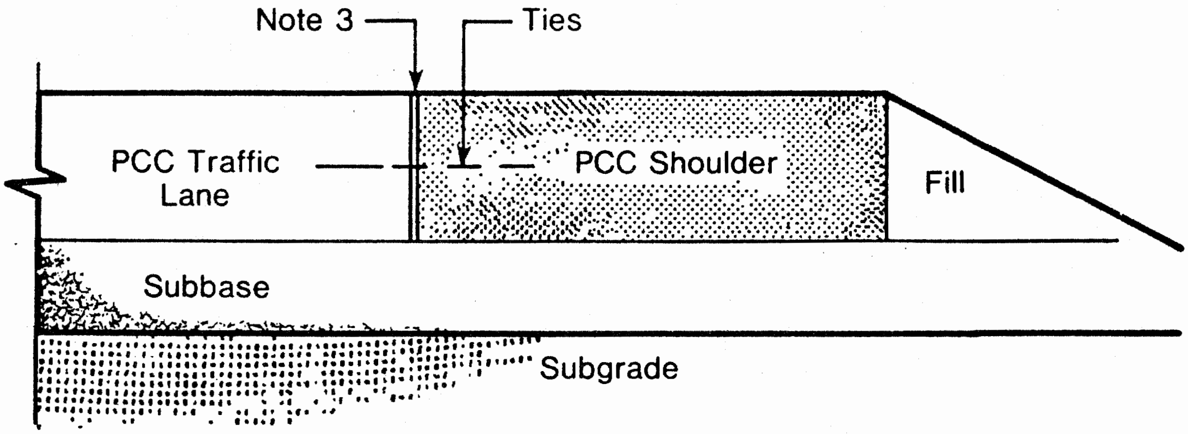
Many concrete pavements exhibit distress resulting from loss of support beneath the slab edge and transverse joint. The major cause of this support loss is heavy repeated truck loads and the infiltration of water into the pavement system (particularly along the shoulder joint) and the subsequent erosion of the base and/or subgrade material. This causes an increase in the corner and edge deflections of the slab which results in faulting, corner breaks, transverse and longitudinal cracking.

One approach to the reduction of these types of distresses would be the construction of a rigid edge support. The major objective of providing increased edge support for an existing pavement is to reduce slab edge and corner deflections (as well as stresses) by providing either a slab edge beam or a tied shoulder.(25,26) Another benefit is the reduction of moisture entering the pavement directly at the slab edge. Examples of different design concepts are shown in figure 36. Type I represents a typical PCC shoulder, and Type II is a much narrower edge "beam" tied to the slab.

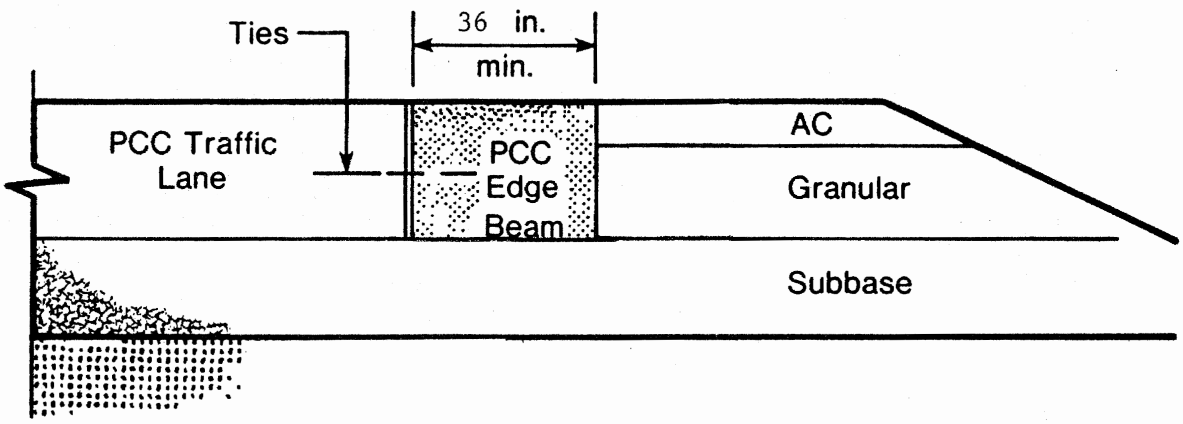
The need for improved edge support depends directly on the extent of damage occurring in the traffic lane from traffic. If this is extensive, then the improved edge support should have a beneficial effect in reducing this deterioration.

Effectiveness

The effectiveness of increased edge support depends upon the reduction in edge deflection and critical stresses. To investigate the effectiveness of the edge support techniques, the ILLI-SLAB finite element program was used.(27) This program was developed for the analysis of a variety of jointed concrete pavement systems. ILLI-SLAB is capable of analyzing the behavior of pavements utilizing various types of load transfer systems such as dowel bars or tie systems, aggregate interlock, or a combination of both. The model is also capable of handling the effect of a stabilized base on the structural response of the pavement system. The model has been verified by comparison with the available theoretical solutions and results from field experimental studies.



TYPE I-PCC SHOULDER EDGE SUPPORT



TYPE II-PCC EDGE BEAM SUPPORT

Notes:

(1) Ties should be No. 5 deformed rebar or equivalent at middepth of slab.

(2) Existing shoulder to be removed to the extent required.

(3) Joint between traffic lane and shoulder should be either edged, or a reservoir that is formed or sawed and then sealed.

(4) 1 in = 2.54 cm

Figure 36. Diagram of different edge support designs.(6)

Figure 37 shows the effect of edge support on edge deflection. A concrete shoulder with a strong tie that provides 100 percent deflection load transfer efficiency reduces the deflection by one-half as shown. Figure 38 shows the effect of both the joint load transfer efficiency and the width of the shoulder. Again, good load transfer reduces the stress by one-half. The width of the support beam/shoulder has a major effect from 1 to 3 ft [0.3 to 0.9 m].

Using ILLI-SLAB to illustrate the effectiveness of the edge beam in decreasing the critical edge deflections and stresses of a pumping pavement, a void was placed beneath the slab at the joint under the corner of the leave slab. Initially, the corner deflection of the leave slab was computed using the finite-element analysis to determine the response of the system before the edge beam was placed. Then the corner deflection was calculated for varying shoulder widths and undercut lips of the edge beam. A slab thickness of 9 in [22.9 cm] with a granular and stabilized subbase was used as an example.

Results from this analysis shows that the edge support concept could substantially decrease critical edge and corner deflections and stresses in pavements even when voids are present (however, voids must always be filled). For example, figure 39 shows that a 9-in [22.9 cm] slab with a stabilized subbase and a void beneath the corner had a corner deflection of 0.047 in [0.119 cm] under a 9-Kip [40 kn] wheel load. The attachment of a 24-in [61.0 cm]-wide edge beam with a depth of 9 in [22.9 cm] reduced the deflection to 0.023 in [0.058 cm], or 50 percent. If the edge beam was thickened to 15 in [38.1 cm] and undercut the slab 6-12 in [15.2-30.5 cm], the corner deflection was reduced to 0.018 in [0.046 cm], or a 62 percent reduction. Increasing the edge beam width to 48 in [121.9 cm] decreases the deflection more, but at a decreasing rate. The effect of an edge beam on a 9-in [22.9 cm] slab with a granular subbase is shown in figure 40. The effect is similar to a stabilized subbase.

These reductions in deflection may be beneficial; however, they may not be adequate to prevent pumping.

4.5.2 Concurrent Work

The effectiveness of the edge support can be enhanced by the application of several other repair methods. One method which should be applied along with the installation of the edge support to decrease pavement deflections even further is restoration of support by subsealing of voids. This should be accomplished after the edge support has been placed. Slab replacement, spall repair, grinding and joint resealing may also be accomplished at the same time depending on the overall pavement condition. The combination of these repair methods could serve to substantially increase the service life of a jointed concrete pavement.

The need for subdrainage must be considered to remove free water that infiltrates at the edge joint. If placed, the longitudinal pipe should be placed along the outer lane-shoulder edge joint at the bottom of the beam. A longitudinal drain may also be needed along the inner lane depending on cross slope (such as at a superelevated curve).

Note: 1 in = 2.54 cm
1 ft = 0.3048 m

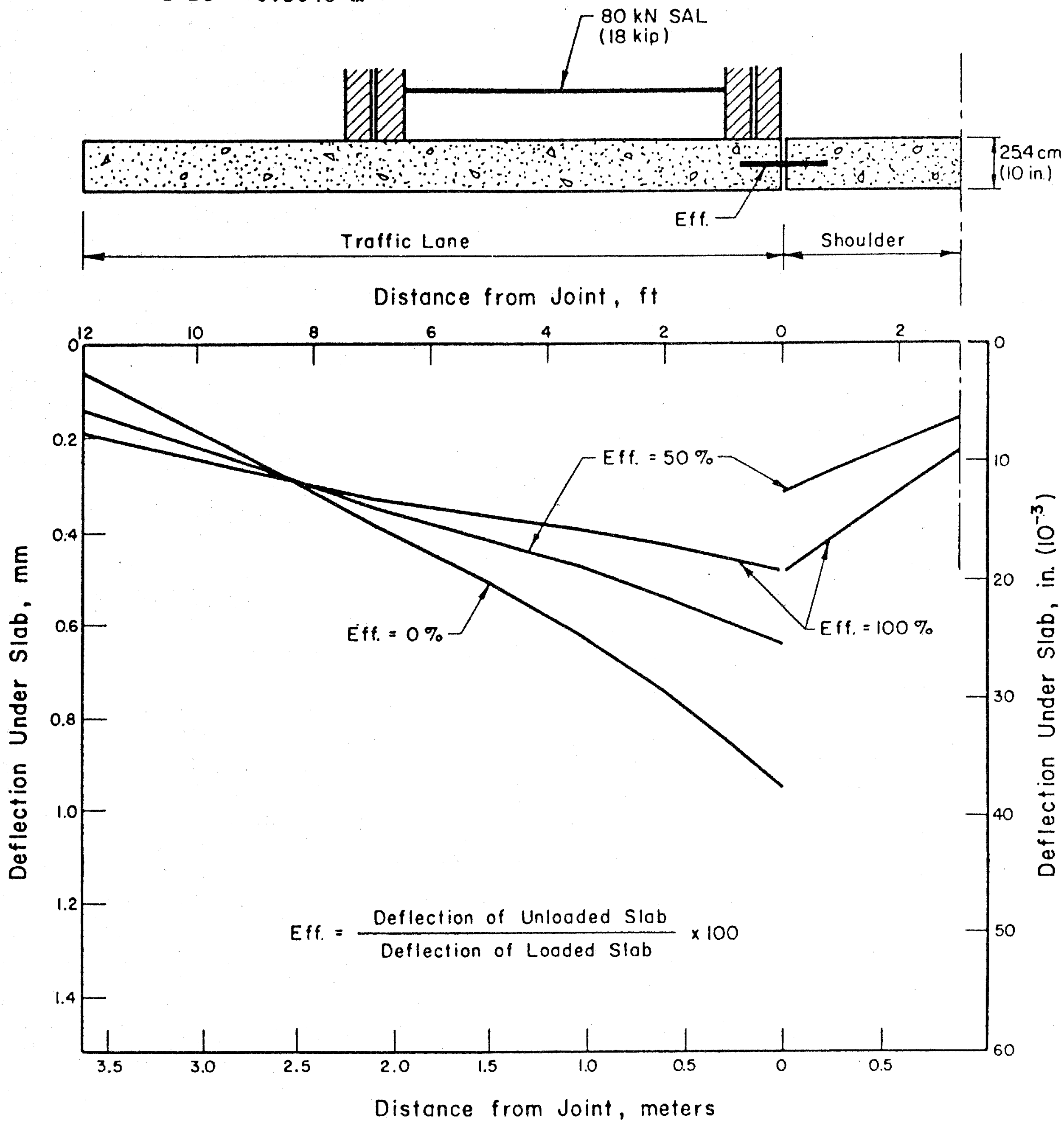


Figure 37. Effect of lane/shoulder tie on deflection of PCC traffic lane.(6)

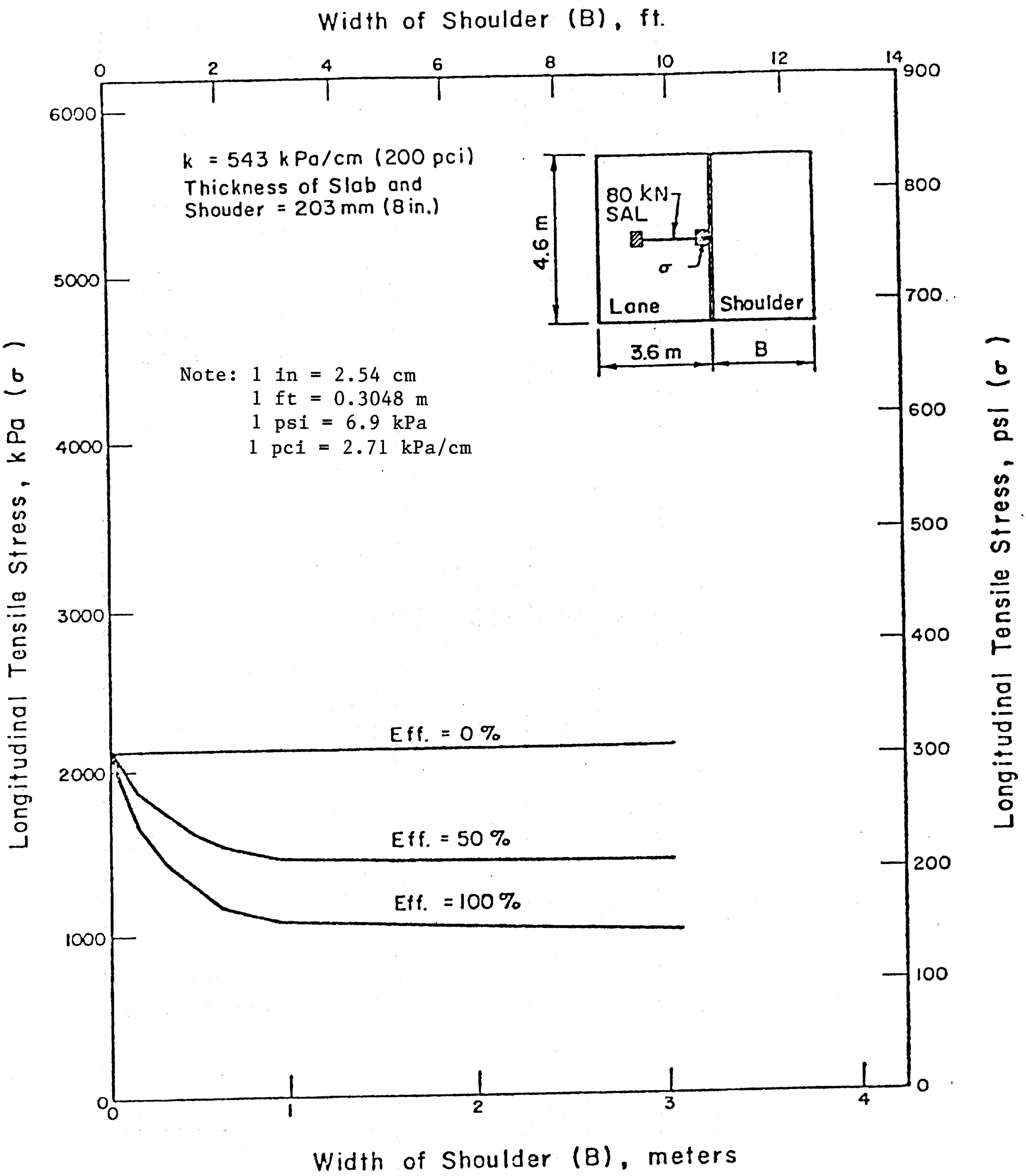


Figure 38. The effect of lane/shoulder tie and width of PCC shoulder on tensile stress of traffic lane.(6)

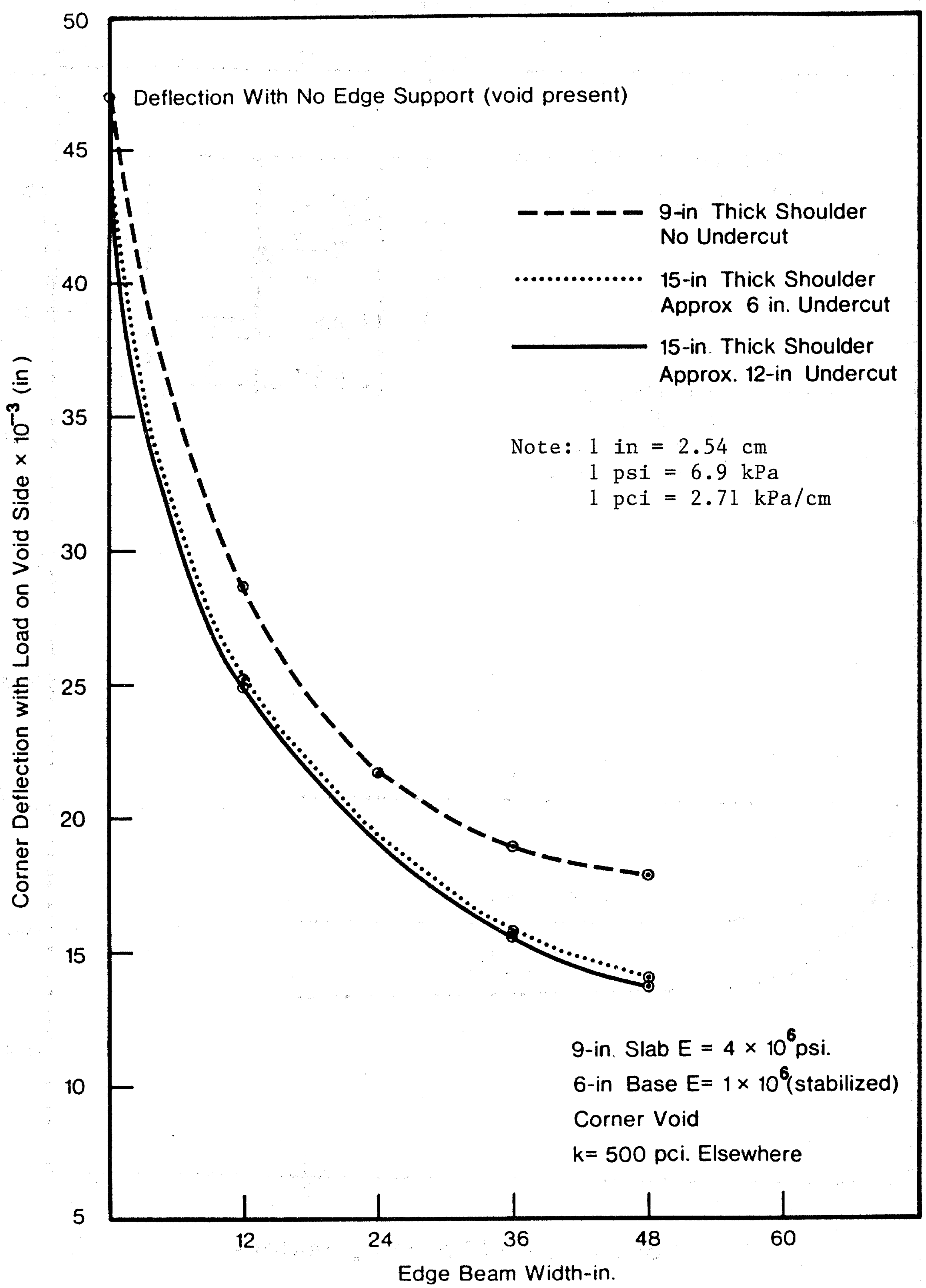


Figure 39. Effect of edge support on slab corner deflection with a stabilized subbase.(6)

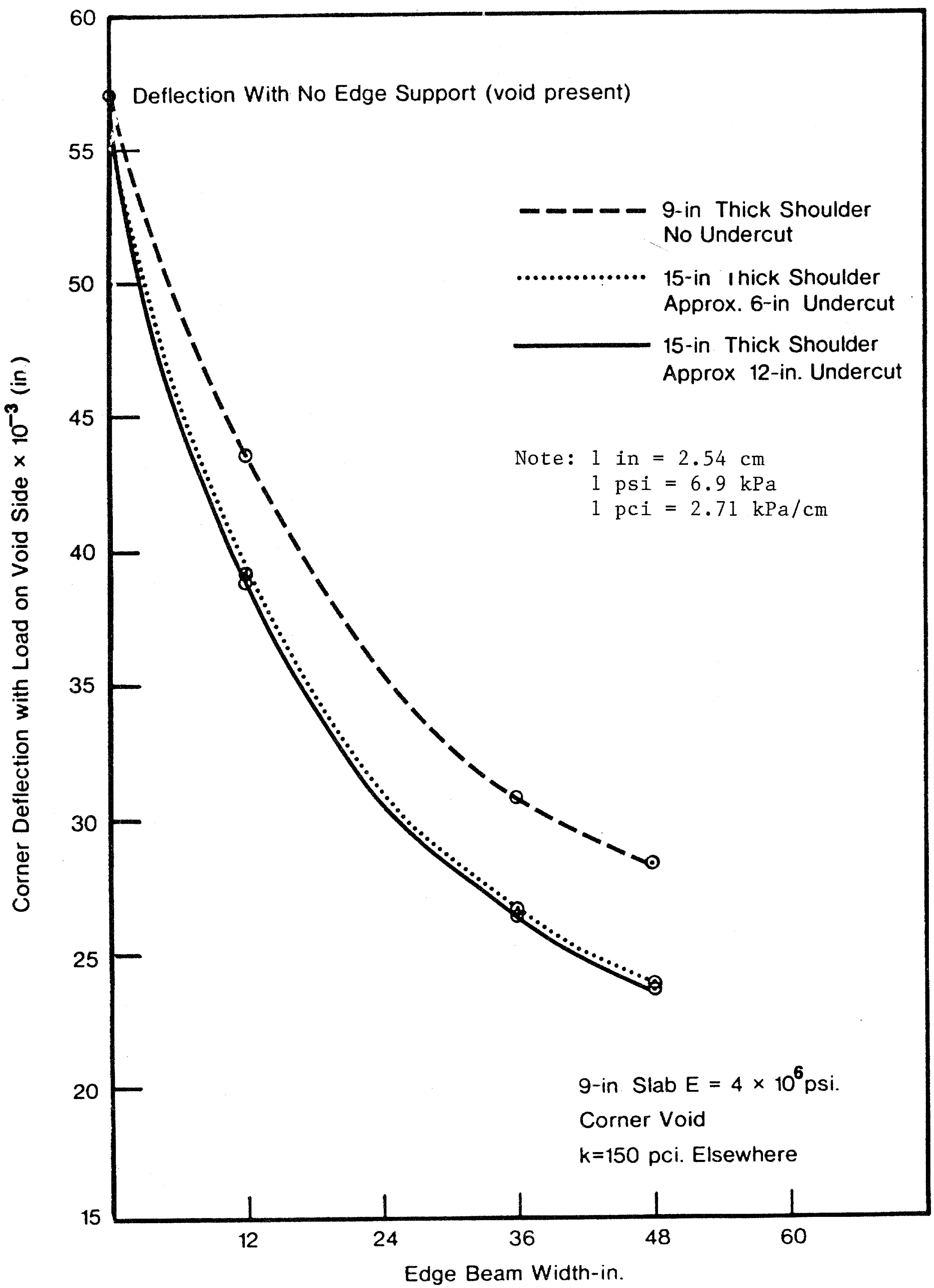


Figure 40. Effect of edge beam on slab corner deflection with a granular subbase.(6)

4.5.3 Design

General

There are two different types of edge support designs shown in figure 36:

- A full-width concrete shoulder.
- A narrow beam attached to the edge of the slab.

The selection of one or the other is a matter of cost and condition of the existing shoulder. If the existing shoulder is deteriorated, a full-width PCC shoulder may be the most cost effective since extensive shoulder rehabilitation will be required anyway. If the shoulder is in good condition, the narrow edge beam may be the most cost effective (although this is not always the case due to the quantities of materials and cost trade-offs involved).

Shoulder Design (Full Width)

The design of the PCC concrete shoulder involves selecting its thickness, tapering (if any), transverse joint spacing and load transfer, and the lane/shoulder tie system. A detailed design procedure is provided in references 25 and 30. A summary of design recommendations are as follows:

1. A jointed plain concrete pavement (JPCP) shoulder is recommended. The slab thickness of the shoulder can be designed by considering fatigue damage which generally shows that the outside edge is critical because of parking truck traffic.(25) A slab thickness equal to that of the main line and tapering somewhat to the outside edge to account for the normal increased slope of the shoulder may be the most cost effective design. The bottom of the shoulder slab would extend directly out as shown in figure 36. The outer edge must be at least 6 in [15.2 cm] thick and thicker if much heavy-truck parking is expected. An abrupt change in shoulder thickness at the lane/shoulder interface may result in differential frost heave. The subbase must not be a frost susceptible material (in deep frost areas).

2. Transverse joints should be weakened plain type with no mechanical load transfer, unless the shoulder will carry heavy traffic for lane closures for a significant time period. Joint spacing is critical and should be limited to avoid cracking from thermal curling. The author's experience indicates that a maximum joint spacing of 1.5 to 1.75 times the slab thickness in inches is recommended as general guidance:

Slab Thickness	Maximum Joint Spacing
6 in [15.2 cm]	9 to 10.5 ft [2.7 to 3.2 m]
8 in [20.3 cm]	12 to 14.0 ft [3.7 to 4.3 m]
10 in [25.4 cm]	15 to 17.5 ft [4.6 to 5.3 m]

Each joint and joint type in the adjacent traffic lane must be matched with a similar joint in the shoulder (e.g., expansion joints must be extended into the shoulder). If the traffic lane slab was 30 ft [9.1 m] long and the shoulder slab was selected to be 9 in [22.9 cm] thick to match the traffic lane, the shoulder joint spacing recommended would be 15-ft [4.6 m], for example. If the shoulder thickness was 6 in [15.2 cm], the recommended joint spacing would be 10 ft [3.0 m].

3. The lane/shoulder tie system is crucial to the success of the edge support design. Good load transfer can be achieved by placing deformed rebars across the joint. After drilling holes into the existing slab, the bars must be installed in the holes with epoxy or a nonshrinkage cement grout. The embedment length of the bars in the existing slab and the new shoulder should be adequate to develop full bar yield strength. This would be 8 in [20.3 cm] minimum for a No. 4 bar and 10 in [25.4 cm] minimum for a No. 5 bar according to the ACI Code (0.0004* Bar Diameter * fy) for a Grade 40 bar. To ensure that an adequate strength is obtained, minimum pull out loads should be based on the yield strength of the reinforcement bars.

   Malleable tie bars of small diameter (No. 4 or 5) spaced 12-24 in [30.5-61.0 cm] at midslab depth are preferable as shown in figure 41. In areas where deicing salts are used, the bars should be coated with a corrosion resistant coating. Other <u>means</u> of tying the shoulder to the traffic lane (such as a 5/8-in [1.6 cm] round tie bolt with a hook) should be fully tested to ensure full bar yield strength development.

<u>Edge Beam Design</u>

The edge beam design is similar to the PCC shoulder design except that it is much narrower than the shoulder and can be thicker than the traffic lane slab. From the analysis performed with varying widths of the undercut lip, it was concluded that the corner deflection was not very sensitive to this parameter. Thus, due to this and obvious construction and subsurface drainage difficulties, the undercut is not recommended. This is not to say that the undercut is not important, it may be helpful in assuring long-term high load transfer efficiency across the shoulder joint.

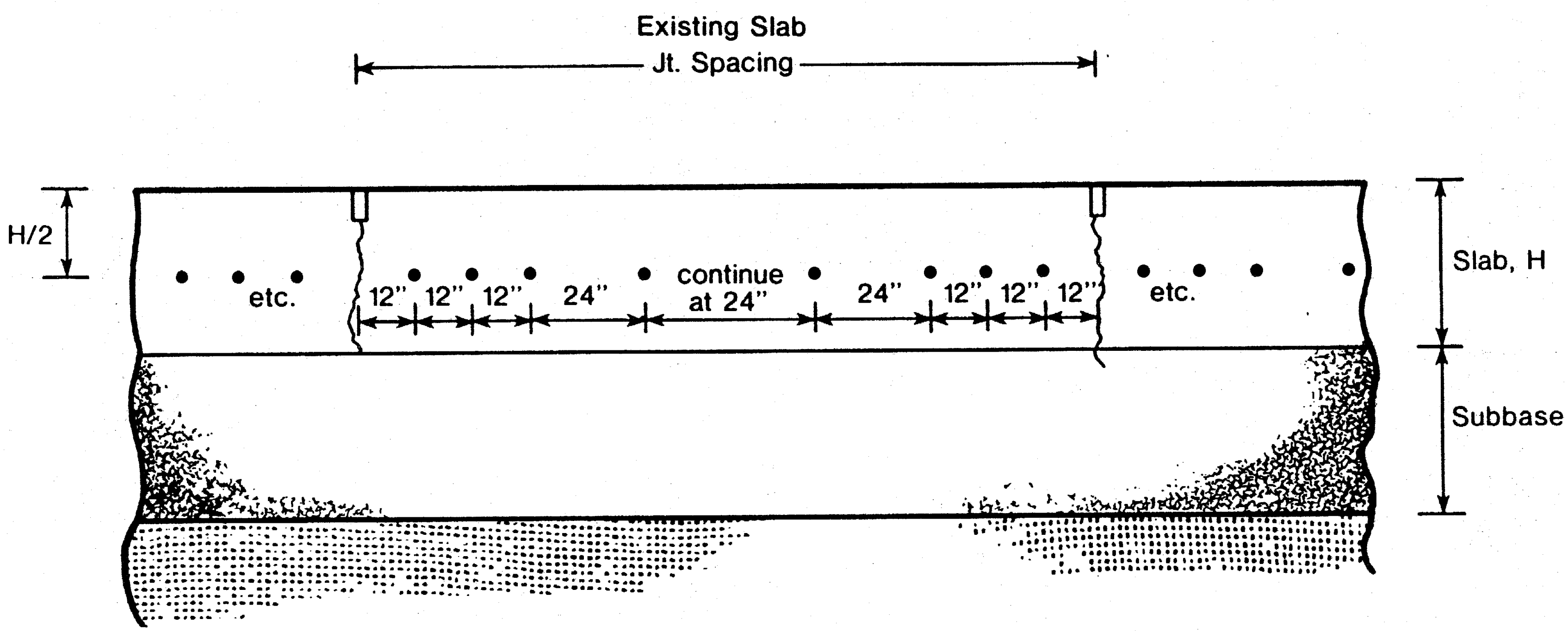
The two critical design parameters are the edge beam width and its thickness. Field performance of edge beams indicate that the width should be at least 36 in [91.4 cm]. to limit transverse cracking of the beam from heavy encroaching truck loads. The finite-element analysis showed that the beam should be at least 24 in [61.0 cm] wide to contribute significant structural benefit to the traffic lane.

The depth of the beam should be at least the thickness of the slab. The edge beam should be jointed to prevent thermal curling stresses and to match the existing pavement. Weakened-plane contraction joints, perhaps with dowels for structural stability, should be formed as soon as possible after placement. Figure 41 illustrates the joints and tie bar design recommended by NCHRP 1-21 to provide improved corner load transfer.(32)

A critical part of the edge beam concept is the design and installation of the tie system. The purpose of the tie system is to provide the best possible joint load transfer across the lane/beam joint.

<u>Sealing Longitudinal Joint</u>

It is recommended that the joint between the existing slab and edge support be sawed to provide the recommended reservoir dimensions for the chosen sealant. The transverse joints in the shoulder/edge beam should also be sealed.



Note: 1 in = 2.54 cm

Figure 41. Recommended tie bar spacing (No. 5 deformed rebar or equivalent).(6)

4.5.4 Construction

<u>Procedures</u>

Since the edge beam is a new concept there is no tested procedure for their installation, although the procedures used in constructing concrete shoulders on an existing traffic lane would be similar. The following should be considered in the construction of edge beams and PCC shoulders.

It is important that the base be in good condition. If the base material is disturbed during excavation, it should be adequately recompacted. Settlement of the shoulder/beam can produce very high "pullout" stresses in the joint tie system. The magnitude of these stresses may be sufficient to exceed the strength of the tie bars and drastically decrease the edge support effectiveness.

Holes are drilled into the existing slab for the tie bars. Epoxy or grout can be used to secure the tie bars in these holes in the existing slab. The holes must be placed at slab mid-depth. Great care must be taken to ensure that the deformed tie bars are adequately anchored in the existing slab. A minimum pull out strength that is equal to the yield strength of the bars used is required.(28) The grout must be a nonshrinking grout.

After the bars have been secured and the shoulder area is prepared, the fresh concrete should be placed.

The texturing of the edge beam or shoulder should be different than the pavement and rumble stripes placed, if possible. Drivers should be able to differentiate between the traffic lane and the shoulder. The edge beam should be textured perpendicular to the traffic lane. If the lane is textured longitudinally, the edge beam should be textured transversely, and vice versa.

4.5.5 Preparation of Plans and Specifications

The plans should clearly show the areas where edge support is to be placed. A diagram showing the cross section with dimensions must be provided as well as specifics on transverse joints and the lane/shoulder longitudinal joint.

4.6 CONCLUSIONS AND RECOMMENDATIONS

The outer traffic lane edge and corner have long been identified as critical locations for high stresses and deflections. The outer edge develops high stresses and usually become the critical fatigue damage point where transverse cracks initiate and work across the traffic lane. The outer corner develops high deflections that results in pumping and subsequently faulting, loss of support and corner breaks/diagonal cracks.

Tied concrete shoulders have been shown to reduce the corner deflection and the edge stress produced by edge wheel loading. It is theorized that this reduction in deflections and stresses will result in a life extension to the mainline pavement. In addition, the expected benefits also may include a more reliable longitudinal lane-shoulder joint for effective joint reservoir construction and sealing which reduces the amount of water that can enter the pavement structure and deteriorate the underlying structural or supporting layers. Another benefit is a long lasting low maintenance shoulder pavement. These benefits, if true, are significant enough to warrant the consideration of edge support as a rehabilitation alternative for rigid pavements of sound concrete.

While there has been some field evidence that tied PCC shoulders are beneficial for new designs, there has not been field evidence that retrofit PCC shoulders have the same effect. The major concern is that the tie between the lane and shoulder is adequate to provide substantial load transfer. If load transfer is lost over time, the PCC shoulder will not have a significant effect on the traffic lane. The shoulder may separate greatly, eliminating the possibility of sealing the joint.

Thirteen uniform sections of edge support were located and surveyed in nine States. These uniform sections were broken down into 22 sample units that were up to 1000-ft [305 m] long, where possible. Edge support was found to exist in the following main categories:

- Edge beam (narrow strip of PCC about 2 to 3 ft [0.6 to 0.9 m] wide tied to the existing traffic lane) (two uniform sections).
- Tied retrofit PCC shoulder (three uniform sections).
- Tied concrete shoulders in conjunction with the construction of a new concrete overlay (six uniform sections).

The edge support designs evaluated varied significantly in all aspects. The major design variables are summarized as follows:

Design Factor	Mean	Range
Shoulder width, ft	9.6	8 - 10
Edge beam width, ft	2.5	2 - 3
Thickness, in	8.3	6 - 12
Tie bar spacing, in	33.4	24 - 60
Tie bar diameter, in	0.56	0.5 - 0.625
Tie system	All deformed rebar, except one hook bolt	
Joint spacing, ft		
JPCP	17.4	13.5 - 25
JRCP	70.5	41.0 - 100

Note: 1 in = 2.54 cm; 1 ft = 0.3048 m.

No control sections were monitored in this study; therefore, direct comparisons showing the effect of the edge support could not be made. These projects have been in service from 1 to 21 years at the time of survey. One interesting example can be cited. The oldest section of retrofit PCC shoulders in the U.S. on Route 116 in Illinois was included in the database. This section showed the following cracking and faulting after 21 years:

Traffic Lane	ESAL	Faulting, in	Deteriorated Cracks, ft/mile
Outer (with PCC sh.)	2,759,000	0.05 [0.13 cm]	317 [59.9 m/km]
Inner	287,000	0.11 [0.28 cm]	412 [77.9 m/km]

The inner lane has much greater cracking and faulting than the outer lane which was tied to the PCC shoulder, despite having about one-tenth the traffic.

Edge support overall conclusions and recommendations from this research study are as follows:

1. The deterioration identified on the PCC shoulders is summarized as follows:

Distress Type	Severity	Mean	Range
Transverse Cracking	Low	137	0 to 1120 ft/mile
	Medium	70	0 to 634 ft/mile
Longitudinal Cracking	Low	5	0 to 106 ft/mile
Corner Breaks	All	1	0 to 11 /mile
"D" Cracking	All	8 percent sample units	
Pumping	None	100 percent sample units	
Joint Spalling	Low	95 percent sample units	
	Medium	5 percent sample units	
Faulting, in	Mean	0.03	0 to 0.18 in
Lane-Shoulder Dropoff	Mean	0.03	-0.5 to 0.53 in

Note: 1 in = 2.54 cm; 1 ft/mile = 0.189 m/km.

These results show that the mean distresses for all of the sections were generally minor; and overall excellent performance was achieved. The upper range of a few distress types indicates that a few PCC shoulders had some deterioration (e.g., transverse cracking, faulting, and lane-shoulder dropoff). This was primarily the 21-year-old Illinois PCC shoulder on Rt. 116 and the Pennsylvania I-376 project that had differential frost heave.

The mean age of the PCC shoulders was 3.5 years, with a range of 1 to 21 years. The mean ESAL carried by the outer lane with PCC shoulders was 1.4 million with a range of 0.4 to 2.8 million.

2. The predictive faulting model developed using all projects that had been diamond ground (both with and without tied PCC shoulders) showed that tied concrete shoulders had about a 9 percent effect on reducing faulting (see figure 35). The data was very limited and thus this effect must be considered as very approximate.

3. Edge support improvement at the time of restoration should be considered on highways that are exposed to high volumes of heavy truck traffic where one or more of the following conditions exist:

   - Existing shoulder is deteriorated and needs replacement.
   - Significant distress has developed in the outer traffic lane due to edge loadings.
   - The PCC outer traffic lane does not have serious durability problems.

4. The effectiveness of the edge support can be enhanced by the application of several other repair methods, including restoration of support by subsealing of voids, subdrainage pipes along the slab edge and joint resealing.

5. The selection of edge beams or shoulders is a matter of the costs, condition of the existing shoulder and geometrics (e.g., use of the shoulder as a temporary traffic lane). If the existing AC shoulder is deteriorated, a full-width PCC shoulder may be the most cost effective, since extensive shoulder rehabilitation will be required anyway. The quantity of PCC will also dictate the cost effectiveness of an edge beam versus a regular tied PCC shoulder.

6. A jointed plain concrete pavement shoulder is recommended. The slab thickness of the shoulder can be designed using fatigue consideration; however, it is recommended that a shoulder slab thickness equal to that of the outer traffic lane be utilized. It may taper somewhat at the top to the outer edge to provide an increased slope. The bottom of the shoulder slab would extend directly out to provide free movement of infiltrated water. The outer edge must be at least 6 in [15.2 cm] thick and thicker if much heavy-truck parking is expected. An abrupt change in shoulder thickness at the lane/shoulder interface may result in differential frost heave and a "bathtub" design. If a "bathtub" design results, then the installation of a positive subsurface drainage system is mandatory.

7. Transverse joints for PCC shoulders should be weakened plain type with no mechanical load transfer. Each joint in the traffic lane must be matched with a similar joint in the PCC shoulder. In addition, if the traffic lane joint spacing is greater than 20 ft [6.1 m], additional joints should be placed in the shoulder to keep the maximum joint spacing to less than 20 ft [6.1 m] (thickness and spacing selected from section 4.5.3).

8. The lane/shoulder tie system is crucial to the success of the increased edge support. Good load transfer can be achieved by placing deformed rebars across the joint. After drilling holes into the existing slab, the bars must be installed in the holes with epoxy or a nonshrinkage cement grout. The embedment length of the bars in the existing slab and the new shoulder should be adequate to develop full bar yield strength. In areas where deicing salts are used, the bars should be coated with a corrosion resistant coating. The following tie design appeared to perform satisfactorily:

Tie bar spacing:	24 in [61.0 cm] (12 in [30.5 cm] near joints)
Tie bar diameter:	0.625 in [1.6 cm]
Tie bar length:	30 in [76.2 cm]

9. The edge beam design is similar to the PCC shoulder design except that it is much narrower than the shoulder and can be thicker than the traffic lane slab. The two critical design parameters are the edge beam width and its thickness. Indications are that the edge beam width should be at least 24 in [61.0 cm] to contribute significant structural benefit, as well as to provide sufficient lateral clearance for the hole drilling operation to achieve adequate horizontal placement of the tie bars.

   Based upon the edge beams surveyed in this study, transverse cracking may develop in the beam if it is not of adequate width and thickness. A minimum width of 3 ft [0.9 m] is recommended to minimize transverse cracking. The depth of the beam should be at least the thickness of the slab. The edge beam should be jointed to match the existing pavement plus have additional joints as discussed for PCC shoulders. Weakened-plane contraction joints should be formed as soon as possible after placement.

10. The need for a more substantial design of the edge beam to significantly improve the load deflection response of the existing pavement is evident. Comparisons of transverse joint efficiencies in Minnesota (NCHRP 1-21) at slab corners and respective edge beams suggested the need for longitudinal tie bars at locations very near to the existing transverse joint. This would allow the entire system to work simultaneously to dampen the effects of traffic loadings at this critical location.

11. It is important that uniform support be provided under the entire shoulder. If the base material is disturbed during excavation, it should be adequately recompacted. Settlement of the outer shoulder edge can produce very high "pullout" stresses in the joint tie system. The magnitude of these stresses may be sufficient to exceed the strength of the tie bars and drastically decrease the edge support effectiveness.

CHAPTER 5

FULL-DEPTH REPAIR

5.1 INTRODUCTION

The purpose of full-depth repair of jointed portland cement concrete pavement is to reconstruct deteriorated areas and restore the overall structural integrity of the pavement. To be most effective, a full-depth repair should remain serviceable for as long as the surrounding slabs.

The performance of full-depth repairs has been inconsistent. While there are many documented cases of repairs that have performed satisfactorily, the performance record of many other inservice full-depth repairs has been poor.(33,34) Failures, such as repair settlement, rocking, faulting, premature cracking, spalling, pumping and frost heave have often been observed within a year after construction.

The construction of full-depth repairs of portland cement concrete (PCC) pavements has become a major part of pavement rehabilitation programs of transportation agencies throughout the United States. As such, it consumes a large portion of the total budget set aside for pavement rehabilitation. The high construction cost and inconsistent field performance of full-depth repairs indicates that there is a critical need to identify and develop more cost-effective and reliable full-depth repair designs and construction procedures.

Although nearly all components of the full-depth PCC pavement repair process could benefit from the results of additional research, one area that has great potential to advance the state of the art and performance of full-depth repairs is the successful establishment of load transfer across all transverse joints associated with full-depth repairs. The design of effective load transfer systems for full-depth concrete pavement repairs has consistently posed a major problem for most transportation agencies.

Agencies have utilized a "trial and error" design approach that has resulted in the use of many different PCC repair designs. Some of these repairs have performed well, while others have performed very poorly. Often a design that performed well in one installation has failed in another. Much of the variability in the performance of a given design is probably attributable to variable construction quality control. Dowels and tie bars have generally offered the greatest potential for consistently providing full-depth repair joints with good load transfer characteristics without detrimental side effects (i.e., differential frost heave, etc.).

Dowels and other mechanical load transfer devices installed in <u>new</u> jointed concrete pavements often lose much of their effectiveness after a period of service allowing the joints to fault. This loss may be due to initial poor consolidation of concrete, the effects of dowel/concrete bearing fatigue or failure from repeated heavy loadings and mechanical failure due to corrosion.

When these same devices are installed in full-depth repair joints, loss of load transfer is often accelerated due to built-in defects, such as weak, damaged or missing grout or epoxy material in the immediate vicinity of the device, inadequate structural design of the device, and improper installation or construction.(35) Figure 42 illustrates the elongation of transverse repair joint dowel holes due to poor dowel bar grouting and erosion of the supporting concrete that was observed after only 1 year of service on a heavily trafficked pavement.

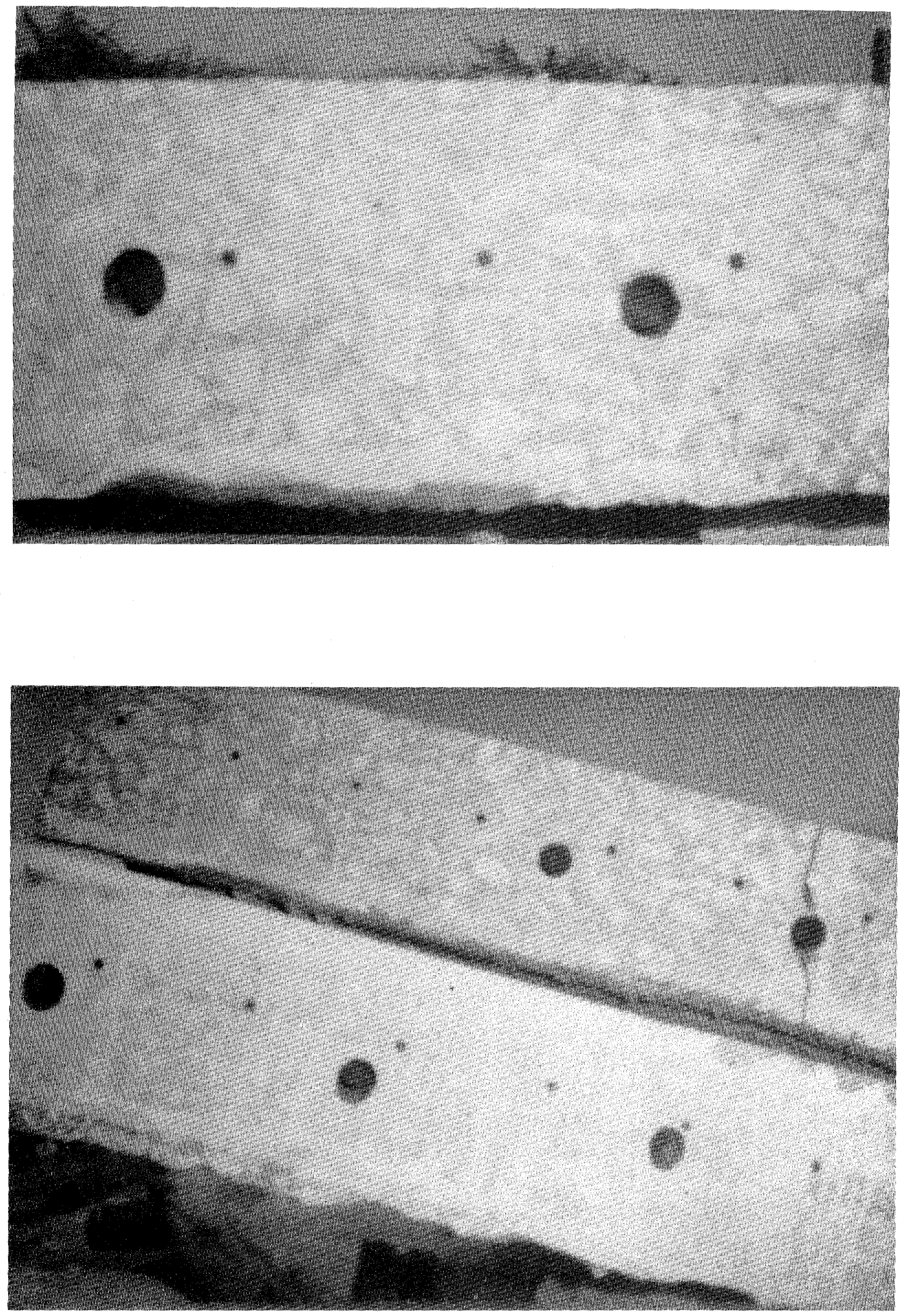


Figure 42. Elongation of transverse repair joint dowel holes after one year of heavy traffic.

There are <u>many</u> factors that affect the performance of dowel load transfer systems in repairs, including:

- Dowel design (diameter, length, coating, elasticity, etc.).
- PCC slab and anchor material properties.
- Hole size relative to dowel diameter (annular gap).
- Installation conditions (e.g., moisture, temperature, alignment, cleanliness, and adequacy of construction techniques).

Of these factors, poor installation conditions and construction quality are most often considered responsible for repair failures.(6) On many rehabilitation projects, dowels are loose enough to be moved by hand after installation.(36)

A more thorough knowledge of the effects and interactions of dowel diameter, length, and placement, anchor materials, construction procedures and other variables will lead to more reliable cost-effective repair design and construction techniques which are expected to result in substantial extensions to serviceable pavement life.

5.2 DATABASE AND DATA COLLECTION

5.2.1 Project Field Database

The inservice repair design and performance data used to develop full-depth repair performance models and distress correlations were collected for 2001 individual repairs on more than 125 rehabilitation projects located in 22 States. This database represents a variety of different repair designs and transverse joint load transfer designs as well as the effects of several different types of climates and rates of traffic accumulation. These repairs were surveyed between June 1985 and June 1987.

The development of repair performance models and improved design and construction guidelines and procedures required the collection of several types of data, including:

- Field distress.
- Original pavement structural design, in-situ condition and historical improvement information.
- Rehabilitation design and timing data.
- Detailed traffic data (including traffic classifications, volumes and accumulated 18-kip [80-kN] single-axle loads both prior to and since repair construction.
- Environmental data.

The sources and procedures used in the collection of each of these types of data are described in volume IV. A complete list of the pavement condition variables considered in the field surveys is presented in table 16. A listing of the original pavement design variables included in the database is presented in table 17. A listing of the full-depth repair design variables that were included is presented in table 18.

Table 16. Pavement condition variables collected during field survey.

General:

- Sample Unit.
- Condition of Drainage Ditches.
- Number of Transverse Joints in the Sample Unit.
- Sample Unit Foundation.
- Subsurface Drainage Present and Functional.

Slab Distress Variables:

- Transverse Cracking.
- Longitudinal Cracking.
- Longitudinal Joint Spalling.
- Transverse "D" Cracking.
- Longitudinal "D" Cracking.
- Scaling, Crazing, Map Cracking, and Shrinkage Cracking.

Joint Distress Variables:

- Transverse Joint Spalling.
- Pumping.
- Transverse Joint Width.
- Reactive Aggregate Distress.
- Incompressibles in Joints.
- Corner Spalling.
- Transverse Joint Faulting.
- Corner Breaks.
- Joint Sealant Damage.

Additional Distress Variables Related To PCC Repairs:

- Transverse Repair Cracking.
- Longitudinal Repair Cracking.
- Location of Spalls and Corner Breaks (e.g., Approach or Leave Joint, On Repair or Adjacent Slab)

Table 17. Original pavement design and construction variables.

General:

- Identification Number (Highway Number, Milepost, Traffic direction.
- Beginning and Ending Mile Post and/or Station.
- Number of Through Lanes.
- Type of Original Concrete Pavement (JRCP, JPCP).
- Layer Descriptions, Thicknesses and Material Types.
- Date of Originial Pavement Construction.
- Dates and Description of Major Pavement Improvements.

Joints and Reinforcing:

- Average Contraction Joint Spacing.
- Skewness of Joints.
- Expansion Joint Spacing.
- Transverse Contraction Joint Load Transfer Spacing.
- Dowel Diameter.
- Type of Slab Reinforcing.
- Longitudinal Bar/Wire Diameter and Spacing.

Subgrade, Shoulder and Drainage:

- Type of Subgrade Soil (Fine-Grained, Coarse-Grained).
- Outer Shoulder Surface Type.
- Original Subsurface Drainage Type.
- Original Subsurface Drainage Location.

Table 18. Full-depth repair design and construction variables.

General:

- Project ID Number (Highway Number, Milepost, Traffic Direction.
- Project Sample Unit ID Number.
- Repair Location (Station) Within Sample Unit.

Design Considerations:

- Repair Joint Types (Expansion, Contraction, Tied) for Each Repair Joint.
- Joint Load Transfer Types (Dowels, Undercut, Ties, Other, None) for Each Repair Joint.
- Skew of Transverse Joints.
- Transverse Repair Joint Bar Diameter, Length, Spacing and Locations.
- Dowel or Tie Bar Anchor Material.
- Reinforcing Steel in Repair.
- Repair Joint Sealing Details.

Construction Considerations:

- Equipment Used to Cut Repair Boundaries .
- Depth of Boundary Saw Cut.
- Method of Slab Removal.
- Foundation Repair Details.
- Repair Curing Details.
- Curing Time Prior to Reopening to Traffic.

5.2.2 Range of the Database

The database contains as many projects as could be reasonably included given the available resources and a wide range of data values were included to develop broad-based conclusions and useful performance prediction models. Still, not all combinations of important variables were found in the field. Thus, conclusions and models drawn from the database must be used with the knowledge that extensions beyond the scope of this database may be inaccurate.

Figure 43 shows the distribution of repairs across the United States. Figure 44 presents the age distribution of the surveyed repairs and table 19 presents the distribution of cumulative 18-kip [80-kN] equivalent single-axle loads and load transfer system designs for these repairs.

5.2.3 Illinois DOT Experimental Full-Depth Repair Project (I-70 near St. Elmo)

An experimental project was constructed by the Illinois Department of Transportation (IDOT) on I-70 in 1984 to determine the effects of various load transfer design parameters on repair performance. Variable features included dowel bar diameter (1.25-in [3.2 cm] vs. 1.50-in [3.8 cm]), number of dowels per wheel path (3, 4, or 5), dowel bar anchor material (nonshrink cement grout vs. epoxy mortar), and the use of tie bars rather than dowels in the repair approach joint. A summary of the individual repair design features is presented in table 20. The repairs are all constructed in the outer lane of the highway using stringent quality control and inspection procedures. The repair joints were sawed and sealed after construction.

IDOT has monitored the performance of these repairs since construction by periodically measuring deflection load transfer using a Dynatest Model 8000 Falling Weight Deflectometer. Measurements were taken in the outer wheel path at both repair joints with the load placed on both the original slab and the repair itself for a total of four measurements per repair. These measurements were taken six times during the first year of repair service and twice annually thereafter. Repair joint faulting has been measured annually by IDOT personnel (beginning in December 1984) and additional faulting measurements were obtained by the University of Illinois project team in July 1985 and June 1987. Preliminary conclusions drawn from this project were presented by Lippert in early 1987.(43)

All other pertinent design, construction, climatic and performance data for this project are included in the research project database described previously. This data subset provides an excellent basis for determining the field performance of various repair joint designs as well as identifying the relationship between joint load transfer and repair faulting.

5.3 FIELD PERFORMANCE AND EVALUATION

While several full-depth repair distresses have been identified in previous studies and were noted during the condition surveys conducted for this study, very few impact serviceability enough to cause a failed condition by themselves. These critical distresses include transverse joint faulting and spalling (due to any source), and high-severity slab cracking. Many repairs are still serviceable and provide an acceptable ride in the presence of lower severity slab cracking and other distresses.

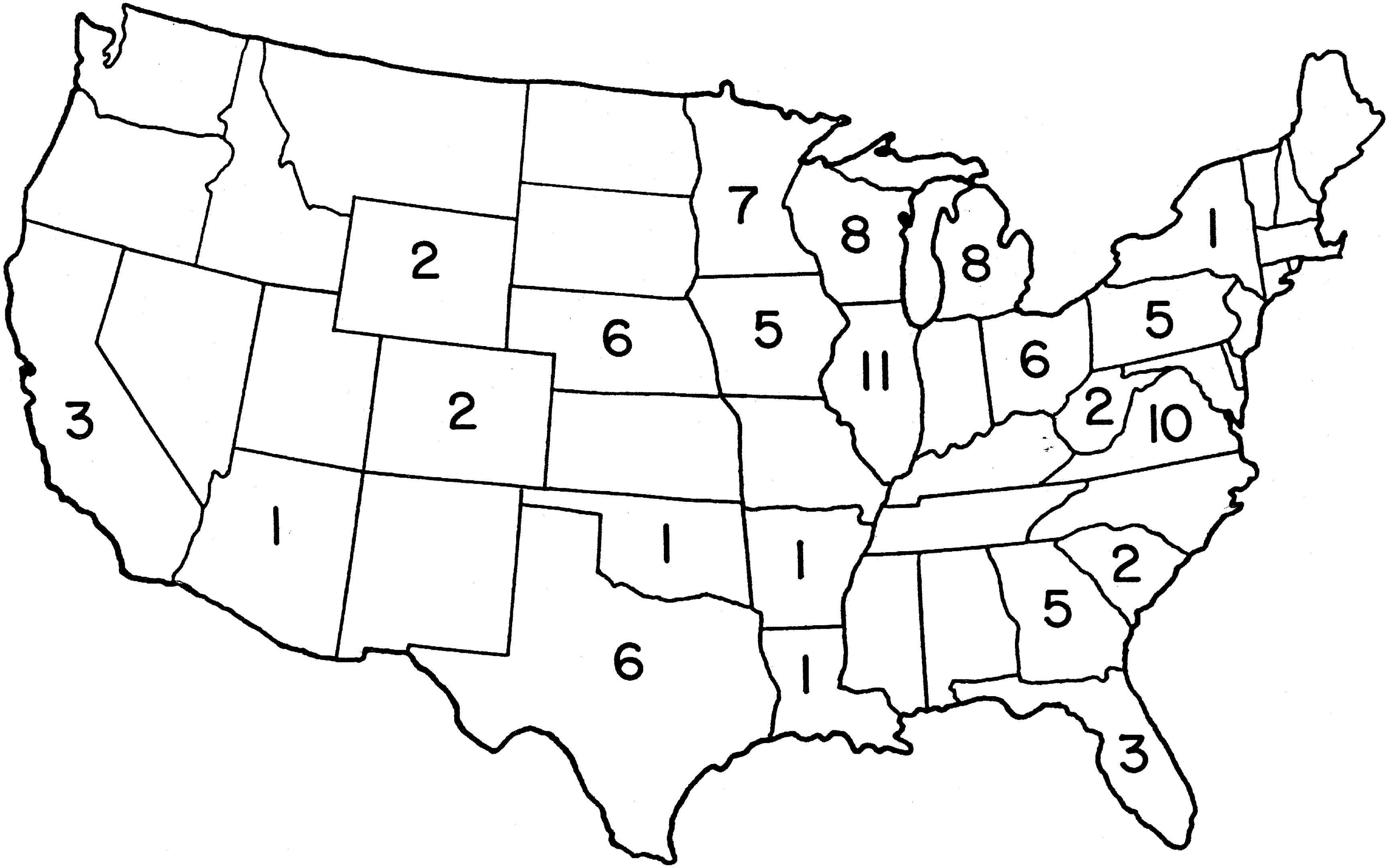


Figure 43. Full-depth repair projects surveyed.

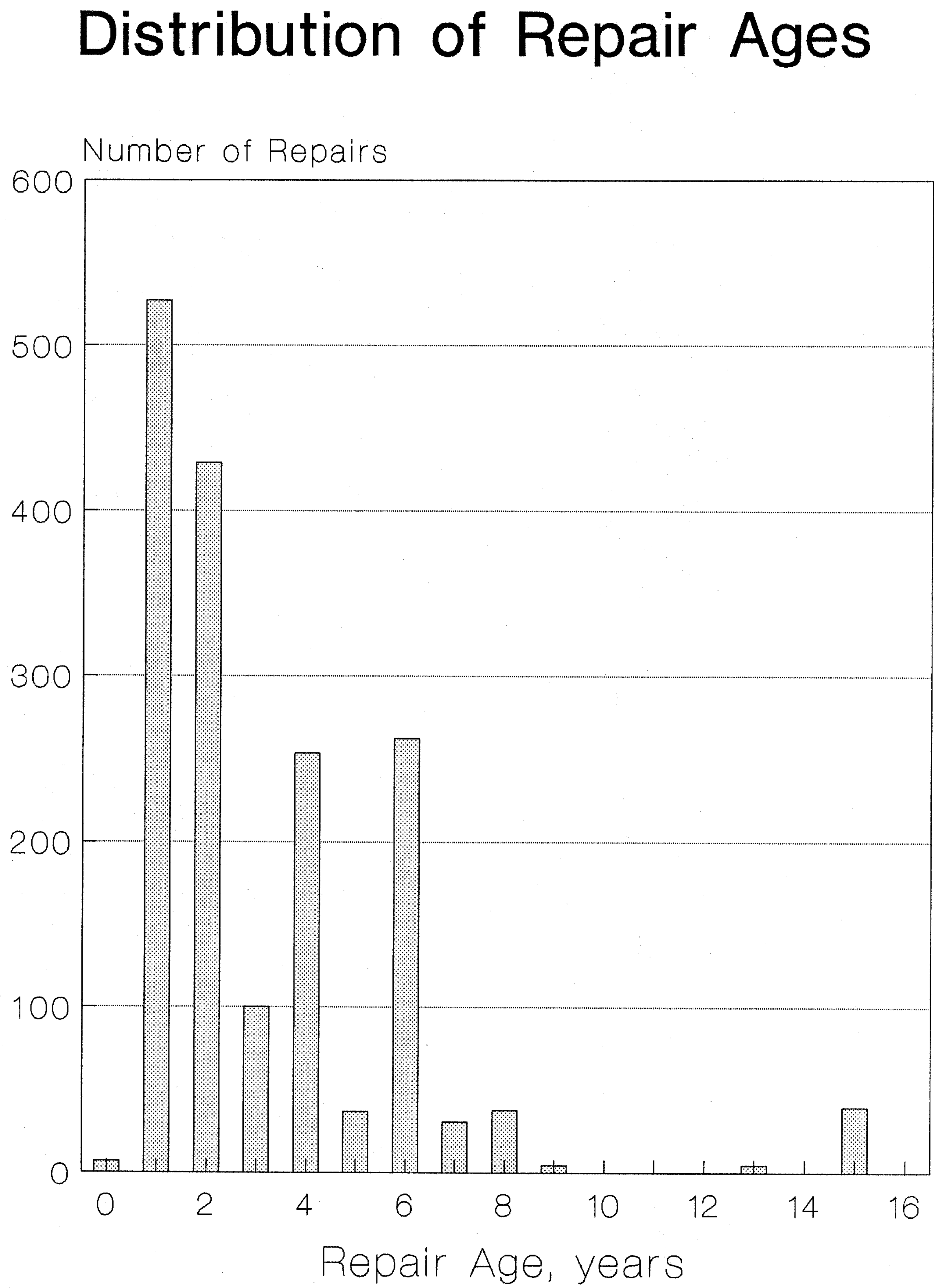


Figure 44. Distribution of full-depth repair ages.

Table 19 Distribution of cumulative 18-kip [80-kN] ESALs (millions) and load transfer system designs for the surveyed repairs.

Joint Load Transfer Type

Count / Row / Col Pct / Tot Pct	Other or Missing	None	Aggreg. Inter.	Undercut	Tied or Dowelled	
0 < ESAL < 1	184 24.6 79.7 10.6	10 1.8 8.9 0.6	117 15.6 41.1 6.7	84 11.2 47.2 4.8	353 47.2 37.8 20.3	748 43
1 < ESAL < 3	0 0 0 0	51 8.5 45.5 2.9	48 8 16.8 2.8	94 15.6 52.8 5.4	410 68 43.9 23.6	603 34.7
3 < ESAL < 5	47 15.9 20.3 2.7	34 11.5 30.4 2	105 35.6 36.8 6	0 0 0 0	109 36.9 11.7 6.3	295 17
5 < ESAL < 10	0 0 0 0	17 18.3 15.2 1	15 16.1 5.3 0.9	0 0 0 0	61 65.6 6.5 3.5	93 5.3
	231 13.3	112 6.4	285 16.4	178 10.2	933 53.7	1739 100

Table 20. Illinois I-70 experimental patch design data.

PATCH ID	DIRECTION	STATION	MILEPOST	DESIGN	DOWEL SIZE	GROUT
1	WB	1438	81.40	3-3	1.25	NON-SHRINK
2	WB	1421	81.23	4-4	1.25	NON-SHRINK
3	WB	1309	79.15	5-5	1.25	NON-SHRINK
4	WB	1265	78.24	3-3	1.50	NON-SHRINK
5	WB	1246	78.05	4-4	1.50	NON-SHRINK
6A	WB	1205		5-5 (Tied)	1.25	NON-SHRINK
6B	WB	1204		5-5 (Tied)	1.25	NON-SHRINK
6C	WB	1203		5-5 (Tied)	1.25	NON-SHRINK
6	WB	1162	76.01	5-5	1.50	NON-SHRINK
7	EB	1209	77.22	3-3	1.25	NON-SHRINK
8	EB	1210		3-3	1.25	NON-SHRINK
9	EB	1211		3-3	1.25	NON-SHRINK
10	EB	1212		4-4	1.25	NON-SHRINK
11	EB	1214	77.27	4-4	1.25	EPOXY
12	EB	1264	78.25	4-4	1.25	NON-SHRINK
12A	EB	1271		5-5	1.25	NON-SHRINK
12B	EB	1275		5-5	1.25	NON-SHRINK
13	EB	1276		5-5	1.25	NON-SHRINK
13A	EB	1277		5-5	1.25	EPOXY
14	EB	2178+60	78.40	3-3	1.50	NON-SHRINK
15	EB	1279		3-3	1.50	NON-SHRINK
16	EB	1315+10	79.24	3-3	1.50	NON-SHRINK
17	EB	1318		4-4	1.50	NON-SHRINK
18	EB	1319	79.26	4-4	1.50	NON-SHRINK
19	EB	1320		4-4	1.50	NON-SHRINK
20	EB	1322	79.29	5-5	1.50	NON-SHRINK
21	EB	1329	79.36	5-5	1.50	NON-SHRINK
22	EB	1368	80.22	5-5	1.50	NON-SHRINK

General field observations related to the development of these distresses in full-depth repairs are presented.

5.3.1 Transverse Joint Faulting

<u>Special Considerations in Analysis</u>

The analysis of transverse joint faulting data for full-depth repairs presents some problems not encountered in the analysis of new pavement sawed or formed contraction joints. The construction of sawed or formed contraction joints generally results in a relatively smooth joint and all joints formed or sawed at the same time are typically of the same quality. Additionally, the quality of construction of underlying load transfer devices is relatively uniform between joints. This situation simplifies the analysis because the rate of development of faulting (and the variability of that rate between joints) is purely a function of the joint design, loading and climatic conditions, and the inherent variability of the materials.

"Faulting," or a difference in elevation across the joints, is often "built-in" to full-depth repair joints as the repair is over- or underfilled with concrete so that an initial offset exists. The amount of this built-in faulting may vary between joints within a given repair and certainly varies between repairs on a given project or between projects. The quality of construction also commonly varies both within and between projects such that similar designs often perform very differently under the same traffic and environmental conditions. The rate of development of full-depth repair faulting (and the variability of that rate between joints, repairs, and projects) is dependent on many factors that are very difficult to assess, as well as on the more easily quantified factors identified for regular contraction joint faulting.

The analysis of faulting data collected under this study included several techniques that were intended to reduce the effects of the problems described above. These techniques included:

- The use of correlation and partial correlation matrixes (using the entire data base) to identify relationships between faulting and independent variables.
- The separate analysis of data from repairs where a time sequence of faulting data (including the built-in faulting at time zero) is known.
- The development of faulting prediction models using only repairs that were known to have zero faulting at the time of construction (i.e., only those repairs that were diamond-ground immediately after construction).

The results of these analyses are summarized below.

<u>Preliminary Analyses</u>

Preliminary analyses of the faulting data indicated that repair leave joint faulting was often much greater than approach joint faulting and that detailed analyses should consider these two independently. This finding was in agreement with at least one previous study.(34)

Possible explanations for this finding were sought in the accepted mechanisms for faulting at pavement contraction joints, which are illustrated in figure 45. Where excess moisture, heavy traffic and erodible pavement layers are present, the

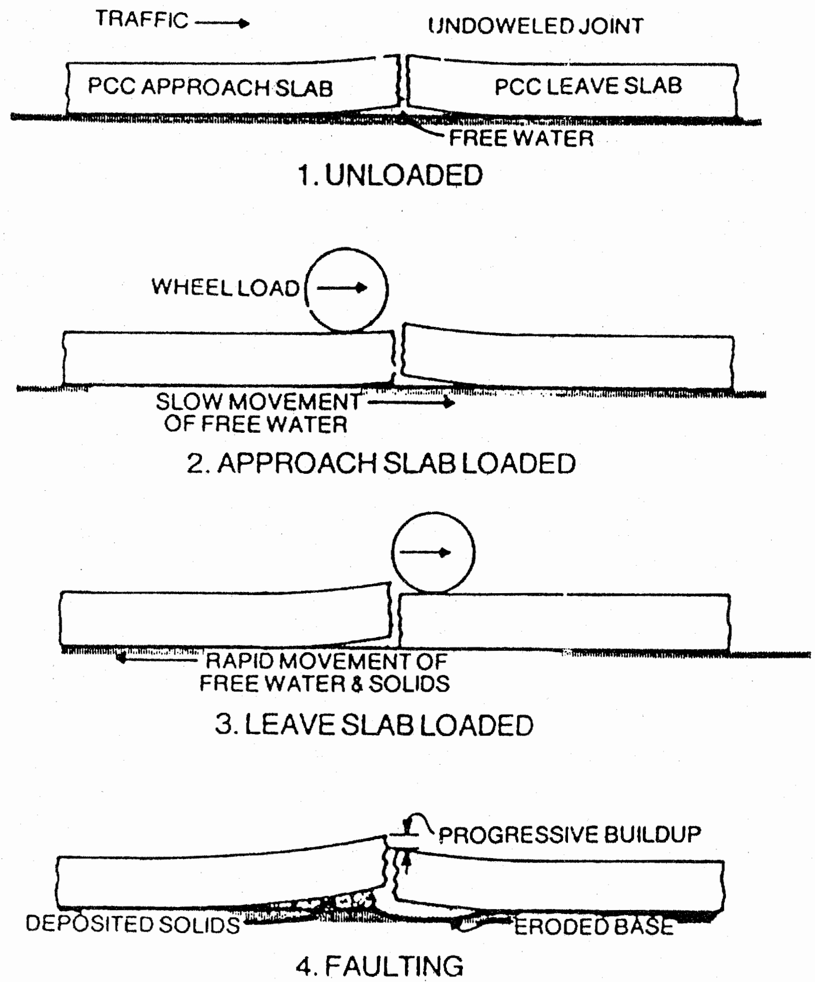


Figure 45. Illustration of the development of faulting at transverse joints.(7)

aoisture under the slab can be moved (relatively slowly) across the joint by approaching traffic and ejected (rapidly) as the traffic crosses the joint, carrying eroded material back to the approach side of the joint, where it is deposited. A buildup of this material will lift the approach slab while a void develops beneath the leave slab. This mechanism also depends on the independent vertical movement of the two slabs at the joint. Faulting is reduced by nearly one-half where good load transfer capacity exists.

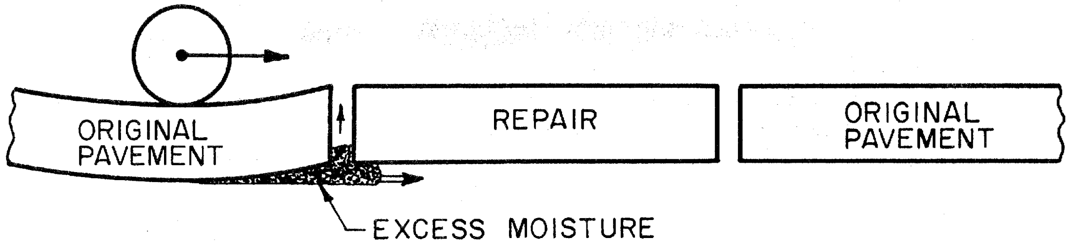
The behavior of full-depth repairs under load is more difficult to assess. Long repairs (greater than 2 to 3 times the radius of relative stiffness) are likely to behave like a long slab on grade, as described above. More typically-sized repairs (3 - 6 ft [0.9 - 1.8 m] long) may "rock" or "punch down" under passing loads, depending upon the length of the repair, degree of load transfer present at the repair joints, and the stiffness of the base/subgrade.

These different types of behavior would result in different moisture movements than were described above. A rocking motion of the repair would allow the moisture (or a substantial portion of it) to continue to be pushed forward as the traffic crosses the repair approach joint. As the traffic crosses the repair leave joint, the water would be ejected from beneath the original slab and deposited under the repair leave joint, resulting in a larger repair leave joint fault. This process is illustrated in figure 46.

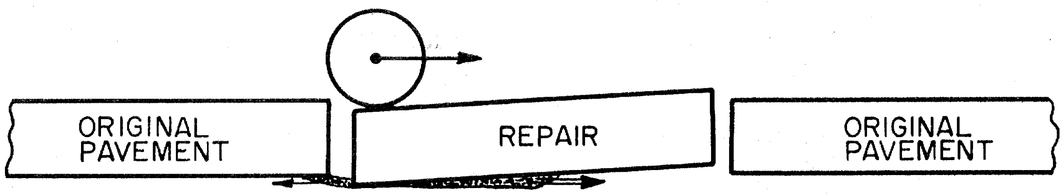
In many of the cases where larger faults were observed at the leave joints, the leave joint widths were also greater than the approach joint widths. It has been theorized that this occurs when the repair slips backwards (opposite the flow of traffic) under the torque of passing wheel loads and that this action is facilitated by the momentary "floating" of the repair on saturated support materials as the passing traffic impacts the repair (see figure 47). Moving the repair toward the approach joint would result in improved load transfer at that joint because of increased friction or aggregate interlock and smaller dowel deflections. Conversely, moving the repair away from the leave joint causes a loss of load transfer at that joint. Additionally, since bearing stresses increase with increasing joint width, the leave joint would experience increased bearing stresses, which have been shown to have a strong relationship on contraction joint faulting and repair joint faulting.(1,52,53,73) Some agencies have constructed repairs using tie bars (rather than dowels) along the approach joint to prevent longitudinal repair movement, although the long-term effectiveness of this approach is still unverified.

Finally, the overfilling of many repairs results in a "built-in" negative approach joint fault (leave slab higher than approach) and a positive leave joint fault (leave slab lower than approach). If accepted pumping and faulting mechanisms are active at both repair joints, the approach slab will be raised relative to the leave slab at each joint, causing the magnitude of approach joint faulting to decrease until it changes from negative to positive. The magnitude of faulting at the repair leave joint will increase from its initial value. This process is illustrated in figure 48.

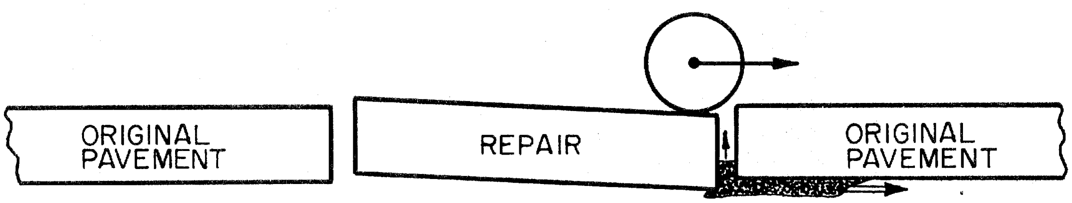
Since repair leave joint faulting was typically found to be more severe than the approach joint faulting, it is the more critical of the two in terms of repair performance and serviceability. The remaining discussion and analyses deal only with leave joint faulting in the outer lane, although similar analyses and conclusions can be drawn for approach joint and inner lane repair faulting.



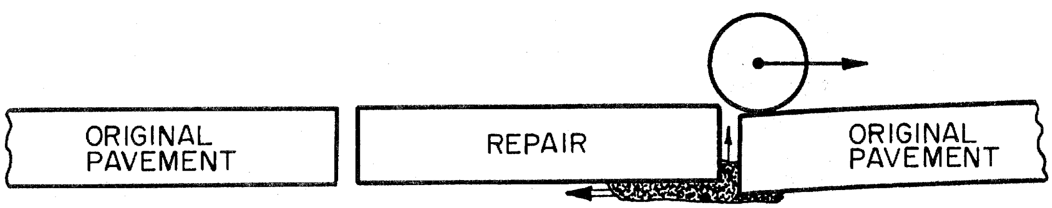
a) Moisture is Squeezed Forward by Approaching Wheel Load



b) Wheel Crosses Repair Joint and Repair Tilts Slightly, Allowing Some Moisture to Eject Backwards (Causing Some Approach Joint Faulting) But Forcing Most Moisture Forward



c) Wheel Approaches Repair Leave Joint, Pushing Excess Moisture Ahead



d) Wheel Crosses Leave Joint and Most Moisture is Ejected Backward, Carrying Erodible Foundation Materials and Causing Leave Joint Faulting

Figure 46. Illustration of possible mechanism for repair faulting where leave joint faulting exceeds approach joint faulting.

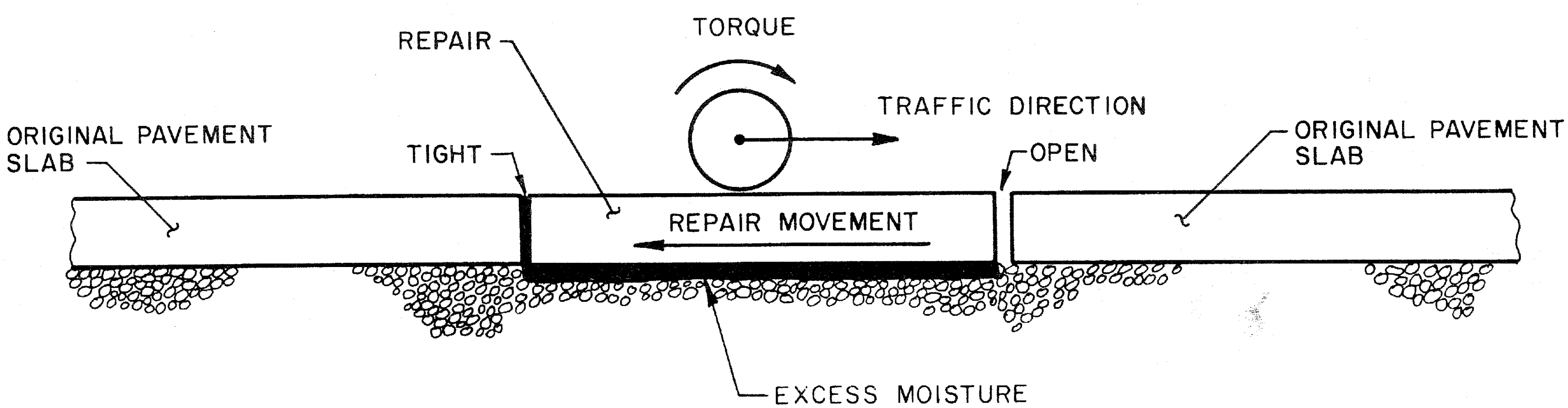
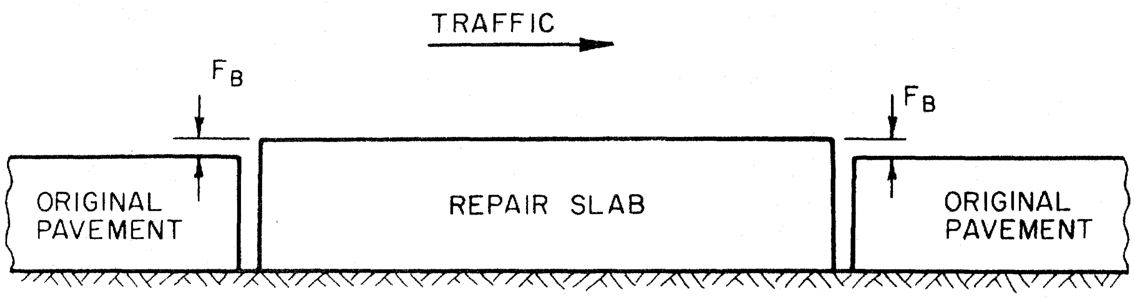
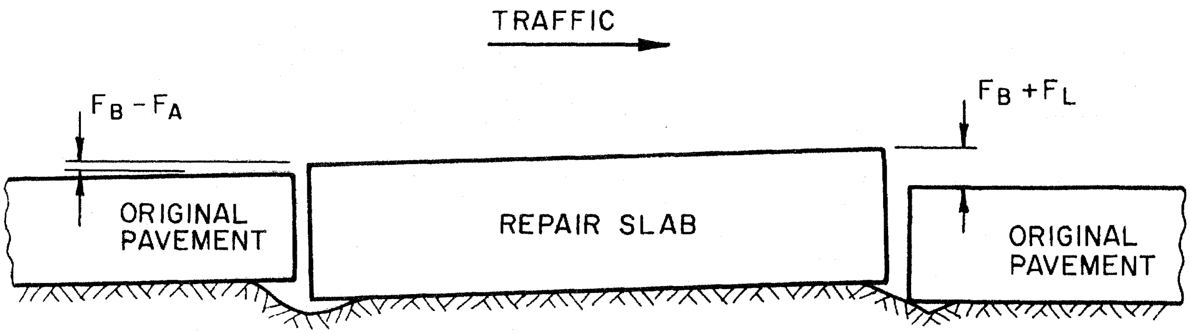


Figure 47. Full-depth repair loading and faulting mechanism.



a) Overfilled Repair Immediately After Construction (Built-In Approach Joint Fault = $-F_B$, Built-In Leave Joint Fault = F_B)



b) Overfilled Repair After Development of Faulting

F_B = "Built-In" Faulting Due to Overfilling of Repairs

F_L = Developed Faulting at Leave Joint

F_A = Developed Faulting at Approach Joint

Figure 48. Illustration of possible development of faulting at overfilled repairs.

Factors Affecting Repair Leave Joint Faulting

Table 21 presents the correlation coefficients and their significances for some key variables that have often been considered related to the development of full-depth repair transverse joint faulting. These relationships may be stronger than indicated because the tremendous variability of built-in faulting between repairs (mean faulting = 0.13 in [0.33 cm], std. dev. = 0.0645 in [0.164 cm], C.O.V. = 49.6 percent for 28 repairs recently constructed in Illinois) results in the computation of weaker correlation coefficients.

The weak relationships indicated for traffic and bearing stress are difficult to interpret since these have correlated much better for new pavements. It may be an indication that repair dowels are typically so poorly anchored in the existing slab that the repairs move freely enough to pump moisture and fault very rapidly under relatively low traffic volumes. Poor anchoring of the dowels also prevents them from developing full bearing capacity until extremely high deflections have taken place. Thus, poorly anchored dowels could result in the rapid development of repair faulting with little consideration of load transfer system design.

Variables related to pavement and repair support appear to strongly affect the development of full-depth repair faulting. The use of strong support layers (increasing the effective k-value through thick or stabilized base layers), pavement drainage systems, and tied shoulders significantly reduce the development of repair faulting (listed in order of decreasing effect).

A strong positive relationship is indicated between pavement contraction joint spacing and increased repair leave joint faulting. Long slabs curl and warp more than short slabs, providing potential for more vertical slab movement near the joints, which can produce pumping and faulting. In addition, longer slabs are subject to greater temperature-related horizontal movements which produce wider joints in cool weather. These wider joints increase the bearing stresses beneath properly installed dowels, which could increase faulting.

A weaker but significant inverse relationship between repair length and leave joint faulting is also indicated. This suggests that where load transfer is poor, longer repairs are more stable and resistant to the pumping/faulting mechanism. Shorter repairs may rock and pump more easily.

The practice of sealing repair joints immediately after construction reduces the entry of moisture into the pavement system, but, since moisture also enters through the lane/shoulder joint and from other sources, its impact on repair faulting is weak. Sealing joints does significantly reduce the development of spalling; this is discussed later.

Repairs located in colder climates were also observed to fault more than those constructed in warmer climates. This may indicate that faulting develops very rapidly during the spring thaw when very soft, moist support conditions exist for an extended period. A portion of this effect is compounded with the previously-described effects of longer joint spacings, which have commonly been constructed in colder climates.

Experimental Project Results

The repair joint faulting and load transfer data collected from the Illinois DOT experimental full-depth repair design project on I-70 is presented in table 22. The faulting data for months 9 and 32 were collected by project personnel; all other

Table 21 Correlation coefficients and their significances for key variables related to repair leave joint faulting (outer lane, dowelled repairs only - 699 cases).

Zero-Order Partials

Variable:	ESAL	Bearing Stress	K-Value	Repair Length
Coefficient*:	0.0189	0.0176	-0.1146	-0.0479
Significance:	0.308	0.321	0.001	0.103

Variable:	Tied Shldr (0=No,1=Yes)	Drainage (0=No,1=Yes)	Joint Seals (0=No,1=Yes)	Joint Spacing
Coefficient*:	-0.0124	-0.0700	-0.0187	0.0841
Significance:	0.372	0.032	0.311	0.013

Higher-Order Partials

Variable:	Tied Shldr (0=No,1=Yes)	Drainage (0=No,1=Yes)	K-Value
Coefficient*:	-0.0774	-0.1106	-0.1607
Significance:	0.021	0.002	0.001
Control For:	K-Value Drainage	K-Value	Tied Shldr Drainage

* - Pearson Product-Moment Correlation Coefficient

Table 22. Summary of Illinois DOT experimental repair project faulting and load transfer data.

Month After Construction	2	13	29	2	9	32
PATCH ID	LV LT (DEC 84) %	LV LT (NOV 85) %	LV LT (MAR 87) %	LV FAULT (DEC 84) in	LV FAULT (JUL 85) in	LV FAULT (JUN 87) in
1	96	90	98	0.07	0.09	-0.02
2	92	95	93	0.05	0.03	0.15
3	83	83	75	0.04	0.08	0.08
4	89	74	72	0.08	0.11	0.15
5	97	83	80	0.16	0.07	0.15
6A	94	86	74	0.15	0.06	0.11
6B	91	91	84	0.16	0.10	0.08
6C	100	85	83	0.22	0.09	0.09
6	93	83	81	0.14	0.27	0.14
7	99	75	55	0.19	0.22	0.20
8	86	67	54	0.17	0.23	0.15
9	96	91	82	0.18	0.11	0.11
10	90	89	71	0.16	0.27	0.20
11	88	41	22	0.02	0.14	0.19
12	94	76	60	0.14	0.14	0.19
12A	100	90	85	0.18	0.24	0.17
12B	91	93	93	---	0.09	0.12
13	89	82	69	0.14	0.12	0.09
13A	100	85	88	0.13	0.10	0.19
14	100	87	74	0.14	0.24	0.11
15	100	84	53	0.18	0.08	0.19
16	90	80	24	0.03	0.04	0.31
17	79	55	65	0.26	0.25	0.18
18	100	86	60	0.18	-0.11	0.20
19	100	81	74	0.16	0.08	0.09
20	88	92	79	0.14	0.24	0.03
21	100	85	81	0.15	0.06	0.11
22	100	92	84	0.02	-0.05	0.10

Note: 1 in = 2.54 cm.

data were collected by Illinois DOT personnel. The load transfer data were collected using a Dynatest Model 8000 Falling Weight Deflectometer. These data represent relatively short-term performance, but they come from one of the few well-designed experimental repair projects that have been constructed in the United States to date and provide a good indication of general performance trends. Figures 49 through 53 contain graphical presentations of some of the collected data and illustrate several key conclusions.

Figure 49 shows illustrates the relationship between number of dowels per wheel path and the deterioration of joint load transfer. The repairs constructed with five 1.25-in [3.2 cm] dowels per wheel path still exhibit leave joint load transfer measurements of more than 80 percent. Much greater losses were measured for repairs with three and four dowels per wheel path. Similar results were observed for the 1.5-in [3.8 cm] dowel installations and for approach joint load transfer measurements.

Figure 50 shows that the use of #8 deformed tie bars along the repair approach joint (five per wheel path) resulted in almost no loss of approach joint load transfer over time. When equal numbers of 1.25-in [3.2 cm] or 1.5-in [3.8 cm] dowels were used, approach joint load transfer losses of nearly 40 percent were observed in less than 3 years. Since the use of deformed bars in repair approach joints is also believed to help reduce the incidence and severity of repair joint spalling, their use apparently improves repair performance in many ways and should be considered wherever feasible. Since cement grout and large diameter drills were used in all three repair designs, the better performance of the tied approach joint may indicate that larger holes (relative to the size of the bar) facilitate good anchoring of the bars, especially where a very stiff grout is used. This would probably not be true for flowable grouts.

Figure 51 shows that the use of deformed bars in the approach joint had little effect on leave joint load transfer when five devices are used per wheel path. This figure also illustrates the general observation that the range of dowel diameters used in this study generally had little effect on the loss of leave joint load transfer or the development of leave joint faulting.

Figure 52 illustrates the observed effects of the use of varying anchor materials and numbers of dowels per wheel path (using 1.50-in [3.8 cm]- diameter dowels) on leave joint load transfer. Unfortunately, only two repairs were constructed using epoxy mortar (one for each joint design), so it cannot be determined whether the poor performance of the 4-4/epoxy repair is typical. If the good performance of the 5-5/epoxy repair is typical, it would bear out the theory (presented under the lab experiment results portion of this report) that it is easier to achieve uniform dowel support in full-depth repairs using epoxy mortars than cement grouts. The consistency of cement grouts can vary widely over short periods of time, from very fluid grouts that run out of the drilled holes to very stiff grouts that make dowel installation very difficult. Many epoxy mortars are preproportioned for uniformity and are mixed and delivered "on demand" using caulking gun-style systems. This uniform consistency is crucial to achieving good dowel installations.

Figure 53 presents the measured leave joint faulting that corresponds to the load transfer measurements presented in figure 51. The relationships between these faulting and load transfer measurements are representative of those observed throughout the experimental project -- reductions in leave joint load transfer are generally accompanied by an increase in faulting. In this figure, the highest

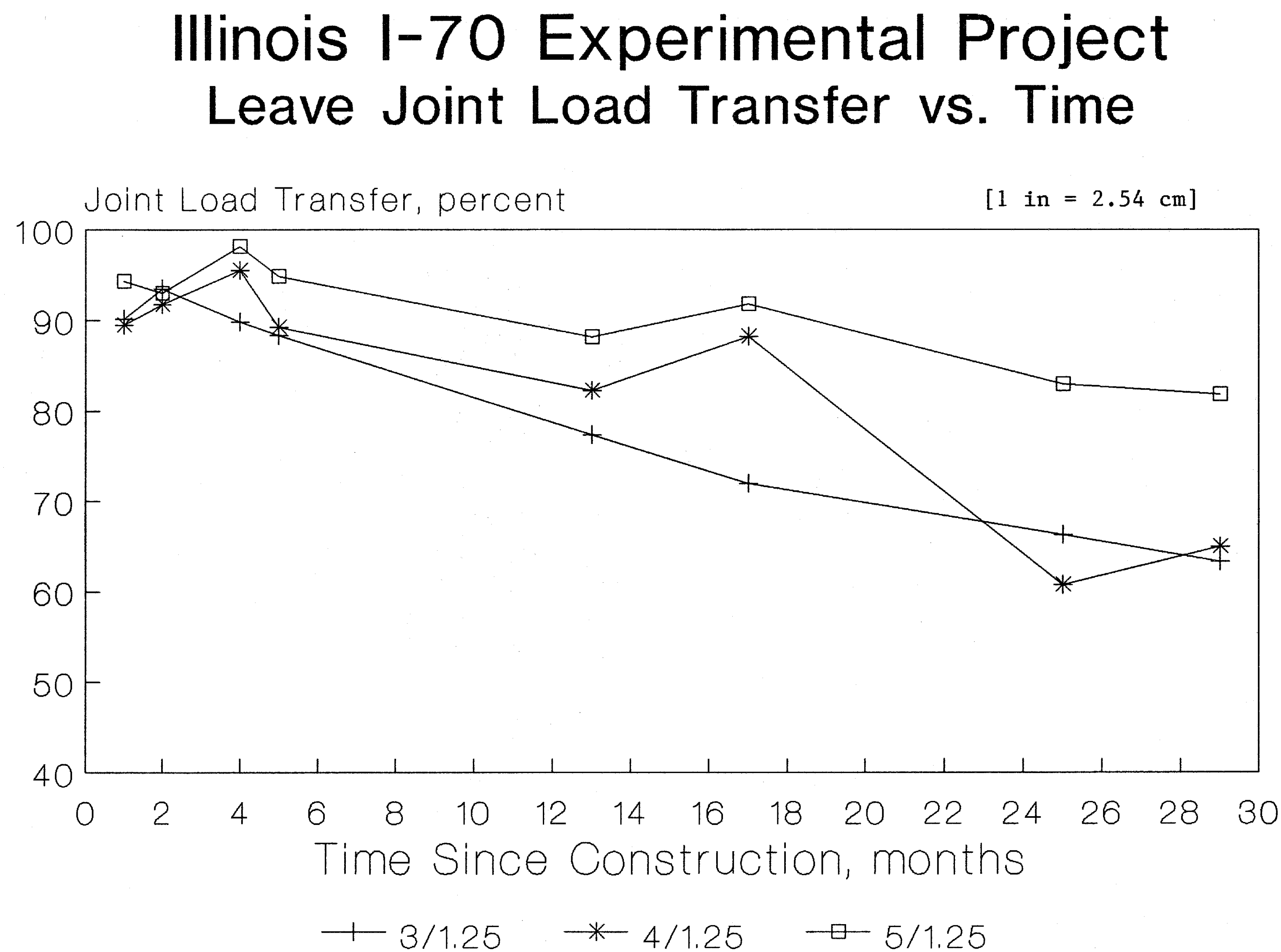


Figure 49. Plot of leave joint load transfer vs. time for varying numbers of 1.25-in [3.2-cm] dowels per wheelpath.

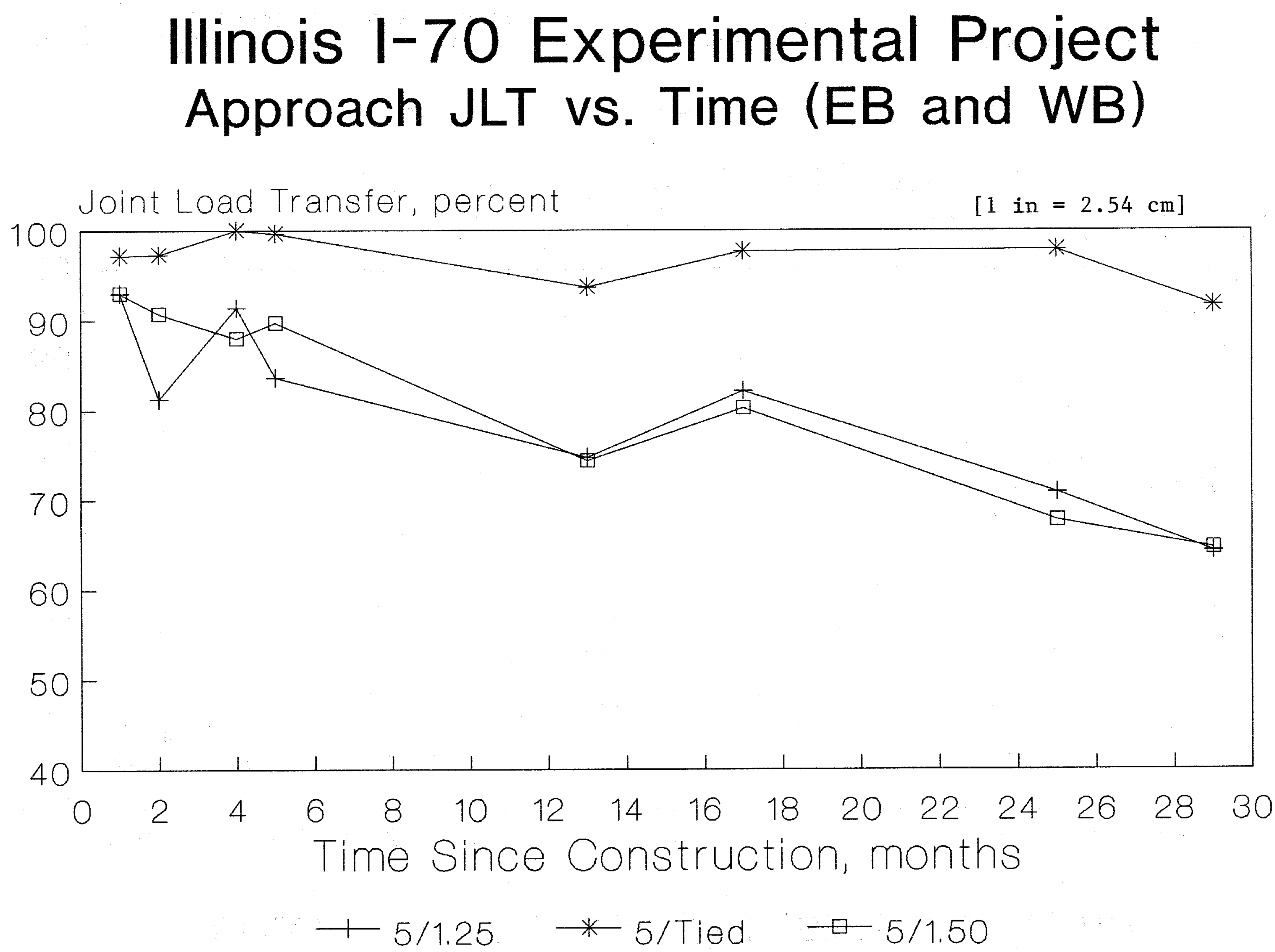


Figure 50. Plot of approach joint load transfer vs. time for tied and dowelled joints.

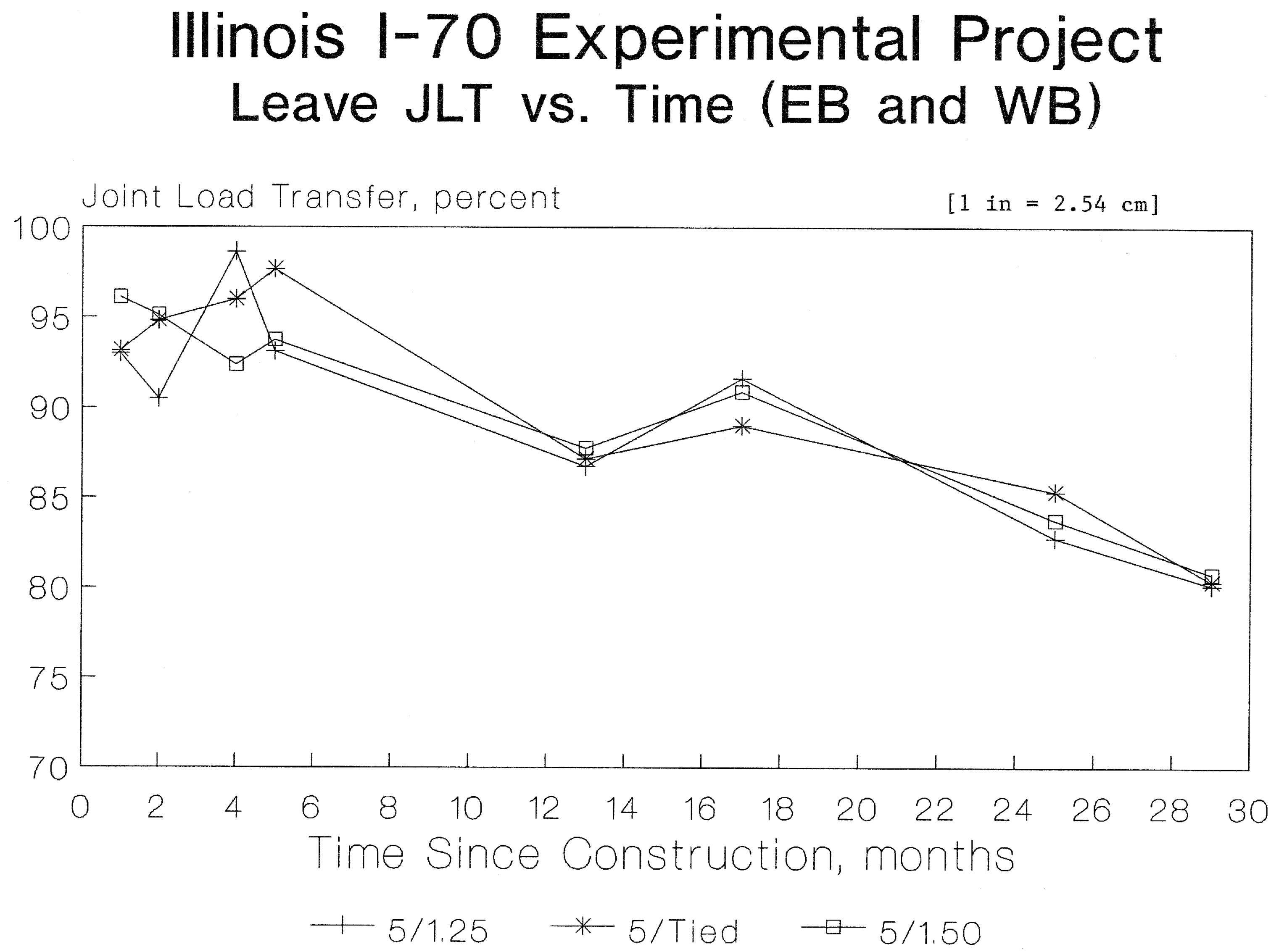


Figure 51. Effect of tied and dowelled approach joints on leave joint load transfer.

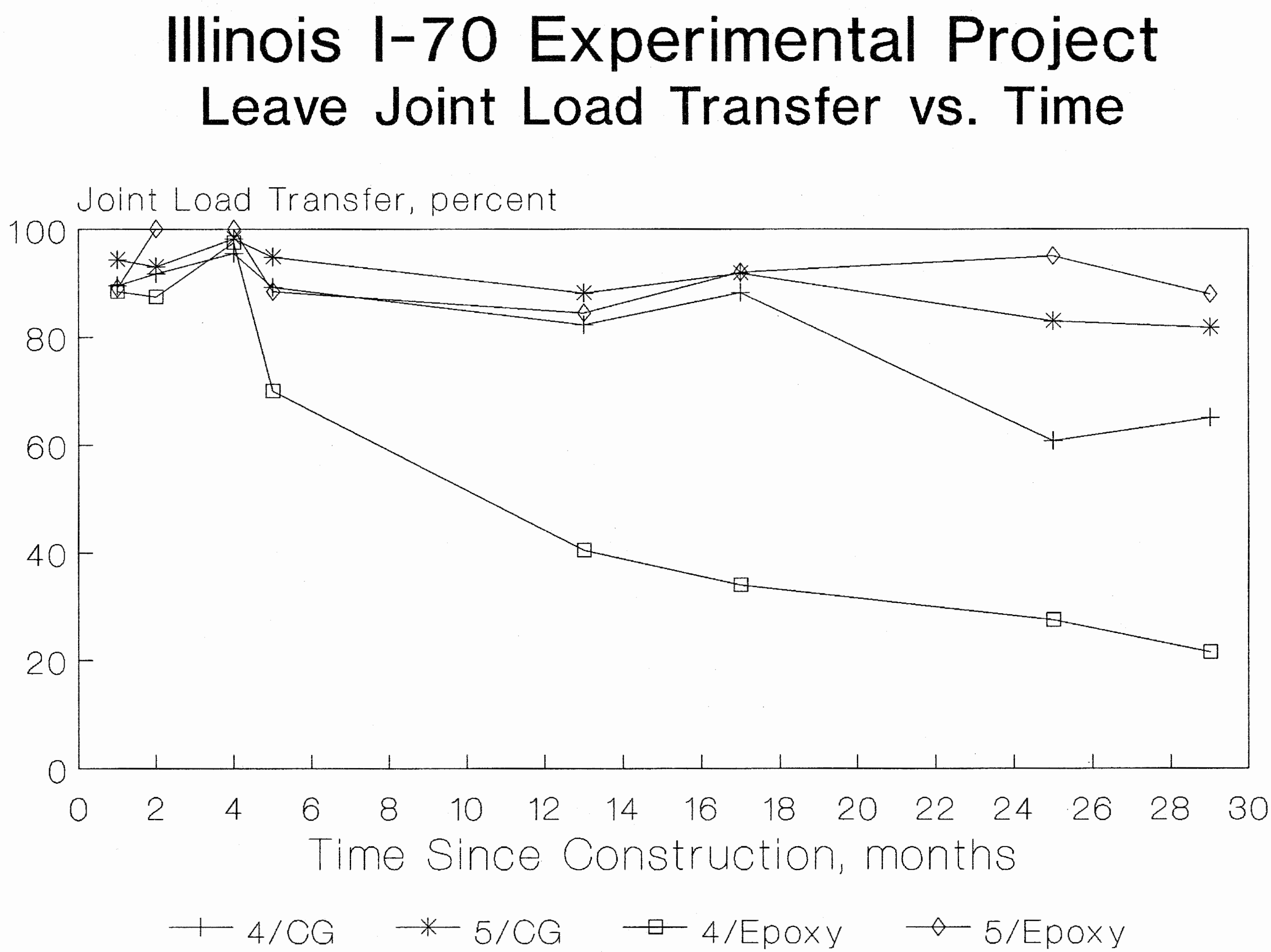


Figure 52. Effect of varying anchor materials and number of dowels per wheelpath on repair leave joint load transfer.

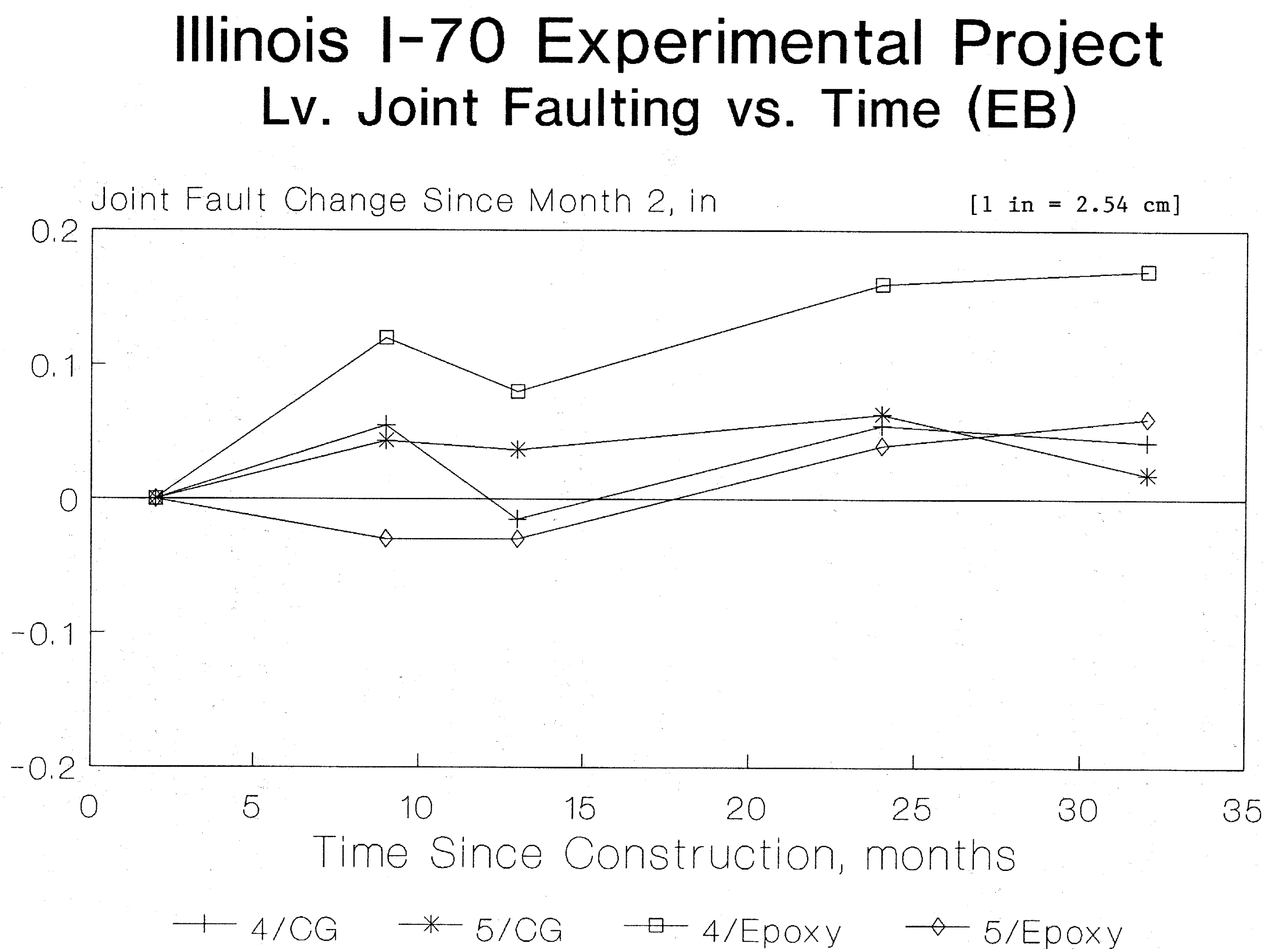


Figure 53. Effects of varying anchor materials and number of dowels per wheelpath on repair leave joint faulting.

faults were measured on the repair with four dowels per wheel path and epoxy mortar anchor material, which also exhibited the poorest load transfer. The repair with the best load transfer (five dowels per wheel path, epoxy mortar) developed very little faulting (less than 0.05 in [0.13 cm]) over 32 months. The other repair types covered in this figure also exhibited very little faulting, which indicates that large dowels (1.25-in [3.2 cm] diameter or greater) and good construction practices can produce good results with either cement grout or epoxy mortar.

Faulting Performance Model

The full-depth repair faulting data were sorted to obtain a data set containing only those repairs that had been diamond ground immediately after construction so that the measured faulting in 1985-1987 truly represented the change in faulting from the date of construction.

The following model for repair leave joint faulting (which was determined to be more critical than approach joint faulting, as discussed previously) was developed using nonlinear regression:

$$\begin{aligned} FAULT = {} & ESAL^{0.7419} \left[0.03641 - 0.02921 \, (BASE) \right] \\ & + 0.2754 \left((AGE)(FI) \right)^{0.01889} - 0.2834 \end{aligned}$$

where:

FAULT	=	repair leave joint faulting, in
ESAL	=	Cumulative 18-kip [80-kN] ESAL since repair construction, millions
BASE	=	0 for granular base throughout, 1 for stabilized material (e.g., CAM, CTB, or BAM) anywhere in the pavement structure
AGE	=	repair age, years
FI	=	Corps of Engineers Freezing Index

Statistics:

R^2 = 0.4063
SEE = 0.048 in
n = 113

Several other variables were determined to significantly impact full-depth repair faulting through correlation analyses (e.g., dowel bar bearing stress, the use of pavement drainage systems and tied shoulders, slab length and repair length), as previously described. The inclusion of these variables did not significantly improve the faulting model over the small database used (113 repairs), which contained relatively constant values for these variables. Attempts at model development using the entire database (including repairs that were <u>not</u> diamond ground after construction) often included these additional variables, but the variability of the faulting data was such that an R-squared of no more than 0.18 could be obtained.

AGE and ESAL exhibited relatively weak relationships with faulting, but were included as multipliers for other variables to provide a means of modelling increases in faulting over time.

The sensitivity of the repair leave joint faulting model is presented in figures 54 and 55. Figure 54 shows the sensitivity of the model to various levels of traffic and base support and the interaction of these factors. The benefit of using stabilized base materials is clear, with predicted faults of 0.1 in [0.25 cm] and less after 20 years of heavy truck traffic (up to 1×10^6 18-kip [80-kN] ESAL/yr). The use of only granular base materials is predicted to produce much larger faults for even short periods of time or relatively light truck traffic. Increases in heavy traffic volume produce corresponding increases in faulting, although the effects of traffic are diminished when stabilized base materials are present, as would be expected due to the resistance of the stabilized materials to pumping erosion and faulting.

The effect of freezing climates (or spring thaws?) is presented in figure 55. Repairs placed in nonfreeze climates are predicted to perform much better than those placed in freezing climates, all other design factors (including drainage, joint design, etc.) being equal. The severity of the freezing climate is predicted to have minimal effect, as indicated by the small change in predicted faulting as the freezing index increases from 250 to 1000.

5.3.2 Transverse Joint Spalling

<u>Preliminary Analysis</u>

Casual examination of the joint spalling data confirmed field observations that repair approach joints were much more prone to spalling than repair leave joints, even where neither joint was sealed. This is explained by the fact that the repairs tend to move horizontally opposite to the direction of traffic, probably due to the tangential shear applied at the repair surface by the passing wheel loads, as described previously. This movement closes the approach joint and causes entrapped incompressibles to produce very high point bearing stresses in the adjacent concrete joint faces. Incompressibles trapped near the top or bottom of the slab usually cause relatively small surface spalls at the top or bottom of the slab. Incompressibles trapped near the slab mid-depth may cause large spalls, compression cracking or blowups. The repair leave joint opens as the approach joint closes, often resulting in the failure of joint seals and the entry of more water and incompressibles, which may cause spalling during periods of pavement expansion.

The few repairs that could be positively identified as having tied approach joints seemed at least initially more resistant to approach joint spalling, but the data is not conclusive since most of these repairs also had sealed joints.

Since the approach joint was found to be more critical than the leave joint for spalling, subsequent analyses and the discussion contained in this report are directed primarily toward repair approach joint spalling.

<u>Factors Affecting Full-Depth Repair Approach Joint Spalling</u>

Table 23 presents the correlation coefficients and their significances for some key variables that have often been considered related to the development of transverse joint spalling. Although the correlation coefficients appear to be relatively low for many of the included variables, the relationships must be considered highly significant considering the size of the database (1113 outer lane repairs) and the wide range of design, climate and traffic condition combinations included.

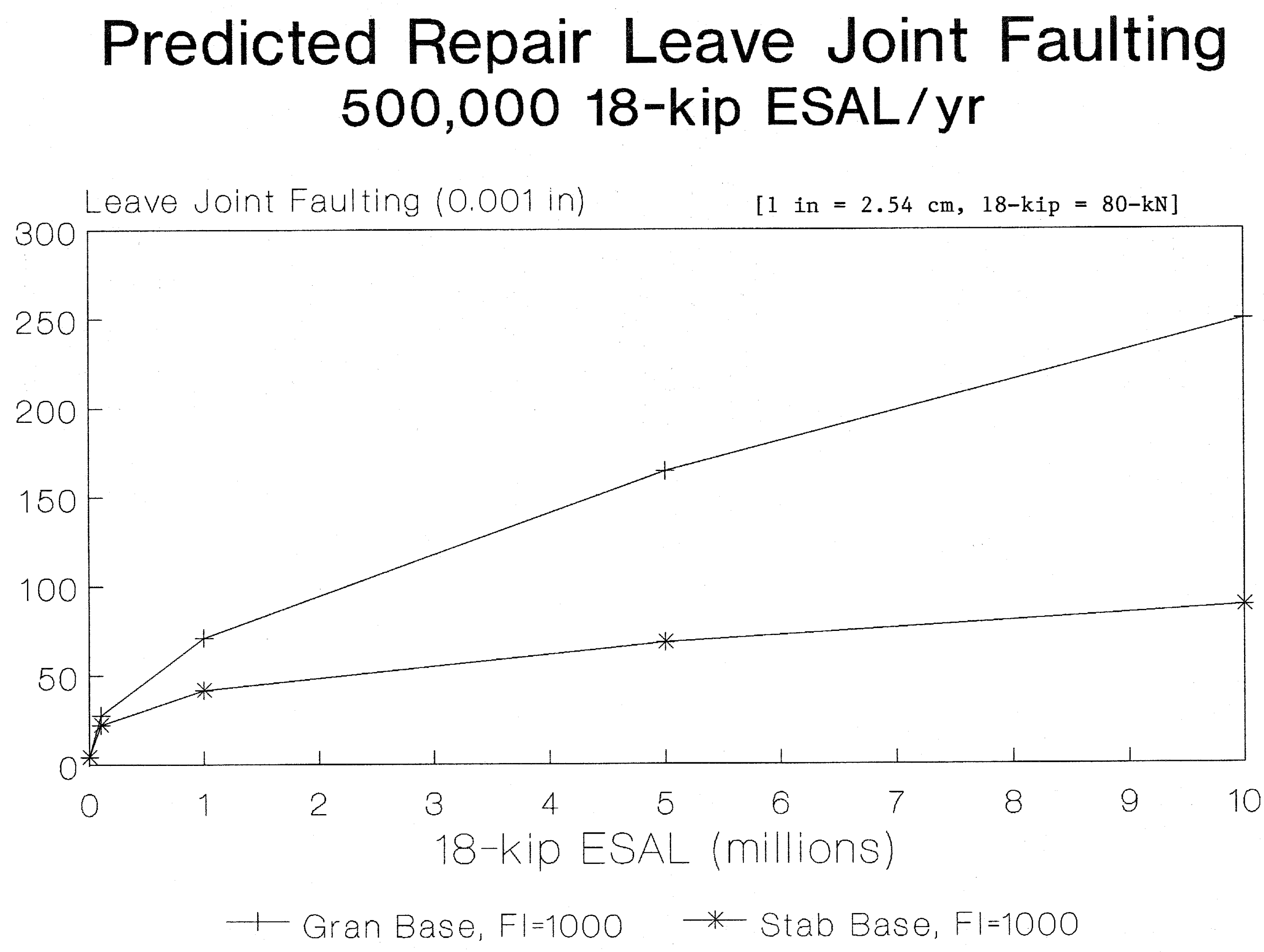


Figure 54. Sensitivity of repair leave joint faulting model to traffic and base support.

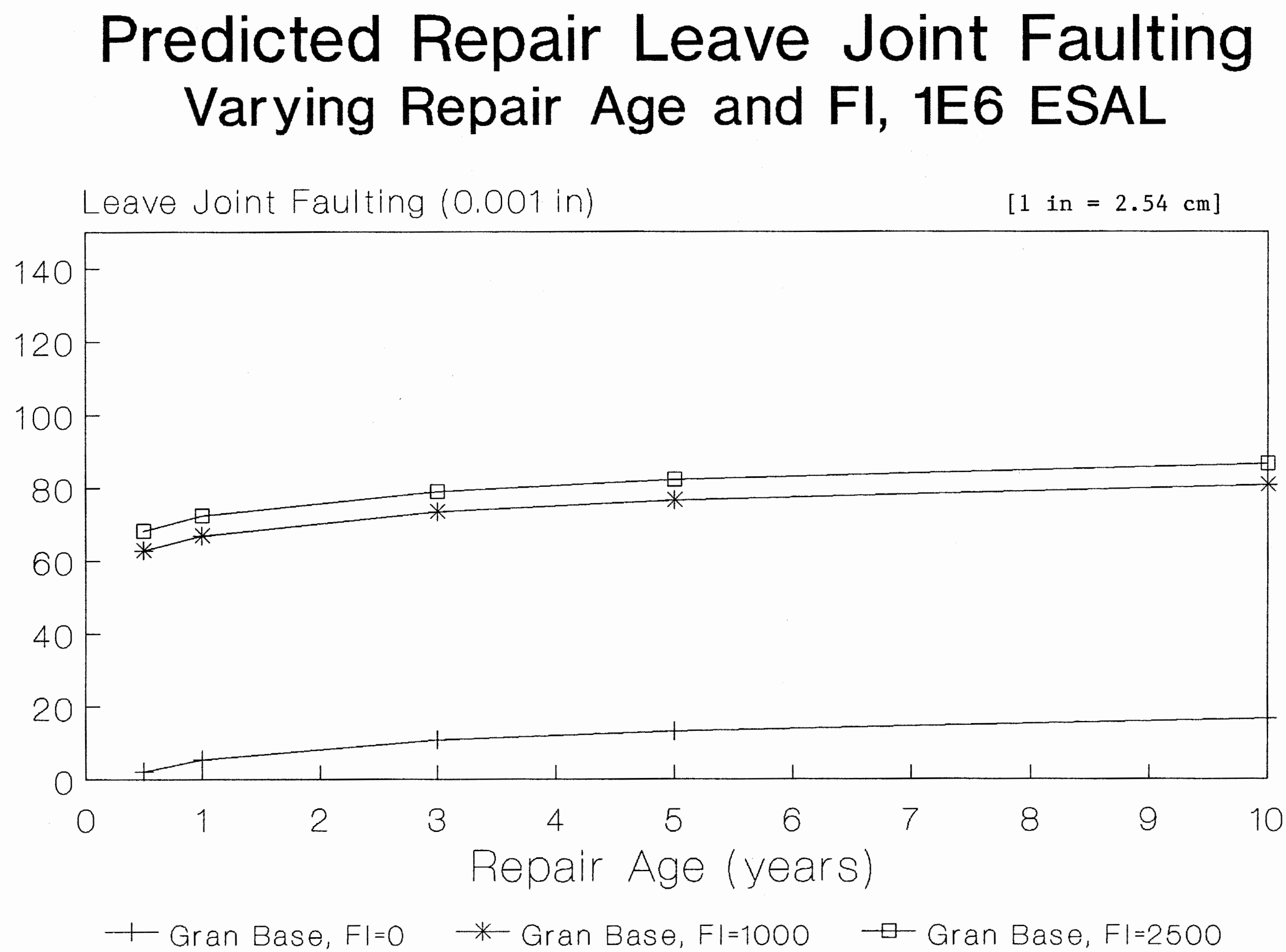


Figure 55. Sensitivity of repair leave joint faulting model to climatic factors (Freezing Index and age).

Table 23 Correlation coefficients and their significances for key variables related to repair approach joint spalling (outer lane - 1113 cases)

Zero-Order Partials

Variable:	ESAL	ln(ESAL)	Joint Seals (0=No,1=Yes)	Seal Damage (0=No,1=Yes)
Coefficient*:	0.0808	0.1174	-0.2325	0.2802
Significance:	0.003	0.001	0.001	0.001

Variable:	Tied Shldr (0=No,1=Yes)	Drainage (0=No,1=Yes)	Base Type (0=Gr,1=St)	Joint Spacing
Coefficient*:	-0.2295	-0.1092	-0.0885	0.1952
Significance:	0.001	0.001	0.002	0.001

Variable:	Joint Width (Measured)	Repair Length	Reactive Aggregate (0=No,1=Yes)	"D" Cracking (0=No,1=Yes)
Coefficient*:	-0.2790	-0.1422	0.0839	0.1264
Significance:	0.001	0.001	0.003	0.001

* - Pearson Product-Moment Correlation Coefficient

The installation (immediately after repair construction) and maintenance of repair transverse joint seals are among the most important factors in preventing repair approach joint spalling because they prevent the introduction of foreign materials into the joints. The table also shows that narrower joint widths (as measured at the time of survey) contribute greatly to increased joint spalling as the joints close on entrapped incompressibles or joint irregularities.

Increased levels of heavy traffic correlated highly with increased amounts of spalling. This is presumably due to repeated differential vertical movement across the joint, which may cause fraying of the joint or failure of the sealant, again resulting in the introduction of foreign materials.

Repair length and original slab contraction joint spacing also exhibited strong relationships with approach joint spalling. Longer repairs appear to be more resistant to spalling. This may be because longer repairs are more stable (as indicated by their previously described resistance to pumping and faulting) and experience smaller vertical movements at the repair joints, which can produce spalling, especially when foreign materials are trapped within the joints. Longer surrounding slab lengths resulted in increased spalling, presumably due to greater thermal movements and accompanying joint closures.

Repairs constructed on projects with tied concrete shoulders typically exhibted less spalling than repairs constructed adjacent to bituminous and granular shoulders. This is probably because the tied shoulder provides better support to the entire pavement, resulting in smaller vertical joint movements. Furthermore, concrete shoulders are not likely to deteriorate and provide a source of incompressibles which could be trapped in the joints.

Higher levels of support for the repair and pavement also appeared to reduce the occurrence of spalling. The use of pavement drainage systems and stabilized base materials provides a more stable platform for the repair and results in decreased vertical movement at the repair joints. Stabilized base materials may also restrain the horizontal movement of the repair and surrounding slabs, resulting in reduced horizontal joint movements and reduced spalling as well.

Finally, the presence of reactive or "D"-cracked aggregates was associated with higher incidences and severities of repair joint spalling, as would be expected.

<u>Spalling Performance Model</u>

The following model was developed to predict the development of full-depth repair approach joint spalling, which was observed to be more critical to repair performance than repair leave joint spalling (as discussed previously):

$$\begin{aligned} \text{SPALL} = [\, & \text{ESAL}^{0.0708} \, [\, 666 - 457\,(\text{SEAL}) \\ & + 0.686\,(\text{JTSPACE})^{1.20} + 131\,(\text{BADAGG}) \\ & - 227\,(\text{JTWIDTH})^{0.463} + 55.4\,(\text{DAMAGE}) \\ & + 9430 \,]\,/\,1000 \end{aligned}$$

where:

SPALL = 1 - None/Low Severity, 2 - Medium Severity, 3 - High Severity

ESAL = 18-kip [80-kN] ESAL applications since repair placement, millions

SEAL = 1 if repair joints sealed at placement, else 0

JTSPACE = Original pavement contraction joint spacing, ft

BADAGG = 1 if reactive or "D" cracking aggregates observed, else 0

JTWIDTH = Measured approach joint width at time of survey, in

DAMAGE= 1 if approach joint sealant is missing, failed or incompressibles were observed in the joint, otherwise 0

Statistics:

R^2 = 0.3671
SEE = 0.189
n = 1102

Correlation analyses were used to identify several other variables that significantly affect full-depth repair spalling. These variables include repair length and the use of tied concrete shoulders, pavement drainage systems and stabilized base materials. However, the inclusion of these variables did not significantly improve the spalling model.

While the R-squared value appears to be somewhat low, it must be considered highly significant considering the size of the database (1102 outer lane repairs) and the wide range of design, climate and traffic condition combinations included.

The sensitivity of the repair approach joint spalling model is presented in figures 56 and 57. Figure 56 illustrates the tremendous effect of using and maintaining repair joint seals. Even after many years of heavy truck traffic, repairs may resist joint spalling if the transverse joint seals are maintained. The exclusion of joint seals, however, is predicted to result in immediate moderate or greater spalling of the joints.

Figure 57 reiterates the effect of good joint sealant construction and maintenance practices and also illustrates the predicted increase in repair spalling that might accompany longer joint spacings.

5.3.3 Transverse Cracking of Full-Depth Repairs

Very few of the surveyed repairs exhibited either transverse or longitudinal cracking and the conditions that appeared to have contributed to cracking on project often had no effect on similar repairs at other projects. Thus, it was difficult to develop predictive models for transverse cracking that were of any significance and no such models are included in this report. Correlation analyses did suggest some interesting relationships, however, and these are discussed below.

Factors Affecting Transverse Cracking of Full-Depth Repairs

Table 24 presents the correlation coefficients and their significances for some key variables that have often been considered related to the development of transverse cracking of full-depth repairs. Although the correlation coefficients

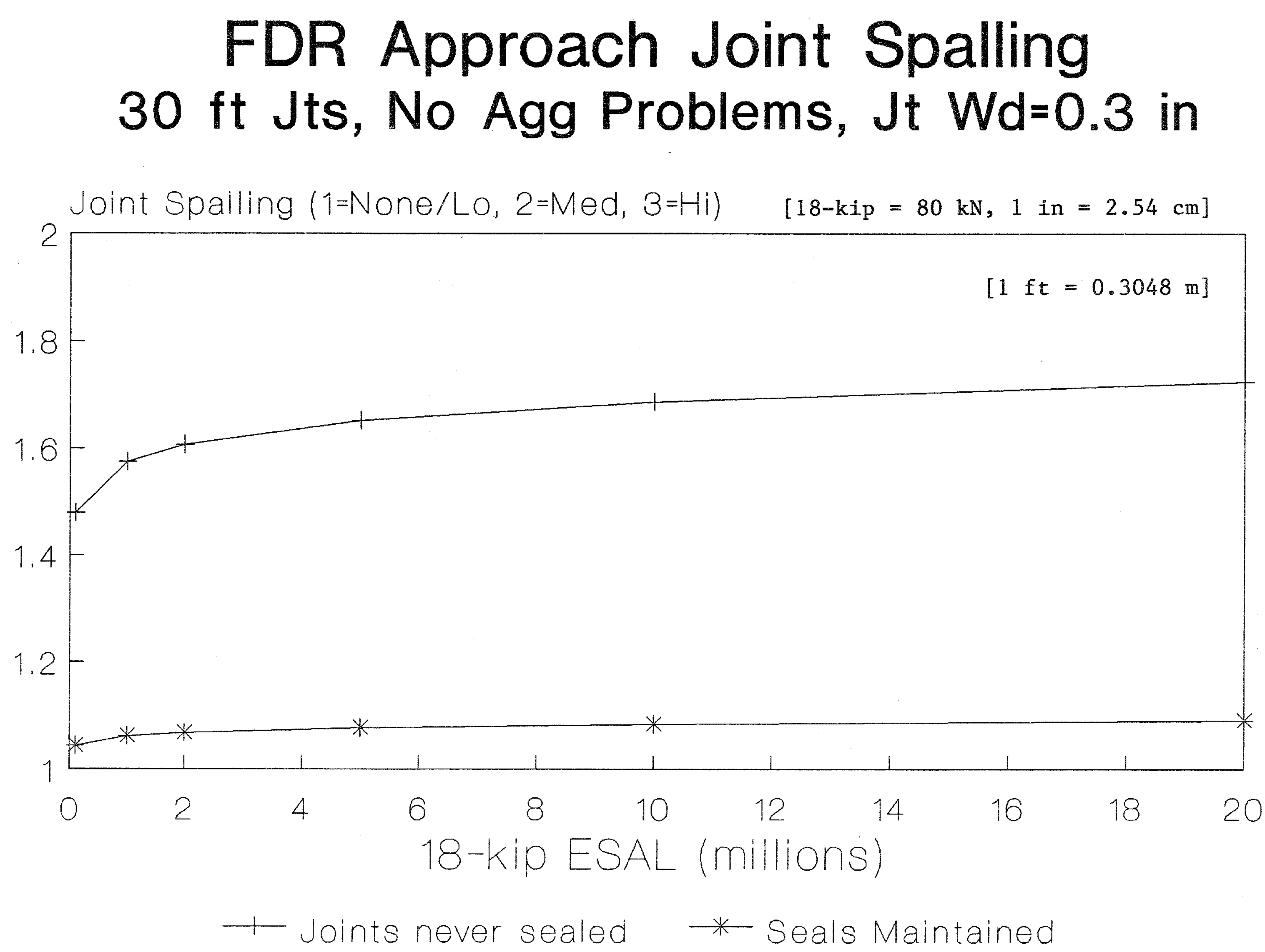


Figure 56. Sensitivity of repair approach joint spalling model to traffic and the use and maintenance of repair joint seals.

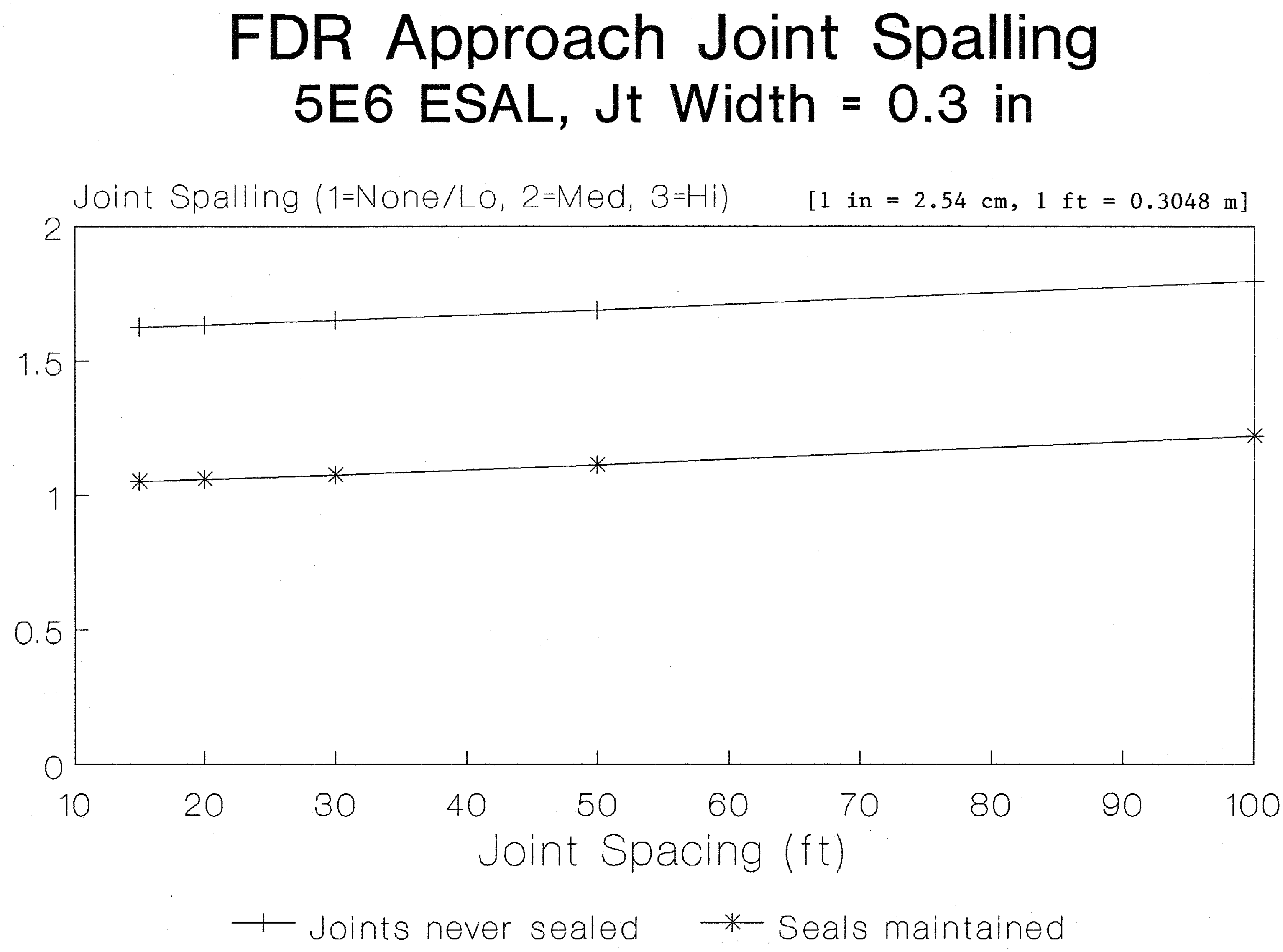


Figure 57. Sensitivity of repair approach joint spalling model to contraction joint spacing and the use and maintenance of repair joint seals.

Table 24 Correlation coefficients and their significances for key variables related to transverse cracking of full-depth repairs (outer lane - 1113 cases).

Zero-Order Partials

Variable:	ESAL	Tied Shldr (0=No,1=Yes)	Drainage (0=No,1=Yes)	K-Value
Coefficient*:	0.0773	-0.0101	0.0373	0.0617
Significance:	0.005	0.369	0.107	0.020

Variable:	Joint Spacing	Repair Length	Approach Seal Damage (0=No,1=Yes)	Leave Seal Damage (0=No,1=Yes)
Coefficient*:	-0.0982	0.1304	0.0712	0.0560
Significance:	0.001	0.001	0.009	0.031

Variable:	Long. Cracking
Coefficient*:	0.1872
Significance:	0.001

* - Pearson Product-Moment Correlation Coefficient

appear to be relatively low for many of the included variables, many of them must be considered highly significant considering the size of the database (1113 outer lane repairs) and the wide range of design, climate and traffic condition combinations included.

Cumulative traffic, foundation support and repair length were three design variables that correlated well with full-depth repair transverse cracking. Higher volumes of heavy traffic, stronger foundations and longer repairs all appear to contribute to increased transverse cracking. Similar observations have been made concerning the development of transverse cracking in regular pavement construction where long slabs on stiff foundations experience higher curling stresses than shorter slabs on weaker foundations. The combination of curling and traffic stresses in the worst cases (long slabs, high traffic and stiff foundation) often produces transverse slab cracking. The same mechanism is probably applicable to full-depth repair slabs.

A study by Ortiz, et. al. found that repairs greater than 6 ft [1.8 m] in length were susceptible to transverse cracking and those less than 3 ft [0.9 m] in length were susceptible to longitudinal cracking, suggesting that repair lengths between 3 and 6 ft [0.9 and 1.8 m] should be selected (where feasible) to minimize repair cracking.(73)

Longer original pavement slabs appear to reduce transverse repair cracking, although the reason for this reduction is not apparent.

The use of tied concrete shoulders and pavement drainage systems exhibited only weak correlations with transverse repair cracking.

The appearance of transverse repair cracking was highly correlated with longitudinal repair cracking, which may be an indication that the development of either (or both) is a sign of severe structural deficiency (e.g., weak concrete, insufficient thickness, etc.). Transverse repair cracking was also highly correlated with repair joint seal damage, but this is probably because sealant damage was a major factor in the development of longitudinal cracking. Sealant damage should have little effect on transverse cracking.

5.3.4 Longitudinal Cracking of Full-Depth Repairs

Few of the surveyed repairs exhibited either transverse or longitudinal cracking and the conditions that appeared to have contributed to cracking on project often had no effect on similar repairs at other projects. Thus, it was difficult to develop predictive models for longitudinal cracking that were of any significance and no such models are included in this report. Correlation analyses did suggest some interesting relationships, however, and these are discussed below.

<u>Factors Affecting Longitudinal Cracking of Full-Depth Repairs</u>

Table 25 presents the correlation coefficients and their significances for some key variables that have often been considered related to the development of longitudinal cracking of full-depth repairs. Although the correlation coefficients appear to be relatively low for many of the included variables, many of them must be considered highly significant considering the size of the database (1113 outer lane repairs) and the wide range of design, climate and traffic condition combinations included.

Table 25 Correlation coefficients and their significances for key variables related to longitudinal cracking of full-depth repairs (outer lane - 1113 cases).

Zero-Order Partials

Variable:	ESAL	Age*Freeze Index	Drainage (0=No,1=Yes)	K-Value
Coefficient*:	0.0326	-0.0819	0.1442	0.2188
Significance:	0.139	0.003	0.001	0.001

Variable:	Joint Spacing	Repair Length	Approach Seal Damage (0=No,1=Yes)	Leave Seal Damage (0=No,1=Yes)
Coefficient*:	-0.0128	-0.0528	0.1463	0.1397
Significance:	0.335	0.039	0.001	0.001

Variable:	Transverse Cracking	Approach Joint Spalling	Leave Joint Spalling	Dowelled/Tied Repair Jts. (0=No,1=Yes)
Coefficient*:	0.1872	0.1201	0.1298	-0.2405
Significance:	0.001	0.001	0.001	0.001

* - Pearson Product-Moment Correlation Coefficient

Joint sealant maintenance practices and joint spalling were found to correlate highly with repair longitudinal cracking, suggesting that at least one mechanism for longitudinal repair cracking is compression cracking caused by high point bearing stresses on entrapped incompressibles. Shorter repairs would be more susceptible to this type of cracking, and the table shows that longitudinal cracking was observed to decrease as repair length increased. Original pavement slab length did not significantly affect repair longitudinal cracking.

The possibility of longitudinal cracking as a load-related distress is suggested by the observation that increased foundation stiffness and the use of pavement drainage systems reduce its development. However, increases in traffic were not found to significantly affect the development of longitudinal repair cracking, which indicates that this distress is probably not triggered primarily by load-related mechanisms. It is more likely that the use of drainage systems and stronger (stabilized) foundations reduces the pumping and entry into the repair joints of incompressibles from below the pavement, which would otherwise cause the compression cracking described above. The use of tied and dowelled repair joints (rather than undercut or aggregate interlock joints) was also found to have a very significant effect on the reduction of longitudinal repair cracking. Reduced deflection through improved load transfer would also reduce the development of pumping and the entry of base materials into the repair joints from below.

5.4 LABORATORY SHEAR TESTING OF DOWELS ANCHORED IN CONCRETE

5.4.1 Introduction

Due to the almost complete lack of research data on load transfer systems for full-depth repairs, it was concluded during the early work in this contract that progress would be greatly hampered if some basic research work was not performed related to repeated load performance of dowels anchored into the face of slabs. Thus, a major laboratory experiment was planned and conducted. The design, conduct and results of this experiment are summarized herein and are described in detail in volume IV (chapter 3).

5.4.2 Experimental Design

The general concept of the study involved the application of repeated shear loads to dowels of various dimensions anchored in holes drilled in concrete specimens obtained from an inservice Interstate highway and the collection and analysis of dowel load and deflection data at several points during the load history of each dowel.

The effects of five design and construction variables -- dowel diameter, annular gap (the width of the void to be filled with anchor material when the dowel is placed in the exact center of the drilled hole), anchor material, embedment length and drill type (varying drill impact energy) -- on the deflection response of dowels in full-depth repairs to repeated shear loads were investigated. Two test levels were selected for each variable except for drill type, for which three "levels" or types were selected. A replicated half-fraction factorial experimental design was employed to provide a statistical basis for determining the main effects and interaction effects of the five variables under consideration. Table 26 summarizes the test values that were selected for each of the variables.

Tests were also conducted on a number of "special" specimens, including two specimens with dowels cast in place in the lab, two specimens with dowels turned on a lathe to provide a very tight friction fit, and one specimen with a large diameter

Table 26. Summary of test values used in dowel bar repeated shear tests.

VARIABLE	Low Value	Medium	High Value
Dowel Diameter	1 [2.5 cm]		1.5 [3.8 cm]
Annular Gap	1/32 [0.08 cm]		1/8 [0.3 cm]
Anchor Material	Cement Grout (Dayton Superior Sure-Grip -- "Flowable" Mix)		Epoxy Resin (Hilti HIT C-10)
Embedment Length	7 [17.8 cm]		9 [22.9 cm]
Drill Type	Standard Pneumatic	Hydraulic Percussion (TAMROCK)	Electro-Pneumatic (Hilti, Inc.)

hollow stainless steel dowel. These tests were conducted for comparison purposes and to provide an indication of future research needs.

5.4.3 Preparation of the Test Specimens

Portland cement concrete slabs for fabricating test specimens were obtained from the outside eastbound lane of Interstate 70 near milepost 89, west of Effingham, Illinois. The highway was constructed in 1962, and had accommodated approximately 13.8 million 18-kip [80-kN] single-axle loads in the design (outside) lane from the date of construction to the date of removal.

Four undamaged 4 ft by 12 ft [1.2 m by 3.7 m] slabs were lifted out and cut into 18-in [45.7 cm] by 12 in [30.5 cm] test specimens. Eighteen usable test specimens were obtained from each slab. Cores were also obtained from each slab for compression, split tensile, and elastic modulus testing.

Sand-cement mortar "caps" were cast on the bottom of each specimen to provide a level base for drilling and testing.

A drilling frame was assembled to hold the specimens and drill rigs in place during drilling. Drilling dust and loose particles were removed using a large test tube brush and compressed air.

The dowels were installed horizontally by injecting sufficient anchor material into the backs of the drilled holes to cause material extrusion when the dowels were inserted. The dowels were allowed to settle or tip in the holes as the anchor material cured. A tight-fitting nylon disk, 2 in [5.1 cm] larger in diameter than the dowel and approximately 3/32 in [0.24 cm] thick was fixed on each dowel at a distance equal to the embedment length from one end of the dowel (see figure 58). These disks were used to prevent the anchor material from flowing out of the holes and creating voids around the dowels. They also forced the anchor material to fill spalls near the dowel hole on the concrete face caused by the drill.

The nylon disks were removed after 24 hours and the anchor material was inspected for surface voids or other visible faults that would affect test results.

An effort was made to test the cement grout specimens no sooner than 7 days and no later than 14 days after preparation. A similar effort was made to test the epoxy resin mortar specimens no sooner than 24 hours and no later than 7 days after preparation.

Two specimens were prepared with 1-in [2.5 cm]-diameter dowels cast-in-place with 9 in [22.9 cm] of embedment. These specimens were cured for 24 hours, subjected to 5000 load cycles (to simulate early opening of the repair), cured for an additional 27 days, and subjected to an additional 595,000 load cycles. The purpose of these specimens was to set a standard of deflection performance against which to compare the anchored dowels, and to simulate the conditions imposed on the end of the dowel embedded in the repair.

Specimens were also prepared to test the performance of dowels installed to an embedment length of 9 in [22.9 cm] in very close-fitting holes. The inside diameter of holes drilled in two specimens using 1.0625-in [2.7 cm] nominal-diameter drill steels mounted in the Hilti drill was measured, and 1.25-in [3.2 cm] dowels were turned on a metal lathe to achieve dowel diameters 0.02 in [0.05 cm] less than the

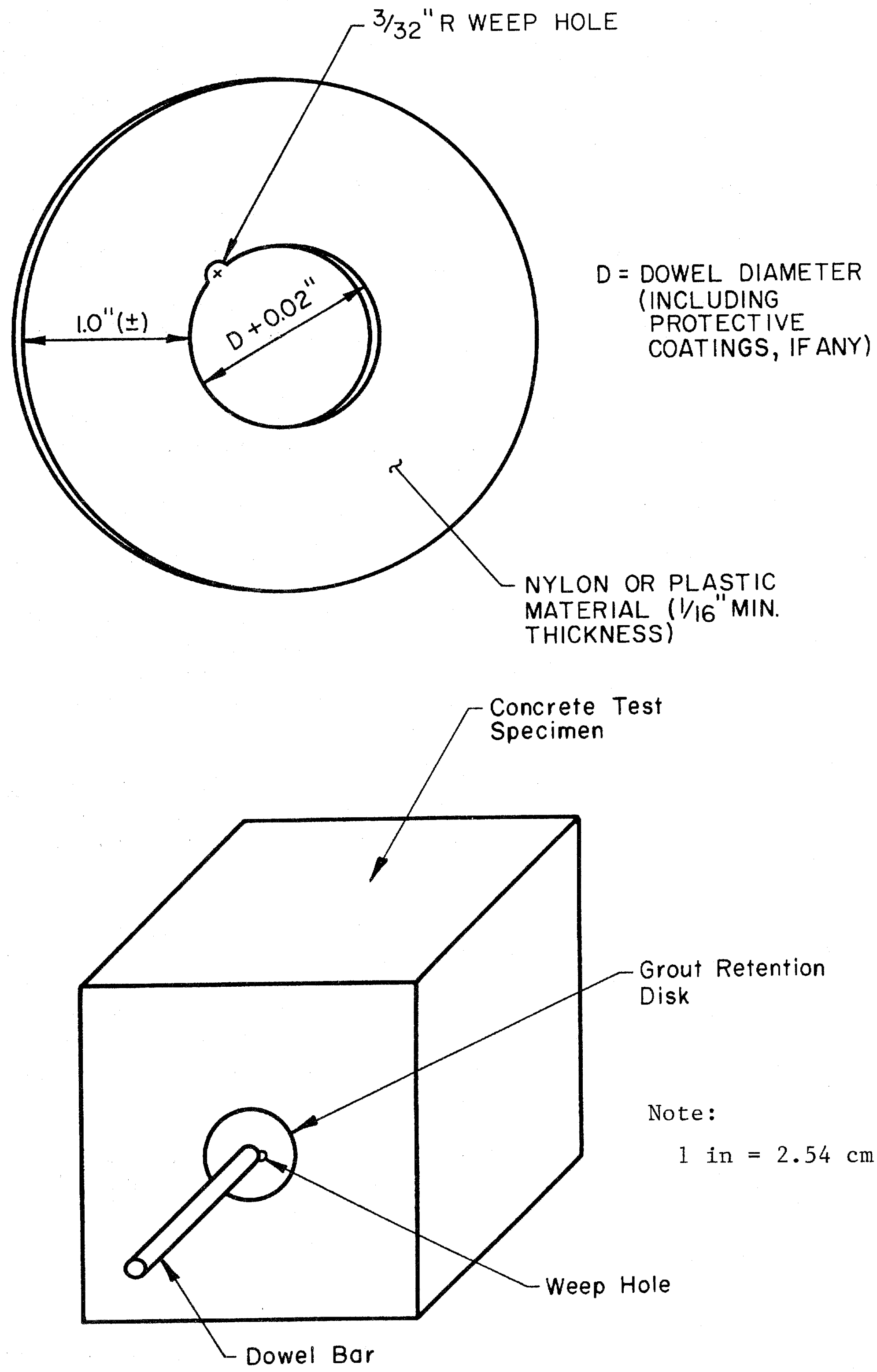


Figure 58. Illustration of grout retention disk used in lab experiment.

smallest diameter measured in each hole. Insertion of the dowels showed that the one of the two was loose enough to be moved slightly in any direction. The other dowel could not be inserted to full-depth by hand and was forcibly inserted without anchor material using a large hammer. This caused the formation of a vertical crack through the center of the face of the specimen, although the crack did not deteriorate under test conditions.

5.4.4 Description of the Test and Related Equipment

Repeated bidirectional vertical shear loads were applied to the test specimens. The load function finally utilized was a continuous sinusoidal form with a peak magnitude of ± 3000 pounds [13.36 kN] and a frequency of 6 Hz (see figure 59). Loads were applied at the rate of nearly 520,000 per day, allowing the application of about a year's worth of heavy traffic loads to a single dowel installation each day.

The specimens were clamped to a steel plate and the applied loads were generated hydraulically using an MTS Model 661 ram with an 11 kip [50 kN] capacity, which was controlled by a simple sine wave function generator. The load was applied to the dowel through a specially fabricated high-strength steel loading collar which was clamped to the dowel using large "set" screws. This collar allowed vertical deflection and associated angular movement of the dowel about a lateral axis.

A linearly varying deflection transducer (LVDT) was mounted on a bracket attached to the face of each specimen and connected to the load collar using a small nylon screw. This device was used to measure the movement of the dowel relative to the PCC specimen. The MTS load cell data was also collected for analysis and was used to assist in the computer control of the test.

The entire test operation was controlled by an IBM Personal Computer using a Data Translations DT-2801A Analog/Digital (A/D) board and a controlling program written in BASIC using the PCLAB library of A/D board control subroutines.(38) Figure 60 shows the entire test assembly arrangement.

Deflection and load data were typically collected during ten load cycles immediately after the completion of 1, 2000, 5000, 20000, 100000, 300000 and 600000 load cycles. Extended test data was also collected after 1,200,000, 2,000,000 and 4,000,000 load cycles for certain specimens. This data was stored on floppy disk with appropriate identification data for later analysis. Data reduction programs were written and used to identify average peak load, deflection, and dowel looseness conditions during each data sampling.

The reduced and summarized design and performance data was loaded into an SPSS database and a Lotus 1-2-3 spreadsheet for analysis, production of graphs, etc.(50,51)

5.5 LABORATORY STUDY RESULTS

5.5.1 Preliminary Results and Observations

Observations of the preparation and testing (and occasional failure) of the test specimens provided some insight into the performance of full-depth repairs.

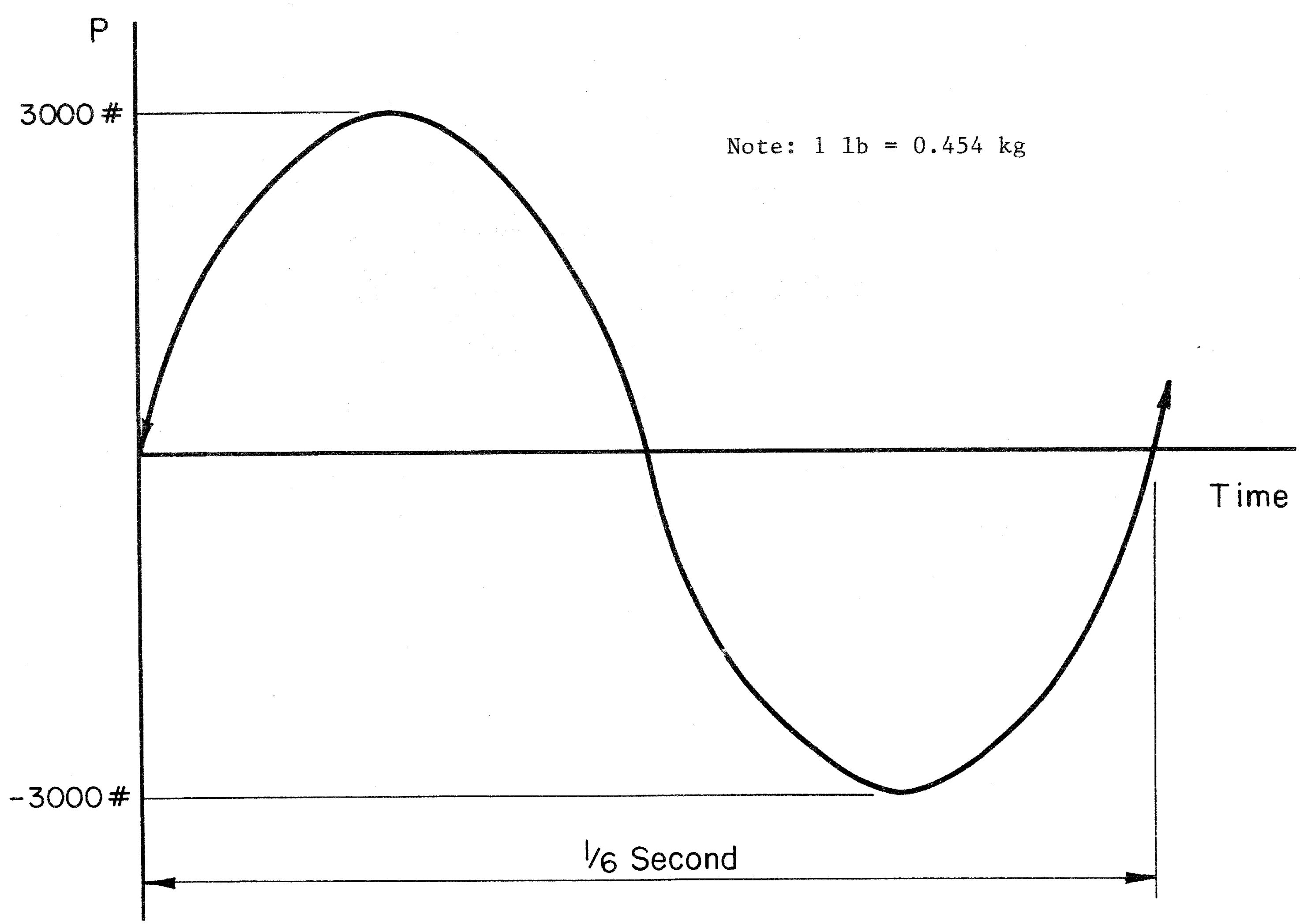


Figure 59. Illustration of load function used for lab testing.

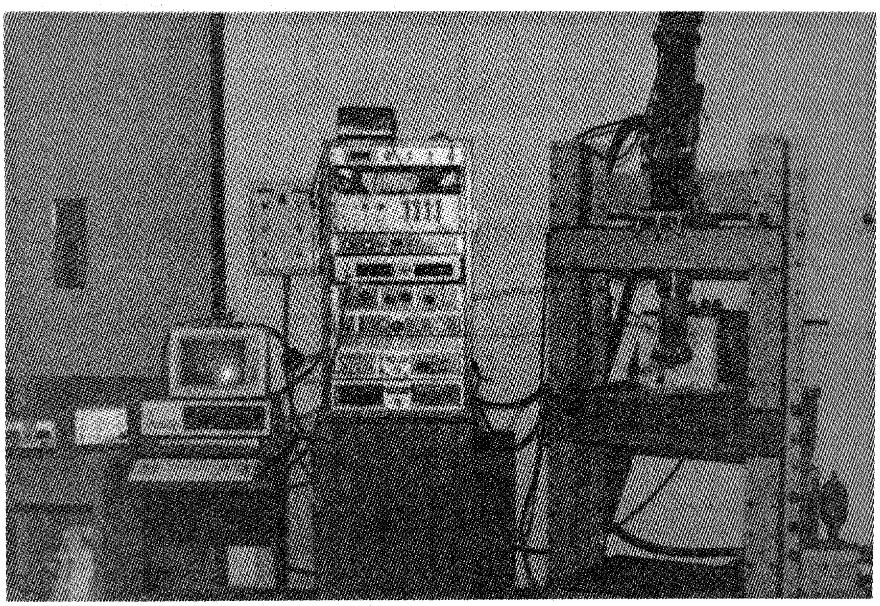


Figure 60. Repeated dowel load test assembly.

Effect of Drill Impact Energy on Spalling

Drills that impart high impact energy produce more spalling on the concrete face near the drilled hole than drills using low impact energy. The electric-pneumatic drill was most acceptable in minimizing spalling, but the reduced impact energy resulted in a three to fourfold increase in the time required to drill each hole. The hydraulic drills provided a substantial reduction in spalling with no discernible increase in drilling time. The excessive spalling produced by the pneumatic drill was usually repaired easily by using the nylon dowel rings to retain the anchor material, and the performance of these "repaired" specimens was equal to similar specimens prepared using other drills.

Consistency of Dowel Anchor Materials

The installation of dowels using cement grout was often difficult. Specimens prepared immediately after mixing the grout received a grout that was almost "pourable," and retention of the grout was difficult, even using the nylon rings. Large voids were often observed around these dowels prior to testing and their deflection profiles were often exaggerated. Specimens that were prepared 5 minutes after the grout was mixed received a grout that was of the desired consistency, were found to have only very small voids and performed relatively well. Specimens that were prepared 10 minutes or more after mixing the grout received a very stiff grout that often compacted at the back of the hole, preventing proper installation of the dowels, rather than extruding out as the dowels were inserted. These specimens had to be cleaned out and grouted again using a more flowable grout.

The wide variation in grout consistency over a relatively short period of time in the highly controlled environment of the laboratory makes questionable the use of the same material in the field, where conditions can be much more harsh and quality control often takes a back seat to production. Field installations require a reliable, easy-to-use dowel installation material. Cement grout does not consistently meet these requirements.

The epoxy mortar used was almost always proportioned accurately and mixed thoroughly using a hand-held double-barrel caulking gun delivery system which produced a mortar that was the desired consistency.

The cost of the epoxy mortar is currently substantially higher than the cost of the cement grout, but the reliability and the uniform consistency of the epoxy should make it the preferred material.(43) Recently-developed epoxy delivery equipment using much larger cartridges and typical discounts for the purchase of large quantities should reduce the cost of the epoxy for field installations.

Dowel Failures

Five of the 1-in [2.5 cm] diameter dowels tested experienced brittle fatigue failures at locations 0.75 to 1.5 in [1.9 to 3.8 cm] inside the face of the PCC specimens. This location corresponds approximately with the predicted point of maximum moment in the dowel presented by Friberg based on the work of Timoshenko.(53,54) Variations from the predicted location are probably due to nonuniform support of the dowel at the face due to spalling of the concrete during drilling and spalling of the cement grout mortar during testing due to high dowel bearing stress.

Some of these failures occurred after as few as 40,000 load cycles while others occurred after nearly 600,000 load cycles. Four of the failed dowels were anchored using cement grout while one was anchored using epoxy mortar. Large voids were visible above three of the four grouted dowels prior to testing.

These observations indicate the variability of quality of the cement grout anchor material (in spite of the use of the nylon grout retaining rings) and demonstrate the importance of providing void-free uniform dowel support in pavement joints.

Effectiveness of Nylon Grout Retention Rings

The nylon grout retention rings were clearly very effective in reducing the outflow of anchor materials from the drilled holes and ensuring more uniform dowel support. They also forced excess anchor material into the spalled area created by drilling, effectively repairing the spall and reducing dowel deflections.

The effectiveness of the rings was highly dependent on the fluidity of the anchor material being used. Very fluid cement grouts were difficult to work with and were not retained well, even with the rings. Excellent results were obtained using materials that were "flowable," because they were fluid enough to be moved into the voids, yet viscous enough not to flow appreciably under gravity alone. A smooth, void-free face resulted in these cases.

The use of these rings probably reduced the difference in performance that would have been observed between the two anchor materials and the three drill types if the rings hadn't been used. Based on initial observations, it would be expected that the elimination of the retention rings would result in much more variability of performance for the cement grout specimens. Higher deflections would be associated with more spalling around the drill hole, so better performance would be expected from holes drilled using low-impact energy drills.

5.5.2 Factors Affecting Dowel Deflection and Looseness

For the purposes of this study, dowel deflection refers to the dowel deflection under an applied shear load of ±3000 lbs. [13.4 kN], measured using the LVDT attached to the load collar at a point approximately 1/2 in [1.3 cm] from the face of the specimen.

Dowel looseness was estimated by plotting measured dowel deflection vs. shear load and projecting the slopes of the loading and reverse loading portions of the load-deflection curve at ± 3000 pounds [13.4 kN] back to intercept the deflection axis. This technique was conceptualized by Teller and Cashell and is shown in figure 61.(47)

The half-fraction factorial experimental design employed in the lab tests allowed direct identification of significant effects through analysis of variance (ANOVA) techniques.

These analyses suggest that all of the main variables may significantly affect the development of dowel looseness and sensor deflection and sensor deflection as follows:

Variable Changed	Effect on Deflection/Looseness
Increasing Dowel Diameter	Decrease
Increasing Dowel Embedment	Decrease
Increasing Drill Impact Energy	Decrease
Epoxy Anchor Material (Instead of Cement Grout)	Increase
Increase Annular Gap	Increase
Increase Load Repetitions	Increase

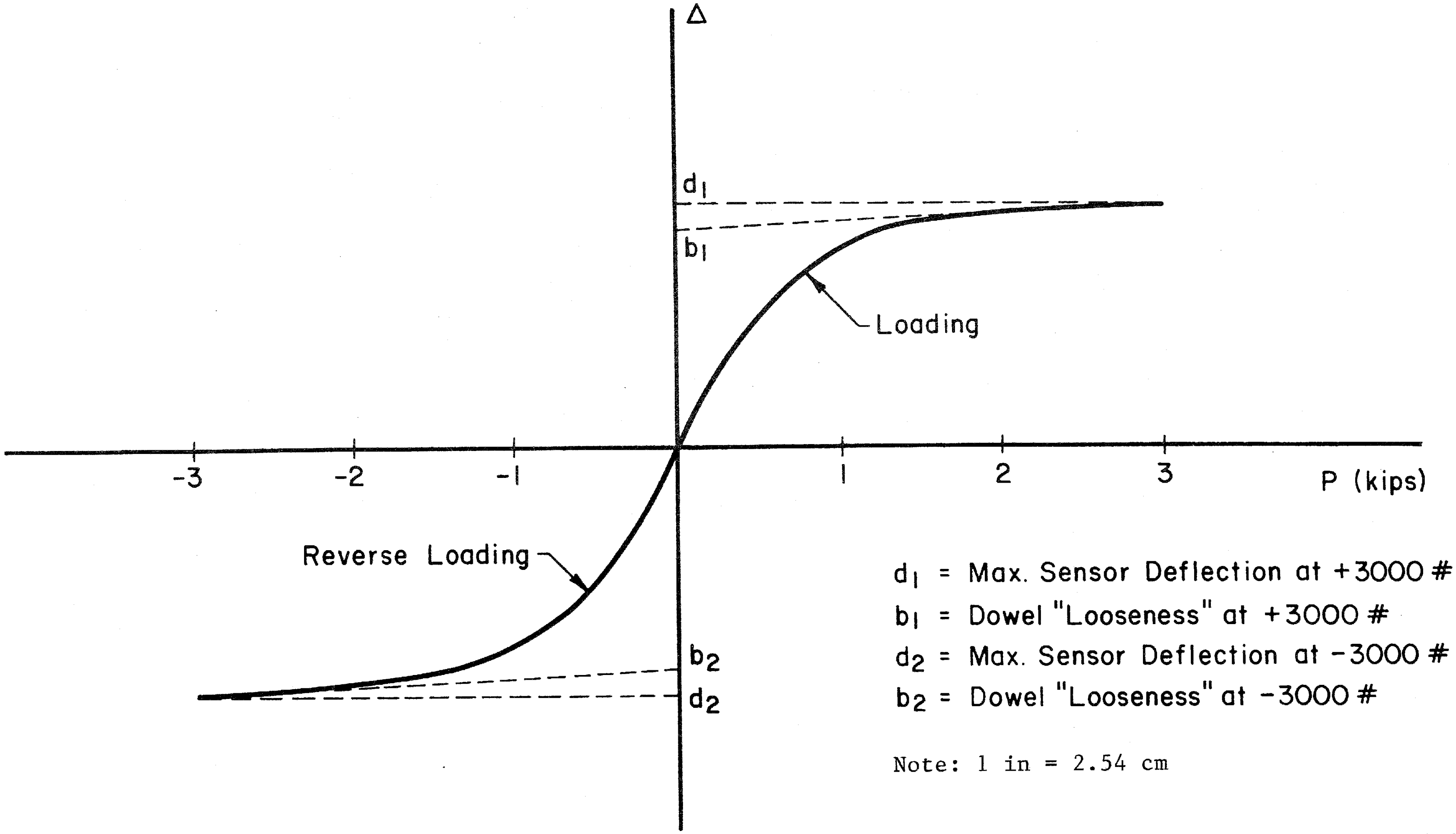


Figure 61. Illustration of estimation of dowel looseness from load deflection curve.(47)

These variables all affect dowel deflection and looseness as expected, with the exception of the effect of drill impact energy. As discussed previously, it is believed that the use of the nylon grout retention rings may have reduced (or eliminated) the effect of spalling caused by the use of high impact energy drills. Since the low impact energy drill was guided but hand-held, the apparent increase in dowel deflection could be due to slight increases in actual drilled hole diameter (which must be filled with a grout that is softer than the surrounding concrete).

Several significant two-factor interactions were also noted, including drill impact energy and dowel embedment length, anchor material and drill impact energy, anchor material and dowel diameter (bearing stress), and anchor material and annular gap. Since many of these two-factor interactions indicated a strong relationship between anchor material and some other variable, the database was subdivided according to anchor material and an analysis of variance was conducted for each of the new data sets. The main effects were still among the most significant in each of the anchor material database subsets. Performance models were developed for each of these data sets.

The strength of the main effects and the significance of several two-factor interaction effects point to additional conclusions concerning the stiffness of the anchor materials. Since the cement grout is more rigid than the epoxy mortar, the effects (and interaction effects) of dowel diameter (bearing stress) and embedment on dowel deflection are reduced for this material. Furthermore, it appears that a larger annular gap generally produces better results for cement grout, presumably because it becomes easier to install the bar in a stiffer grout, which provides more uniform dowel support.

Since the epoxy mortar is a softer material than either the cement grout or the concrete specimen, the deflections of bars embedded in this material are more sensitive to dowel diameter (bearing stress) and embedment, with increases in either resulting in decreased deflections. As annular gap increased, deflections generally increased as well due to the use of larger volumes of softer material. Since the epoxy mortar was always delivered at a uniform consistency that allowed easy insertion of the dowels, there was no apparent need (for installation purposes) for a large annular gap, as with the cement grout. It may be appropriate to use epoxy mortar with the smallest annular gap that will allow dowel installation without excessive force. This would allow the mortar to fill voids and spalls using a minimum thickness of the softer material and allowing the bar to be supported directly by the concrete in many places. Additional research should be conducted to verify this.

5.5.3 Dowel Deflection and Looseness Models

The data sets for each anchor material type were used to develop predictive models for sensor deflection and dowel looseness. Although many factors and interactions appear to affect these performance measures, their inclusion often made the models very complex without significantly improve the accuracy of the models. Satisfactory models were often obtained using nonlinear regression techniques and including only main effects.

The models developed for the epoxy mortar anchor material are presented below:

$$B_{maxmin} = 34840\ (AG) + 1167\ (CT)^{1.058} - 9.899\ (EB)^{1.160} + 1.079\ (BS) - 0.6912\ (EN)^{1.831} + 8380$$

Statistics: $R^2 = 0.594$
$COV = 36.9\%$
$n = 178$

$$D_{maxmin} = 54210\,(AG) + 643.3\,(CT) - 2117\,(EB) + 2.031\,(BS) - 8.822\,(EB)(EN) + 21210$$

Statistics: $R^2 = 0.584$
$COV = 28.7\%$
$n = 178$

where:

B_{maxmin} = Total dowel looseness (as defined previously), mils

D_{maxmin} = Total sensor deflection (as defined previously), mils

AG = (Nominal diameter of drilled hole - Nominal dowel diameter), in

CT = Natural log of number of complete load cycle applications

EB = Dowel embedment, in

BS = Friberg's bearing stress, psi

EN = Estimated drill impact energy, ft-lbs/blow

Figures illustrating the sensitivity of the models to the input parameters are presented in volume IV (chapter 3) and their study produced the following conclusions:

- The epoxy mortar is flexible (when compared to the surrounding concrete) and that thin supporting layers (sufficient to fill drilling voids) are best.
- The epoxy mortar is very resistant to fatigue and undergoes very little permanent deformation or deterioration after many repeated load applications.
- Increases in bearing stress produce proportionate increases in dowel deflection, especially where thick layers of epoxy mortar are present. The model predicts dowel deflection increases of 60-100 percent for bearing stress increases from 1000 psi to 5000 psi [6.9 to 34.5 MPa].
- The flexibility of the epoxy mortar results in increased sensitivity to dowel embedment length because the mortar allows the dowel to deflect slightly inside of the drilled hole, whereas the cement grout has greater potential to hold the bar rigidly. The deflection increase produced by decreasing embedment length from 9 in to 7 in [22.9 cm to 17.8 cm] is approximately 10 percent and is probably not critical.

- The model predicts higher deflections with lower drill impact energy. As discussed before, it is believed that the grout retention rings masked the true effect of drill impact energy by filling the joint face spalls with anchor material and reducing all deflections significantly. The model may be reflecting the use of different drill guide systems for each drill, resulting in variable drilled hole diameters and shapes.

The models developed for the cement grout anchor system are presented below:

$$B_{maxmin} = [\ CT\ (\ -2347 + BS\ (\ 0.762 + 2.604 / EN\ )\ )$$
$$+\ 3883\ ]\ /\ 1000$$

Statistics:
R^2 = 0.647
COV = 61.2%
n = 109

$$D_{maxmin} = (\ 6.072\ (\ BS\ ) - 66.96\ (\ EN\ ) + 13900\ (\ AG\ )$$
$$+\ 572.7\ (\ CT\ ) - 8946\ )\ /\ 1000$$

Statistics:
R^2 = 0.663
COV = 43.3%
n = 110

where:

B_{maxmin} = Total dowel looseness (as defined previously), mils

D_{maxmin} = Total sensor deflection (as defined previously), mils

AG = (Nominal diameter of drilled hole - Nominal dowel diameter), in

CT = Natural log of number of complete load cycle applications

BS = Friberg's bearing stress, psi

EN = Estimated drill impact energy, ft-lbs/blow

It should be noted that these models were developed using only data from specimens that did not fail prematurely and therefore they tend to represent "potential" performance rather than average observed performance. The failed specimens were eliminated because their deflections prior to failure (often from the very beginning) exceeded the capacity of the deflection sensor.

Figures illustrating the sensitivity of these models to the input parameters are presented in volume IV (chapter 3) and their study produced the following conclusions:

- The models suggest increasing dowel deflection with increasing annular gap, which is contrary to the conclusion previously drawn for cement grout installations. This is because the models are based primarily on specimens that performed well (the failed specimens, which had small annular gaps and dowels, produced much unusable data). The predicted effect of annular gap is actually smaller than the variability between measurements for the large dowel diameters.

- Bearing stresses that result from the use of 1-in [2.5 cm] dowels result in deterioration of the anchor material at the joint face. The 1.5-in [3.8 cm] dowels exhibited the smallest increases in deflection under repeated loading and performed acceptably.

- When good installations are achieved using cement grout anchor material, rapid increases in deflection typically occur at first as the dowel becomes "seated" and subsequent increases are generally small. Poor installations exhibited excessive deflections at the start which increased as the dowel impacted the supporting material, causing it to deteriorate.

The results of the lab study can be further illustrated by looking at the deflection profiles and "looseness" envelopes (such as those presented in figures 62 and 63) for various specimens. Each deflection profile consists of four response curves -- loading (lower curve, right side), load relaxation (upper curve, right side), reverse loading (upper curve, left side) and reverse load relaxation (lower curve, left side). Each "looseness" envelope illustrates the development of dowel looseness over time for a given specimen. "Upstroke looseness" is the component of total looseness computed from the reverse loading curve, "downstroke looseness" is the component of total looseness computed from the normal loading curve, and "total looseness" is the distance between the other two curves and corresponds to B_{maxmin} in the regression models.

Deflection profiles and "looseness" envelopes for several specimens are included and discussed in volume IV (chapter 3). A summary of the conclusions drawn from the study of these figures follows:

- When epoxy mortar anchor materials were used, larger annular gaps result in increases in dowel deflection. This was observed at any point in the loading histories of comparable specimens and verifies the model that was developed.

- Reverse loading mode typically produced higher deflections than normal loading for the epoxy mortar specimens. This is presumably due to settlement of the dowel during curing, which results in the dowel bearing on a very thin layer of anchor material on the bottom and a thicker layer on top. Since the deformations are somewhat dependent on the deformation of the supporting layer, the thicker layer on top allows more deflection in reverse loading.

- Dowels properly installed using cement grout typically exhibited lower deflections than those installed using the epoxy mortar. This was observed at any point in the loading histories of comparable specimens. It must be emphasized, however, that it was often difficult to obtain good anchoring using cement grout due to the extreme variability of grout consistency over short periods of time.

- Increasing dowel diameter from 1 in to 1.5 in [2.5 to 3.8 cm] typically produced a tremendous reduction in measured deflections. A 1.5-in [3.8 cm] dowel properly installed in cement grout exhibited a total computed "looseness" of less than 6 mils [0.015 cm] after 600,000 load cycles. Similarly installed 1-in [2.5 cm] dowels exhibited two to four times more deflection and looseness.

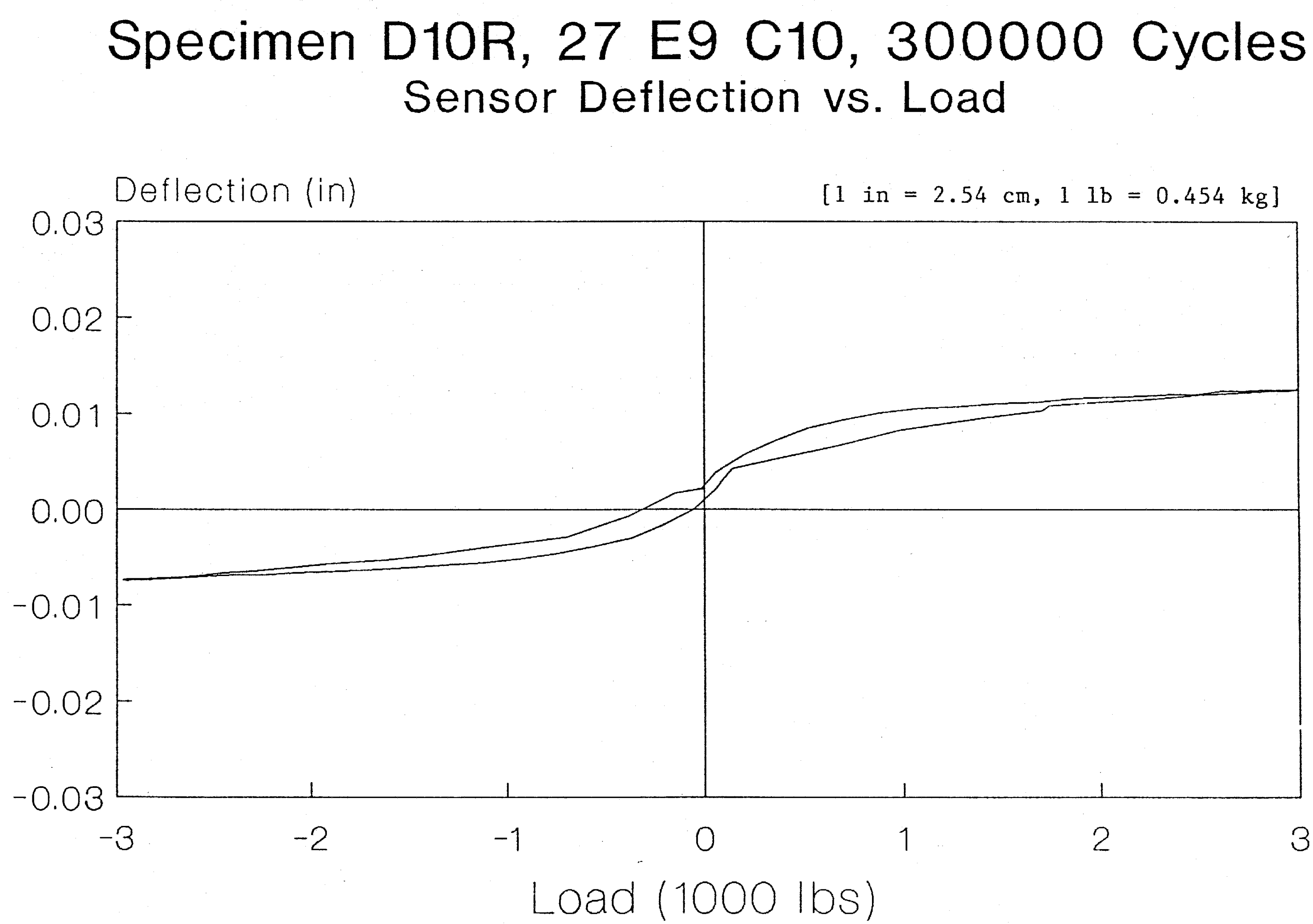


Figure 62. Measured load-deflection profile after 300,000 load cycles for specimen D10R (1-in [2.5 cm] dowel, 1/32-in [0.08 cm] annular gap, 9-in [23 cm] embedment, low-energy drill, epoxy mortar).

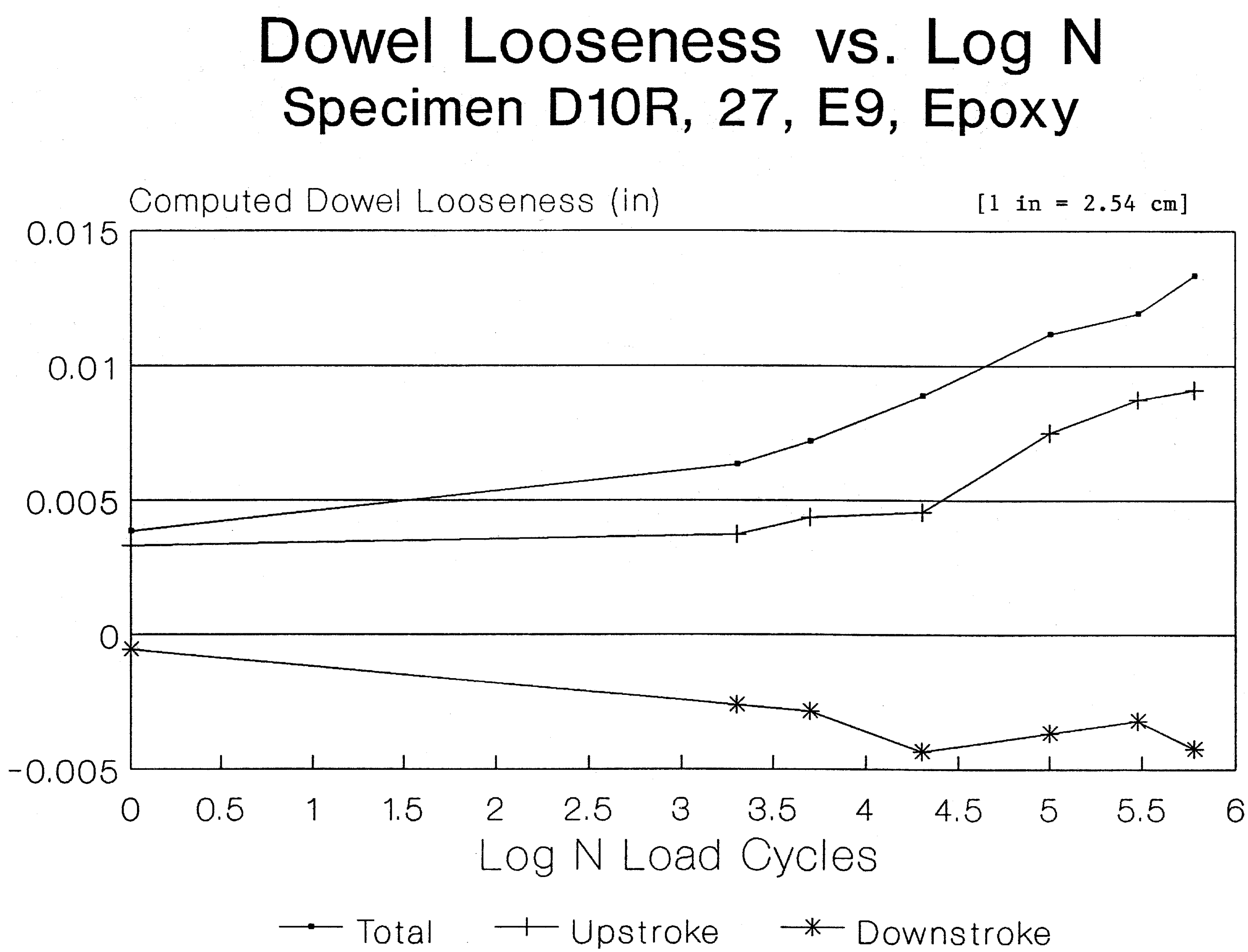


Figure 63. Computed dowel looseness vs. log load cycles for specimen D10R (1 in [2.5-cm] dowel, 1/32-in [0.08 cm] annular gap, 9-in [23 cm] embedment, low-energy drill, epoxy mortar).

- Deflection profiles actually varied very little for different drill types. A slight improvement was noted for the high-energy drill, but it is suspected that this improvement was due to the difference in drill guidance systems rather than impact energy. The use of the grout retention ring is believed to have eliminated the effects of increased impact energy, which resulted in more spalling around the drilled hole and would reduce dowel support if unrepaired.

- The effect of dowel embedment on dowel deflection was typically very small for the range of embedments tested. This confirms other studies which have suggested that embedment lengths of 6 to 7 in [15.2 to 17.8 cm] are adequate for the size dowels currently used in highway applications.

- The cast-in-place specimens exhibited relatively flat deflection profiles, indicating that no real looseness existed at the time of testing and confirms the use of such specimens as an idealized dowel installation. A comparison of this profile to other 1-in [2.5 cm] dowelled specimens suggests that the cement grout specimens have the potential to most closely approach this level of dowel support, particularly when longer embedment lengths and good grout installations are present. The epoxy mortar specimens performed well when the annular gap was small and the embedment length was 9 in [22.9 cm]. The epoxy mortar specimens performed much more consistently than the cement grout specimens.

- The deflection profile for the 1.625-in [4.1 cm] O.D. hollow stainless steel dowel that was installed using the epoxy mortar to a depth of 7 in [18 cm] in a 1.75-in [4.4 cm]-diameter hole was similar to that obtained using a 1-in [2.5 cm]-diameter dowel, 1/8-in [0.3 cm] annular gap and epoxy mortar. A solid bar (or a tube with thicker walls) would probably have provided a more acceptable deflection profile. In addition, the stainless steel did not bond to the epoxy mortar, allowing the bar to be twisted freely after testing, although the bar was not necessarily loose.

- The specimens prepared using "close-fitting holes" and no grout of any type were very loose (compared to the other specimens) and rapidly developed deflections that were beyond the capability of the sensor to measure (>0.05 in [0.13 cm] in either direction). Neither could be tested to the full 600,000 load repetitions because of possible damage to the test equipment. One of the specimens failed after less than 60,000 load cycles.

5.6 DESIGN AND CONSTRUCTION GUIDELINES -- FULL-DEPTH REPAIR

5.6.1 Introduction

These guidelines were originally prepared under NCHRP Project 1-21 and published in NCHRP Report No. 281, Transportation Research Board, 1985. The guidelines were updated in early 1987 based upon the findings and results from the study entitled "Pressure Relief and Other Joint Rehabilitation Techniques" conducted for the FHWA. Further updates resulted from the research described in this final report, "Determination of Rehabilitation Methods for Rigid Pavements," also conducted for the FHWA.

These guidelines present important background information for engineers and technicians involved in designing and constructing projects where full-depth repairs will be placed. These guidelines will also be useful to maintenance engineers and technicians in placing full-depth concrete repairs as part of good pavement maintenance procedures. This document is intended to provide guidance in the preliminary engineering phase.

The procedures and specifications included herein are intended for full-depth repairs and slab replacements which are to be subjected to medium-to-heavy truck traffic over a design life of 10 years or more. These procedures and specifications are applicable to repair projects both with and without overlay.

5.6.2 Need For Full-Depth Repairs

There are several types of deterioration which occur at or near transverse cracks and joints which justify full-depth repair or slab replacement to restore rideability and structural integrity to the concrete pavement. The design engineer must conduct a preliminary condition survey of the project (which may require coring of representative areas) and identify the specific locations and approximate quantities that must be repaired.

The engineer must first determine the causes of joint/crack deterioration. Some typical types of joint/crack deterioration and their causes are listed below:

- Faulting: Heavy-truck-axle loads cause large differential deflections across joints/cracks where poor load transfer exists (typically where no dowels exist), which results in a high potential for pumping and erosion of material beneath the slab and/or stabilized base. If dowels exist, the differential deflection is much lower and thus pumping and faulting is decreased. However, depending upon dowel design, heavy loads can cause high bearing stresses between the dowels and concrete. The result of many repeated heavy loadings can cause the enlargement of the dowel socket, resulting in eventual faulting of the joint. Corrosion of the dowel bars may also be a factor contributing to faulting.

- Spalling: The deterioration of a joint or crack through spalling can be caused by several factors. The major factors are described below:

    a. Infiltration of incompressibles into the joint: This common occurrence results in much of the spalling at joints. The extent of incompressibles in the joint can be determined by visual observations of joints and digging into the joint sealant reservoir with a knife, but is best determined by coring directly through the joint and opening the core to examine the joint faces. Incompressibles can infiltrate from both the top and bottom of the joint.

    b. Disintegration of concrete at the bottom of the joint (non "D" cracked concrete): This is caused by infiltration of incompressibles and large horizontal joint movements. This occurs predominantly in long-jointed reinforced concrete pavement (40-100 ft [12.2-30.5 m]), but can also develop in short-jointed plain concrete pavements where infiltration of incompressibles is extensive. This distress is not initially visible at the surface, but eventually develops into a spall that can be seen at the surface.

    Coring of typical joints prior to full-depth repair to observe the amount of incompressibles and the deterioration at the bottom of the joint greatly assists in identifying this problem.

    Disintegration of the bottom of the slab contributes to a high potential for blowups because less vertical cross-sectional area is available at the joint to bear compressive stress in the slab.

c. "D" cracking or reactive aggregate spalling: "D" cracking is a pattern of cracks caused by freeze-thaw expansion of the aggregate. Reactive aggregate is a cracking pattern caused by the reaction of the aggregate in an alkaline environment. The disintegration and spalling associated with these distresses normally begins near the joints. Cores should be taken to determine the depth of deterioration at different distances from the joint. Four-in [10.2 cm] diameter cores taken at distances of 0, 12, 24, 36, and 48 in [0, 30.5, 61.0, 91.4, and 121.9 cm] from several typical joints will often provide a good visual indication of the extent of deterioration in the vicinity of the joints. These results may also show that partial-depth repairs may be acceptable in certain instances.(55)

d. Joint Lock-up: Corrosion of the dowels or other load transfer devices can eventually lead to nonworking or "frozen" joints. This may be manifested in the following ways:

   A transverse crack can develop across the slab parallel to the joint near the end of the dowels. The area between this crack and the joint often spalls and breaks up, requiring full-depth repair.

   Lock-up of joints from corrosion can also result in the opening of nearby transverse cracks causing the reinforcing steel to rupture in JRCP and resulting in eventual spalling and faulting of the crack. These cracks then act as joints and require full-depth repair.

   Corrosion and lock-up of mechanical load transfer devices can also lead to joint spalling due to expansive pressures or other stresses.

e. Joint inserts: Certain types of joint inserts (e.g., Unitubes) cause spalling of the joint through corrosion, entrapment of incompressibles or other means.

- Slab breakup such as corner breaks or diagonal cracks near the joint: This is caused by a loss of slab support. Faulting of the slab near the joint in the cracked area and fines on the shoulder are definite indicators of pumping. Another early indicator of pumping is the development of a small depression (blowhole) of the asphalt shoulder near the joint or crack where base materials are pumped out.

- Breakup of the slab in several pieces: This is typically caused by repeated heavy truck loads and loss of support from beneath the slab from pumping. Another cause is movement of the foundation from frost heave or swelling soils. If slab breakup is occurring only in the lane with the heaviest truck traffic, fatigue damage is the likely cause, but if slab breakup occurs in all lanes then foundation problems are likely.

The severity of the deterioration of the joint or crack is the main criterion by which the engineer decides if a repair is needed and determines its required size. Comprehensive distress identification manuals are available that include descriptions of joint and crack distress at low, medium and high severity levels.(1)

Low severity level: does not require full-depth repair within the next 2 years.

Medium severity level: may or or may not require repair depending on several factors. Quite often a joint having only medium-severity spalling on the top of the slab is seriously deteriorated at the bottom of the slab. This should be investigated through selective coring near representative joints. The time interval between the preliminary condition survey and actual construction of the repair must be considered. The preliminary survey is conducted for the purpose of making an estimate for bidding purposes. Therefore, if more than 1 year will pass before construction will begin, most of the medium-severity distress and all of the high-severity distress should be programmed for repair. The medium-severity distress is likely to deteriorate into high-severity distress before the construction begins in 1 or more years. Estimated quantities should also be increased by 10-20 percent per year of delay before repair, to allow for the additional deterioration.

High severity level: is a safety hazard and definitely requires repair.

If a typical asphalt concrete overlay of 1 to 6 in [2.5 to 15.2 cm] in thickness is to be placed, it is recommended that there be no difference in the amount or quality of full-depth repair done prior to overlay than would be done if no overlay were placed, because deteriorated joints and cracks will quickly reflect through the overlay and cause premature deterioration and failure of the overlay.

The need for full-depth repair at individual joints can be assessed using the decision chart shown in figure 64. Specific guidelines for repairing individual joints are provided in the section on design.

5.6.3 Limitations and Effectiveness

Full-depth concrete repairs that are properly designed and constructed (particularly with good load transfer at the joints) will provide good long-term performance (e.g., 10 or more years).

Poor load transfer design and poor construction technique has been responsible for much of the faulting and breakup of full-depth repairs. It has also been responsible for the serious deterioration of reflective cracks over repairs in asphalt concrete overlays. The construction of successful full-depth repairs requires high-quality construction quality control, supervision and inspection, particularly in the installation of dowels or other load transfer devices.

5.6.4 Concurrent Work

In addition to full-depth repair, other types of rehabilitation may be required. A general flow chart for determining joint rehabilitation needs is provided in the design guidelines for pressure relief joints (figure 65). Repair of spalls by partial-depth repair is economical when the distress has not penetrated beneath the midpoint of the slab. Deflection tests should be conducted at the joints and corners to determine existing load transfer and the existence of voids. Subsealing of slabs where pumping has eroded the base is essential to prevent rapid slab cracking. Also, the need for subdrainage should be evaluated in wet climates with fine-grained soils and high truck traffic volumes (see the subdrainage recommendations included with the advisory system presented in volume 3 of this report).

Where poor load transfer exists at original contraction joints, consideration should be given to the reestablishment of good load transfer (by using dowels placed in kerfs or other devices) to reduce deflections and stresses. The reduction of

TRANSVERSE JOINT EVALUATION AND REHABILITATION SELECTION
FOR
JOINTED CONCRETE PAVEMENTS
(BASED ON VISUAL INSPECTION OF INDIVIDUAL JOINTS)

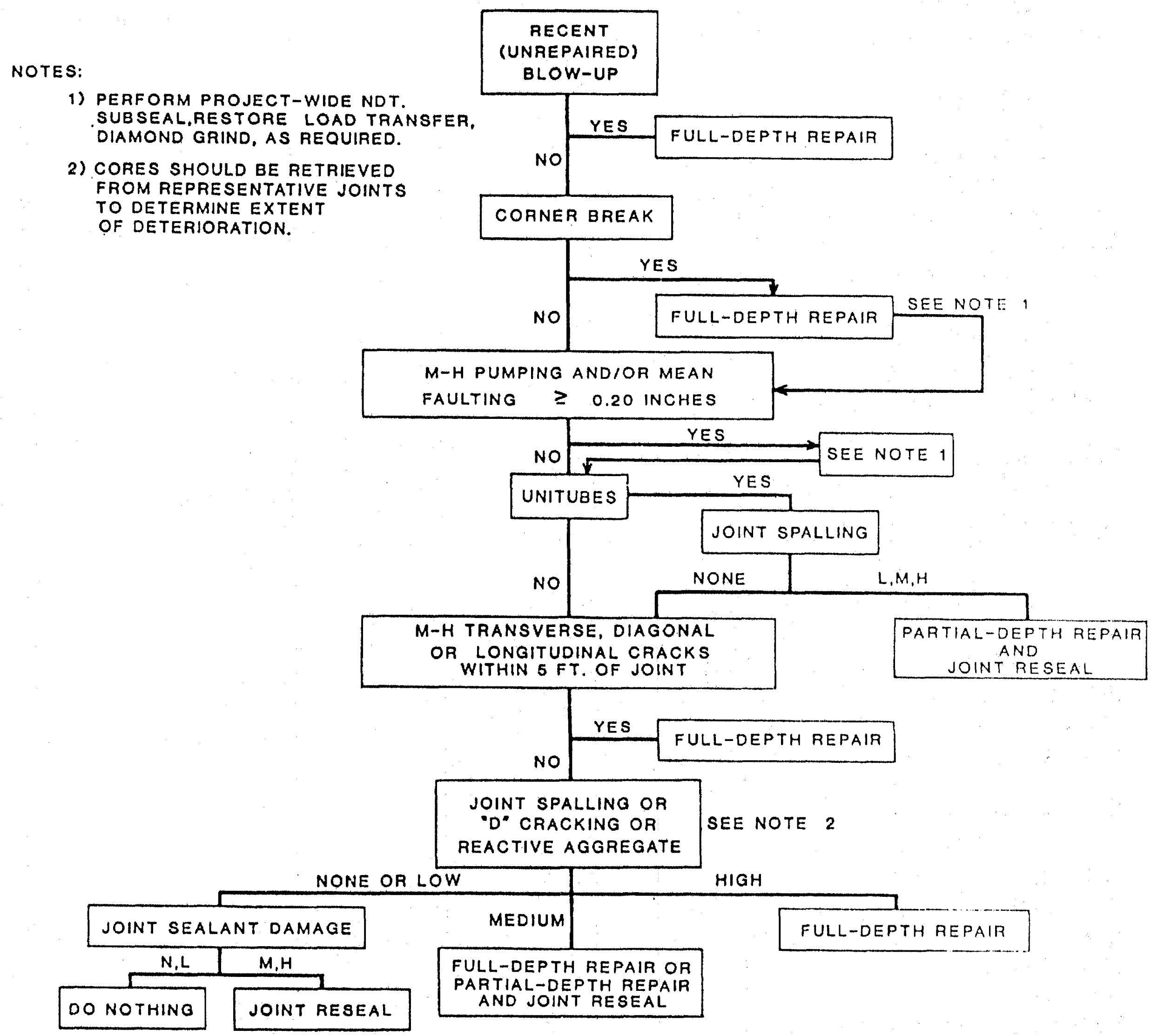


Figure 64. Transverse joint evaluation and rehabilitation selection for jointed concrete pavements.(34)

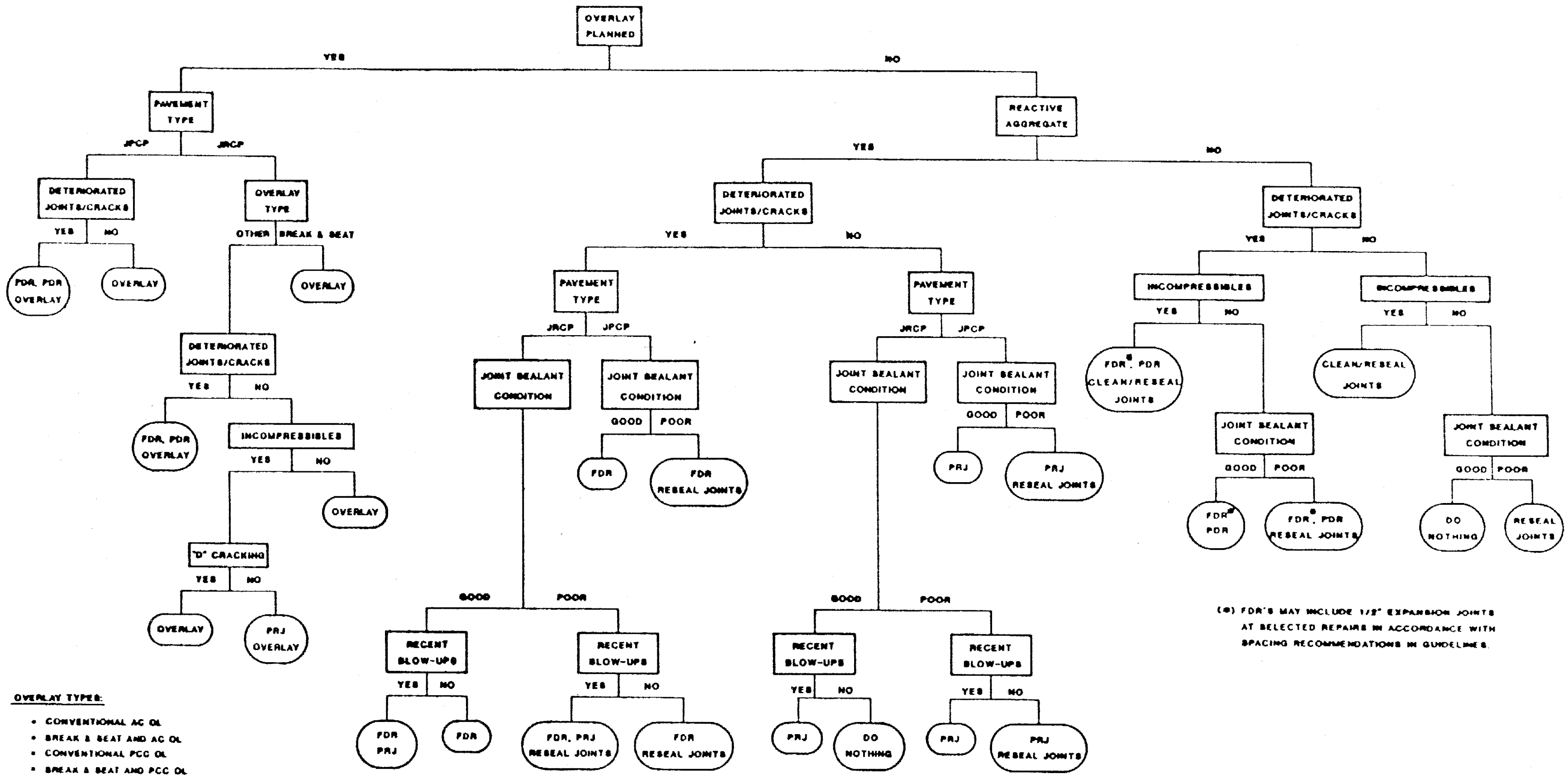


Figure 65. General flow chart for determining joint rehabilitation needs.(34)

free water beneath the slab through joint/crack sealing or the incorporation of underdrains is also very important. If the joint deterioration is due to the infiltration of incompressibles or water into the joint, cleaning and resealing of the transverse joints is necessary.

When a particular JRCP has a history of blowups, construction of pressure relief joints at 1000- to 2000-ft [305 to 610 m] intervals should be considered. However, these joints should be placed not less than 1000 ft [305 m] from the nearest proposed full-depth repair, since the repair itself is a form of pressure relief. Expansion joints should definitely be located at bridge ends, where serious damage to bridge decks and abutments can occur from pavement "growth."

A smooth surface may be restored to the pavement by diamond grinding after most of the above work has been completed.

If an overlay is to be placed, performance of almost all of the same repairs should be considered (except grinding). It is important to realize that medium- to high-severity distress or poor load transfer at joints or cracks that is not repaired will rapidly reflect through the overlay.

5.6.5 Design

General

Full-depth repairs should be designed for specific project conditions. The desired life of the repair and the level of traffic loadings will dictate the design details of the repair. The longer the design life and the greater the truck volumes, the more critical the structural design of the repair becomes. Many full-depth repairs have not performed as desired because the effect of heavy truck traffic was not fully considered in the design of repairs.

Other items to be considered in the design of full-depth repairs are available lane closure time, environmental conditions, subgrade drainability, design of existing pavement, existence of "D" cracking or reactive aggregate in the existing concrete slab and performance history of various repair designs under similar conditions.

Load Transfer

A high degree of load transfer across the transverse joints of the repair is very important in reducing deterioration where heavy truck traffic exists. Poor load transfer causes premature failure of the repair in the form of pumping, faulting, spalling, rocking and breakup. Poor load transfer may be caused by insufficient number or size of dowel bars, poor construction techniques, or a wet climate coupled with poor subbase/subgrade/shoulder drainability. Poor subdrainage greatly increases the potential for pumping, erosion and faulting of the full-depth repair.

Analysis of data from many full-depth repairs in the central U. S. for pavements with poor drainage conditions and granular bases has shown that faulting of full-depth repair joints will, on the average, exceed 0.2 in [0.5 cm] if 100 or more commercial trucks per day use the traffic lane over a 10 year period.(34,73) Transverse joint faulting that exceeds 0.2 in [0.5 cm] is definitely noticeable to drivers. Less precipitation and stabilized bases may allow for much higher truck traffic loadings.

Three different approaches have been used to provide load transfer across full-depth repair joints: (1) aggregate interlock, (2) undercutting and filling with concrete, and (3) dowels and rebars.

Aggregate interlock provides minimal load transfer and is not generally reliable. However, aggregate interlock may be sufficient where low volumes of heavy truck traffic (e.g., less than 100 trucks per day in the traffic lane in a wet climate) are present, a stabilized subbase and/or good subdrainage exist, and/or the repair joints are in compression most of the time due to slab expansions caused by reactive aggregate.

Undercutting alone does not provide adequate load transfer and should not be used in deep frost areas because the existing slab may heave more than the repair, causing severe roughness. Its reliability in nonfrost areas has not been established, but load transfer is often poor due to poor consolidation of concrete in the undercut area, and pumping is often observed in conjunction with such repairs.

The most reliable and recommended method of providing load transfer is to anchor dowels or large tie bars in holes drilled into the face of the slab. (6,8,55,56,57,73)

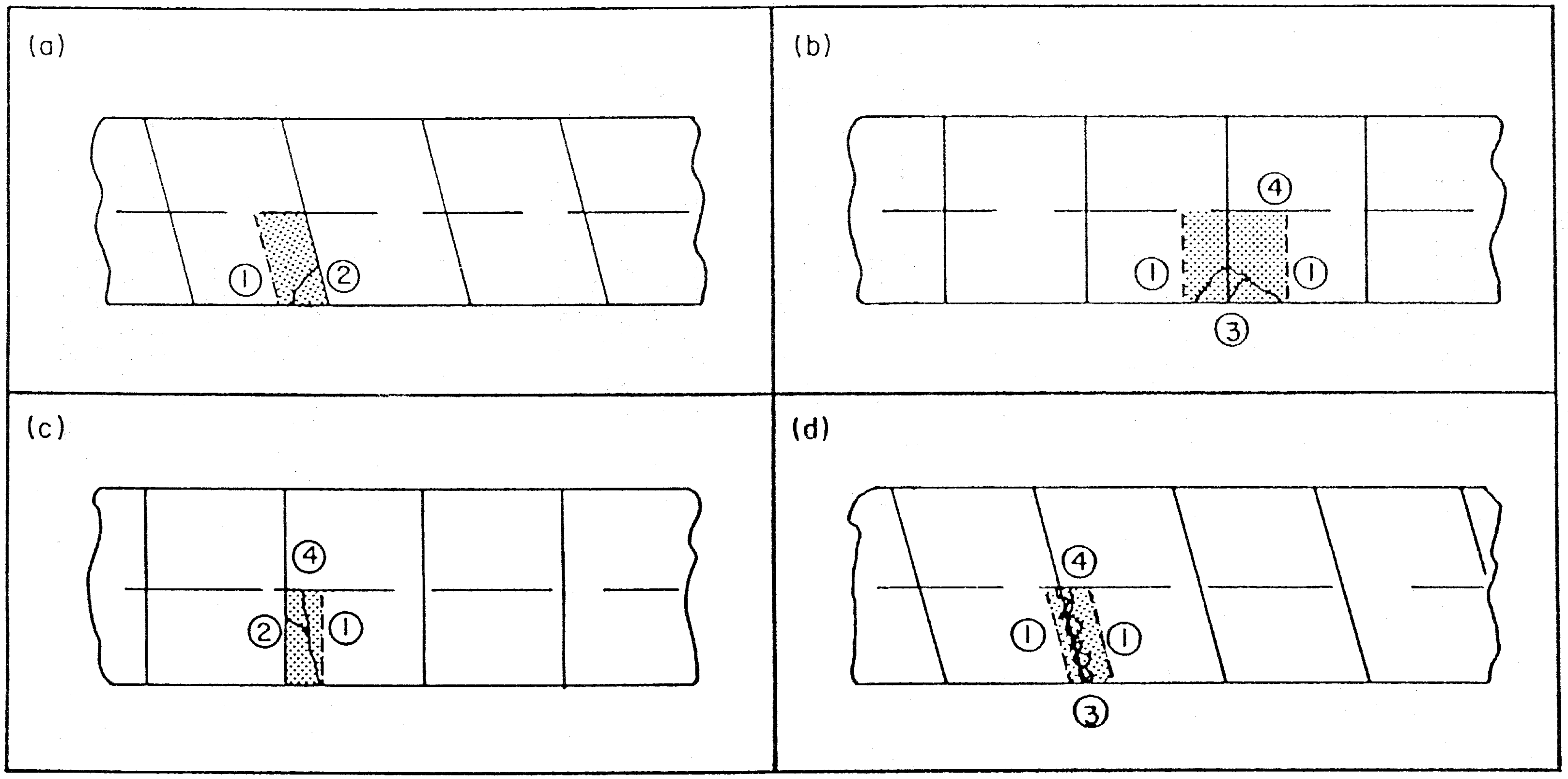
The recommended full-depth repair designs that will provide adequate horizontal movement and load transfer for the indicated situations are shown in figure 66 (for jointed plain concrete pavement) and figure 67 (for jointed reinforced concrete pavement). A detailed layout of the dowels or rebars is shown in figure 68, which shows the load transfer devices located in the wheel paths, where they are needed the most.(6,22)

The number, spacing and diameter of the dowels will determine the amount of future faulting of the transverse joints. An approximate design procedure (prepared using a relationship between joint faulting, equivalent single axle-loads (ESAL) and dowel/concrete bearing stress) is provided in reference 6. The required dowel design determined by this procedure is an iterative process considering the following factors:

- Dowel diameter.
- Number of dowels in each wheel path (spaced at 12 in [30.5 cm]).
- Future ESAL in design lane.
- Allowable faulting of the repair transverse joint.

The major uncertainty in using this procedure is that the relationship was developed from inservice pavement joints featuring cast-in-place dowels that are <u>fully supported by the surrounding concrete</u>. Thus, it is essential that good grouting or epoxying of the dowels is performed to achieve the predicted results.

The use of 1.5-in [3.8 cm]-diameter dowels is recommended in most instances due to the very beneficial effect of reducing faulting for a small increase in cost of the dowel. Recent FHWA research suggests that an acceptable alternate dowel is 1.625-in [4.1 cm] stainless steel pipe (1/8-in [0.3 cm] wall thickness) filled with concrete.



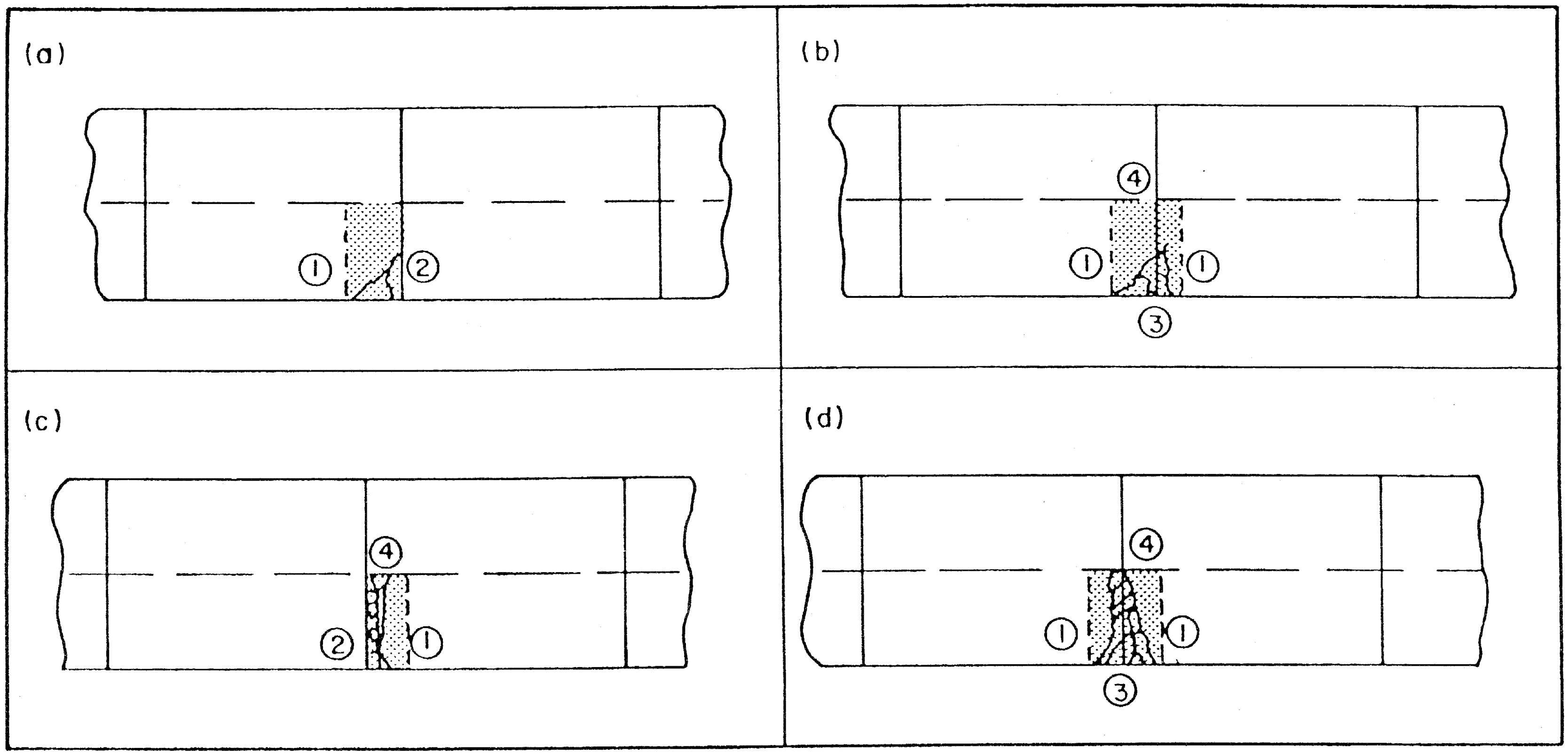
(1) Full Depth Saw Cut With Deformed Tie Bars or Dowels

(2) Existing Joint

(3) Transverse Joint not Required at This Point Unless Tie Bars/Epoxy are Used at (1) Joints, or if Length is > 12 Ft. [1 ft = 0.3048 m]

(4) Sawing of Joint may be Required to Cut Tie Bars and Key-Way. Tie Bars not Required. Bond Breaker Material Must be Placed Along Longitudinal Joint.

Figure 66. Full-depth repair recommendations for plain jointed concrete pavements under heavy traffic.(6)



(1) Full Depth Saw Cut With Smooth Dowel Bars

(2) Butt Type Joint-Dowels Left in Place or as Shown in Figure (b) if Concrete Deteriorated

(3) Transverse Joint not Required at Point Unless Repair > 12 Ft. Long [1 ft = 0.3048 m]

(4) Sawing of Joint May be Required to Cut Tie Bars and Key-Way. Tie Bars not Required. Bond Breaker Material Must be Placed Along Longitudinal Joint.

Figure 67. Full-depth repair recommendations for jointed reinforced concrete pavements under heavy traffic.(6)

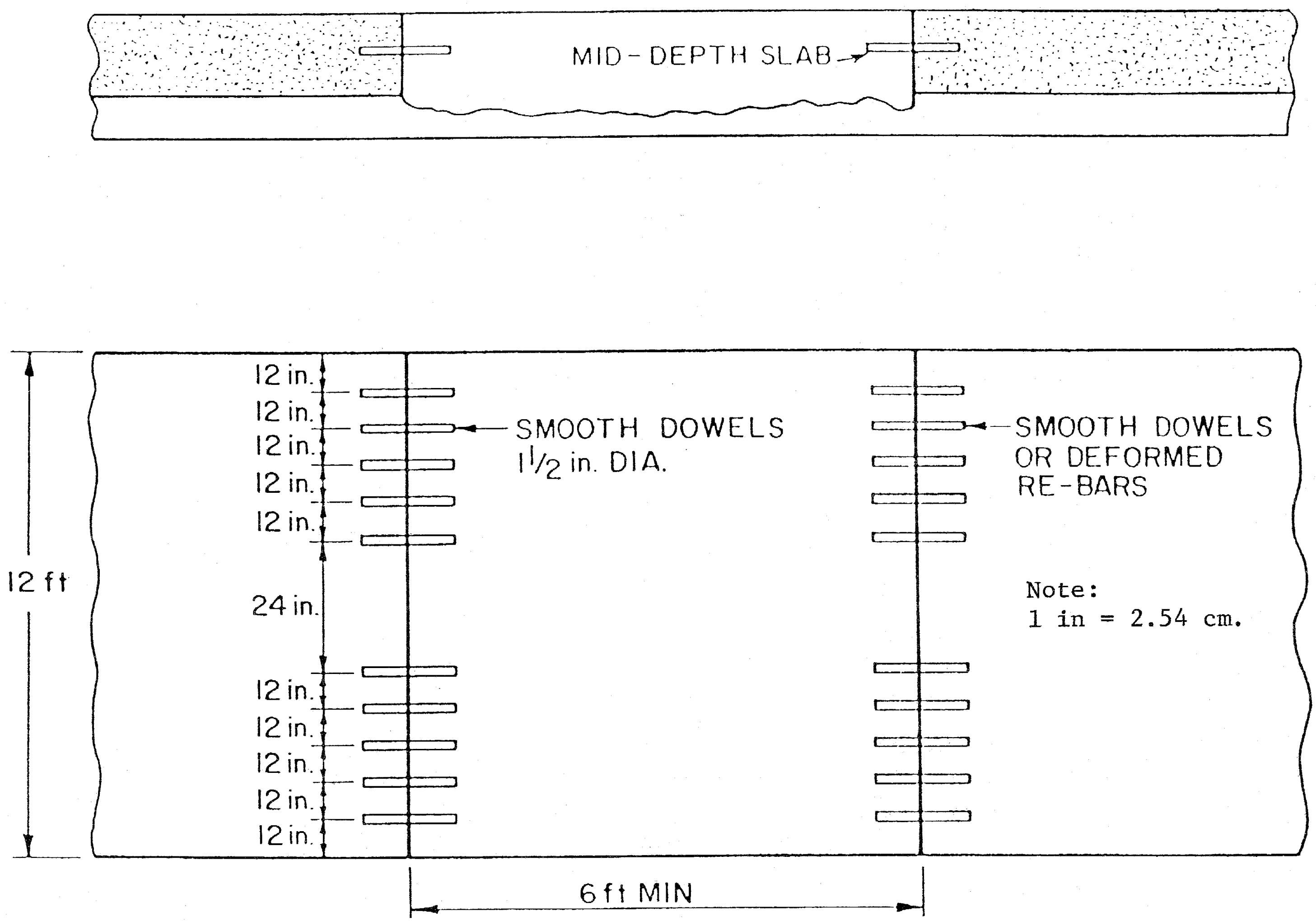


Figure 68. Recommended dowel bar layout for heavy traffic loadings.(6)

The use of smooth dowel bars at both repair joints provides working joints on both sides of the repair and avoids the potential joint damage due to pullout, which is associated with deformed bars. However, in some cases it may be desirable to provide one or more nonworking joints through the use of large deformed rebar. In such cases, the size of the deformed bar can be determined through an analysis similar to that for dowel bars. No. 8 bars (1 in [2.5 cm] diameter) are the smallest size recommended for use in most highway pavements.

Selection of Boundaries

It is important that the boundaries be located so that all significant distress is removed. In general, deterioration near joints and cracks is greater at the bottom of the slab than at the top of the slab. Special attention should be paid to distress caused by "D" cracking or reactive aggregate because of the difficulty in determining their extent beneath the surface of the slab.

The location of repair boundaries also depends on the level of load transfer which is to be provided. The repairs must be of sufficient size to eliminate rocking and longitudinal cracking of the repair. A minimum repair length of 6 ft [1.8 m] and repair width of 12 ft [3.6 m] is recommended to provide stability under heavy traffic (as shown in figure 68) and to prevent longitudinal cracking. In the case of short-jointed plain slabs with high-severity distress, it is normally recommended that the entire slab be replaced.

Repairs longer than 15-ft [4.6 m] may require reinforcement to prevent transverse repair cracking. It may be more economical to place additional dowelled transverse joints at about 15-ft [4.6 m] intervals than to place reinforcement.

Example repair layouts are shown in figure 69 for jointed plain concrete pavements (JPCP) and figure 70 for jointed reinforced concrete pavements (JRCP).

Repair Thickness

The repair should normally be the same thickness as the existing slab, although a thicker repair may be warranted in some circumstances. If truck traffic is very heavy and there has been a history of cracked repairs after a few years, it may be necessary to place the repairs 2 to 4 in [5.1 to 10.2 cm] thicker than the existing slab. Also, if the contractor disturbs the base, the disturbed material should be removed and the volume should be filled with concrete during the repair placement. When the repair is made thicker than the surrounding pavement, care must be taken not ot block drainage, which could result in pumping and/or frost heave problems on adjacent sections.

5.6.6 Construction

Materials

The concrete should be obtained from a nearby approved ready-mix plant or from an on-site mixing plant, and should have the following properties:

- A cement content of 658-846 pounds (7 - 9 sacks) of portland cement type I, II, or III per cubic yard [390-501 kg per cubic meter] of concrete can be used, depending upon the need for rapid strength gain to achieve early opening to traffic. A mix containing approximately 658 pounds per cubic yard [390 kg per cubic meter] is sufficient for most repair work.

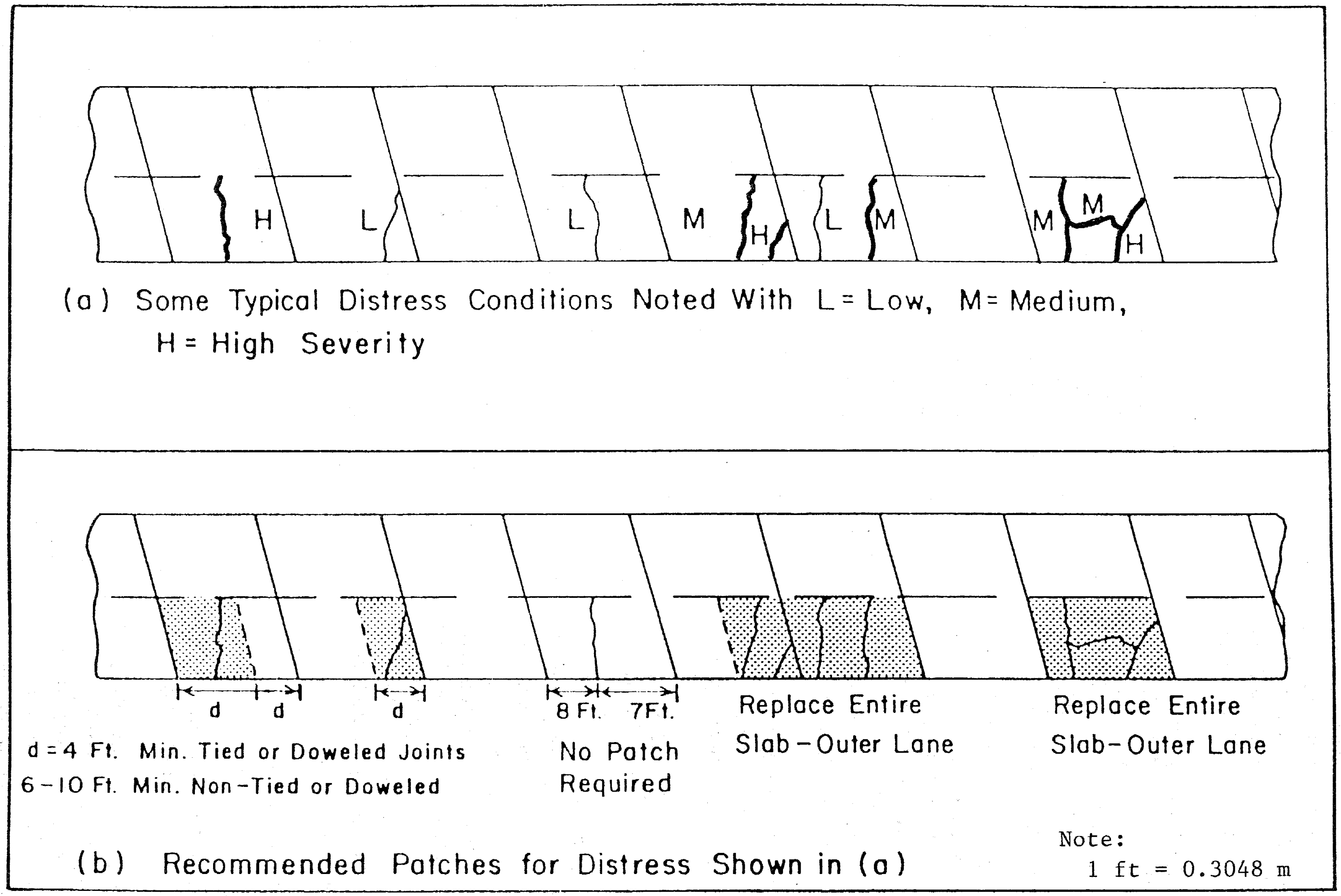
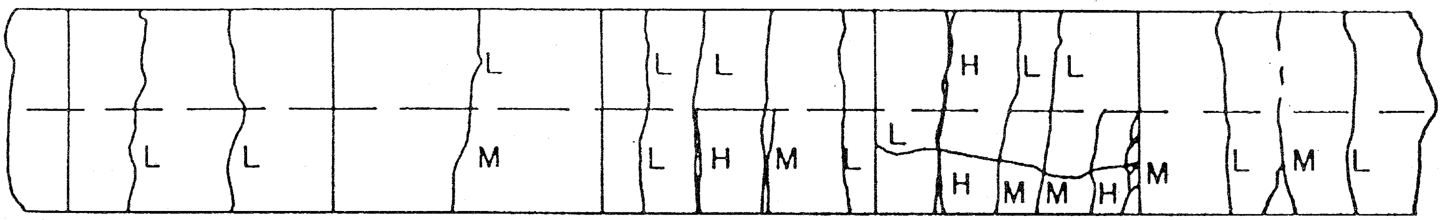
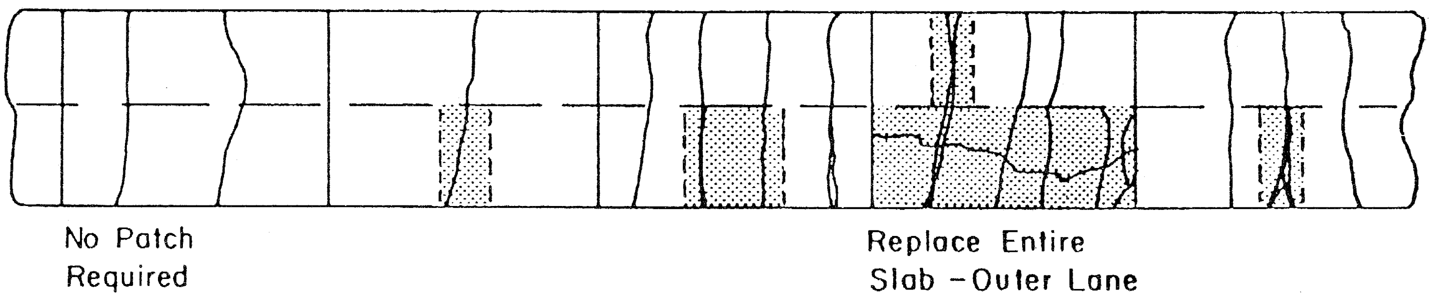


Figure 69. Recommended designs for full-depth repairs for jointed plain concrete pavements.(6)



(a) Some Typical Distress Conditions Noted With L = Low, M = Medium, and H = High Severity (M-H Cracks Have Ruptured Reinforcement)



(b) Recommended Patches for Distress Shown Above in (a)

Figure 70. Recommended designs for full-depth repairs for jointed reinforced concrete pavements.(6)

- An approved air-entraining agent in an amount such that 6.5 $\pm$ 1.5 percent of air is entrained in the concrete.

Calcium chloride or another accelerating chemical admixture is recommended for use as an accelerator in the repair concrete, provided that it is added as specified. It is recommended that no more than 1 percent calcium chloride be used when the ambient temperature is above 80 ^{O}F [27 ^{O}C] because greater amounts can bring on flash set. The maximum percentage is generally limited to 2 percent by weight of cement. On warm days, the initial set of the concrete can occur as soon as 30 minutes after the addition of calcium chloride.

The concrete in the ready-mix truck must be mixed an additional 40 revolutions after the addition of the calcium chloride in solution at the site. Higher early strength can be obtained by the addition of a water reducing agent, or a combination of water reducing and set controlling admixtures, or an approved superplasticizer.

The superplasticizer should be added at the site because of the limited time of its effectiveness. It should be added in accordance with the instructions supplied by the manufacturer to provide a 6-in [15.2 cm] maximum slump concrete.

If both calcium chloride or other accelerating admixtures and superplasticizer are to be added, the calcium chloride should be added before the superplasticizer. The superplasticizer should be added immediately after the calcium chloride has been thoroughly mixed.

If calcium chloride or other accelerating admixtures are being added at the plant and the concrete consistently arrives at the site too stiff, then the calcium chloride should be added at the site. If, after the addition of calcium chloride at the site, the concrete is still too stiff, the ready-mix plant operator should be notified to increase the slump an appropriate amount, provided that the maximum w/c ratio is not exceeded. Concrete containing one or more chemical admixtures may have these added to the concrete at the batch plant, provided short haul to job site and cool temperatures exist.

Trial mixes using all proposed ingredients should be tested in the laboratory prior to use in the field.

Procedures

Sawing of Repair Boundaries

Repair transverse boundaries must be sawed full depth with diamond saw blades. The only exception to this is where a wheel saw (having carbide steel tips) may be used to make wide cuts inside the full-depth diamond saw cuts so that the center portion can be lifted out. The sawcuts must not intrude on the adjacent lane if that lane is not slated for repair. If the wheel saw cut(s) are made, diamond saw cuts must then be made at least 18 in [45.7 cm] outside the wheel saw cuts. The wheel saw cuts produce a ragged edge that promotes excessive spalling along the joint. The wheel saw must not penetrate more than 1/2 in [1.3 cm] into the subbase. The longitudinal joint between lanes should be sawed full depth.

Full-depth sawing creates a smooth joint face with no load transfer capacity, and high deflections will occur if no mechanical load transfer is provided. Thus, it is very important to limit the traffic loadings between the time of sawing and removal. It is recommended that no traffic be allowed over the sawed repairs before removal procedures begin, to avoid pumping and erosion beneath the slab.

Removing Existing Concrete

Removal procedures must not spall or crack adjacent concrete or disturb the base course. This requires the following considerations:

- Heavy drop hammers should not be allowed on the job.
- Hydro hammers (large automated jackhammers) must not be allowed near a sawed joint.
- Whenever the temperature is such that the sawed joint closes up, saw cuts can be made to relieve pressure and spalling when the existing slab is broken up or lifted out. A relief cut pattern that will eliminate spalling is shown in figure 71.

Procedures used for removal must not disturb the subbase or subgrade. The common practice of disturbing and then replacing the subbase does not work well because it is extremely difficult to adequately compact the replaced material. If the contractor disturbs the subbase, he should be required to remove all disturbed material and fill the area with concrete at his own expense when the repair is placed.

There are two basic methods for removing the existing deteriorated concrete within the repair area. These include (1) the breakup-and- cleanout method and (2) the lift-out method. Advantages and disadvantages of each method are summarized in table 27. The lift-out method generally provides the best results and the highest production rates for the same or lower cost, and with the least disturbance of the base, and is the recommended method. Contractors will develop lifting equipment that provides for safe and rapid removal whenever a substantial amount of work is available.

After the existing concrete has been removed, the subbase/subgrade should be examined to determine its condition. All material that has been disturbed or is loose should be removed. If excessive moisture exists in the repair area, it should be removed or dried up before the concrete is placed. Sometimes there is so much water in a given repair area that a lateral side drain must be cut through the shoulder for drainage. The entire foundation should also be compacted before the concrete is placed to minimize the potential of slab settlement.

Dowel and Rebar Placement

Either smooth steel dowels or deformed rebars can be installed in the repair joints. For long-jointed reinforced jointed pavement, it is recommended that smooth dowels be used at both ends to allow free movement (especially if the repair thickness is greater than the existing slab thickness). When deformed rebars are used at one end, they should be placed in the approach joint because this joint tends to become very tight due to the action of truck wheels pushing the repair backwards.

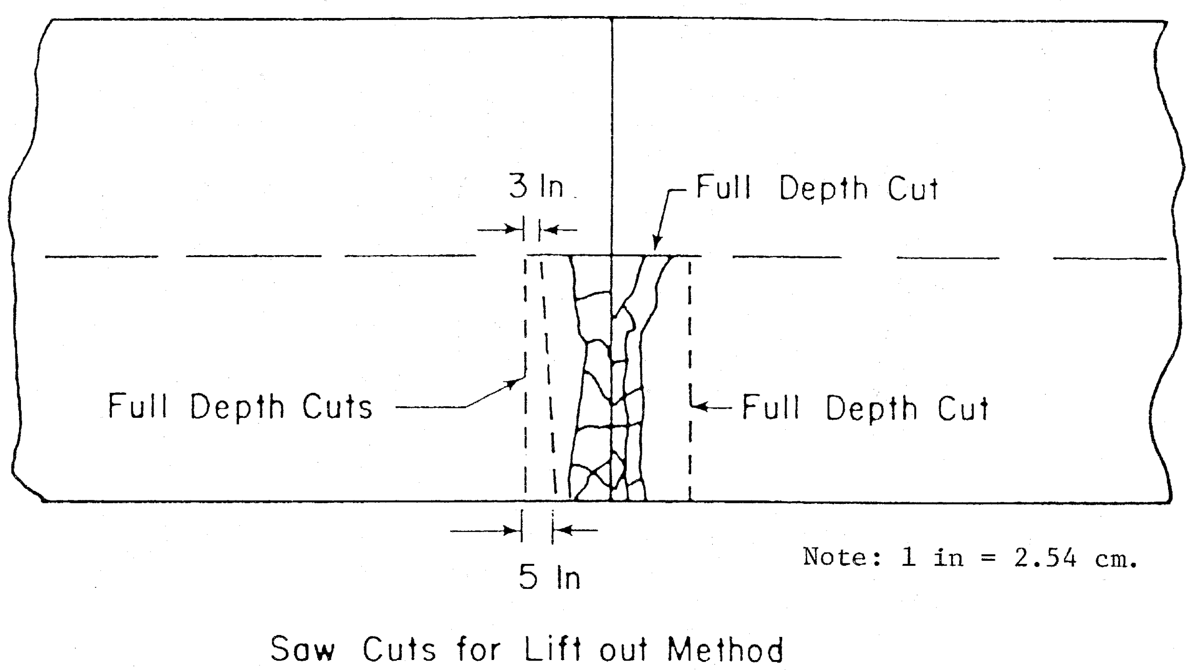


Figure 71. Saw cut locations for lift out method of concrete removal. (6) (Note: Due to equipment lift out limitations, it may be necessary to cut the slab into smaller pieces).

Table 27. Advantages and disadvantages of methods for removal of concrete in patch area.(6)

<u>Method</u>

1. Breakup and Cleanout

    a. Advantages - Pavement breakers can efficiently breakup the concrete and a backhoe having a bucket with teeth can rapidly remove the broken concrete and load it onto trucks.

    b. Disadvantages - This method usually greatly disturbs the subbase/subgrade, requiring either replacement of subbase material or filling with concrete. It also has considerable potential to damage the adjacent slab.

2. Lift-Out

    a. Advantages - This method does not disturb the subbase and does not damage the adjacent slab. It generally permits more rapid removal than the breakup and cleanout method.

    b. Disadvantages - Disposal of large pieces of concrete may pose a problem. Lifting pins and heavy lifting equipment are required for the lift out, or the slab must be sawed into smaller pieces so that they can be lifted out with a front-end loader.

Bar material selection must consider corrosion protection to prevent functional failure of the load transfer system over time due to the action of oxidation and deicing salts. The most common approach is the use of epoxy-coated mild steel bars, although some States believe that the epoxy coating may wear off over time as the slabs expand and contract. Another approach is the use dowels fabricated using noncorrosive materials, such as Type 316 stainless steel pipe and other commercially available products. FHWA tests using 1.625-in [4.1 cm] diameter, 0.125-in [0.3 cm] wall thickness stainless steel pipe yielded satisfactory results when the pipes were filled with concrete to prevent excessive pipe wall deformation.

Installation is accomplished by drilling holes into the exposed face of the slab at specified locations. The holes can be drilled rapidly by placing several drills in a frame that holds them in a horizontal position at the correct height. The dowels must be carefully aligned with the direction of the pavement to provide easy movement.

The nominal diameter of the drilled holes must consider the nominal diameter of the bars and the anchor material to be used. Holes diameters exceeding the bar diameter by 0.25 in [0.6 cm] are recommended for cement grout applications because the dowel will receive better support from a plastic grout mixture (rather than a very fluid mixture) and the larger diameter hole will allow easier insertion of the dowel into the stiffer mixture. Hole diameters exceeding bar diameter by 1/16 in [0.2 cm] or less are recommended for epoxy mortar materials that are premixed and proportioned (e.g., those delivered in "caulking gun" tubes) because they can often be extruded through relatively small gaps, providing uniform support with a minimum use of materials. Since these materials are often more flexible than the supporting concrete, thin layers are desirable to reduce deformation of the epoxy mortar and the accompanying dowel deflection.

The dowels should be located to provide the most benefit. Placing the bars in and near the wheel paths and the outer edges of the slab is believed to be the most effective. This minimizes the number of bars, yet provides load transfer in the wheel paths. Figure 68 suggests a recommended design spacing for bars.

A quick-setting, nonshrinking mortar or epoxy resin can be used to permanently anchor the dowel or rebar in the hole. It is strongly recommended that even smooth dowels be grouted or epoxied into the existing slab to provide a secure fit and reduce potential for faulting. The selected material must uniformly surround the dowel and fill all voids in the drilled hole without running out of the hole during curing. It is extremely important that the material be easy-to-use and be capable of producing consistently good results. While material cost is always a consideration, the prime consideration must be performance. Watery cement grouts are inexpensive and easy-to-use, but rarely achieve acceptable results. Since the success of the entire repair depends largely on the performance of the load transfer system, a high-quality material must be installed using good construction quality control procedures.

The grout or epoxy must be placed into the back of the hole so that when the dowel is inserted it will force the material forward to cover and support the entire dowel. This process requires that the anchor material be sufficiently plastic to be pumped or placed at the back of the hole and extruded forward to fill small voids, but sufficiently stiff to keep from running out of the hole after the dowel has been inserted. Achieving such a grout consistency can be difficult, but is extremely important so that good dowel support is achieved. Plastic or nylon grout retention disks that fit tightly over the dowel and effectively seal the gap around the hole have been used successfully to prevent flowable anchor materials from running out of the hole.

The placement of grout at the back of the dowel hole can be achieved by using a type of flexible funnel with a long nose so that grout can be poured into the funnel end, and it will run by gravity out the nose which is placed in the end of the dowel hole. A grout as flowable as this may not stay in the hole and provide good dowel support, however. Stiffer grouts can be pumped to the back of the dowel holes. Placement of epoxy-type anchor materials can be achieved by requiring the manufacturer to provide a system for mixing, proportioning and placing the material at the back of the hole. At least one manufacturer provides a caulking gun type of arrangement that dispenses the components from cartridges, through a long mixing nozzle, and out into the back of the dowel hole.

The dowel bar should be inserted into the hole with a twisting motion so that the material on the bottom of the hole is forced up and around to cover the entire bar. During insertion of the bar, the grout or epoxy typically runs out the end at the face of the slab and is wasted, and a gap often forms around the dowel at this critical bearing stress point (at the face of the slab). This loss of material can be avoided and a very effective face obtained all the way around the dowel at the entrance to the dowel hole through use of a thin plastic or nylon disk, as mentioned above. This disk may be about 2 in [5.1 cm] larger in diameter than the dowel being used and should be manufactured to fit snugly over the bar and slide up against the face of the slab when the bar is being inserted into the hole. The disk will keep most of the material in the dowel hole and provide an excellent bearing surface at the face of the slab. A high level of inspection and care must be exercised in grouting or epoxying dowel/tie bars to ensure complete coverage of the bars.

When using dowels, the end that extends into the repair area should be <u>lightly</u> greased to provide ease in movement. Thick coats of grease or oil must be avoided because they may result in loose dowel installations.

Load transfer across the longitudinal joint of full-depth repairs is not normally required.

Concrete Placement and Finishing

Critical aspects of concrete placement and finishing include (1) attaining adequate consolidation, (2) avoiding a mix that is either too stiff or has too high a slump, and (3) ensuring a level (flush) finish.

The concrete should be consolidated around the edges of the repair (especially at the corners) and internally. The concrete mixture should have a slump of approximately 2 to 4 in [5.1 to 10.2 cm] at the repair site for best placement. However, this may vary depending on admixtures used and construction conditions. A mix that is too stiff or too fluid could cause serious placement problems. The use of a superplasticizer, as discussed previously, will help in providing a workable mixture. Work crews should not add excessive water to get a highly flowable mix because this will weaken the concrete and cause higher shrinkage.

The repair must be finished level with the existing concrete. This can be accomplished by screeding in a transverse direction (to follow any ruts in existing pavement), a double strike-off of the surface, followed by further transverse finishing with a straight edge.(58) The surface should then be textured similarly to the existing slab surface. Where an overlay will not be placed and diamond grinding will not soon follow, any ruts in the wheel paths caused by studded tires must be incorporated into the surface of the repair.

Joint Sealing

Experience has shown that transverse joints at full-depth repairs must be formed and sealed. This will substantially reduce spalling of the joints and longitudinal repair cracks. A reservoir (dimensions depending on joint sealant specified, climate, and joint spacing) should be either formed or cut in the new concrete. It should be at least 2 in [5.1 cm] deep to avoid point-to-point contact at the top of the slab, thus reducing spalling potential. After cleaning, a backer rod and the sealant should be placed. The width of the joint should be determined as recommended in reference 6 although wider repair joint reservoirs have been shown to reduce the incidence and severity of joint spalling and longitudinal cracking. The longitudinal joint should also be sealed to reduce the potential for spalling and water infiltration.

Smaller sealant reservoir dimensions may be appropriate along repair approach joints where tie bars are used, although the use of the same reservoir design as for the dowelled joints will provide satisfactory performance and may be more expedient to construct.

Figure 72 shows a typical diagram for transverse and longitudinal joints that could be placed in the project plans with appropriate dimensions.

Curing and Opening to Traffic

Ambient temperature at placement and within the next few hours has been found to be the most influential factor in the strength development of concrete repairs.(59,60) The temperature in the repair concrete slab will be higher than ambient or cylinder/beam temperatures. This difference ranges from 10 to 20 ^{O}F [5 to 10 ^{O}C] at 4 hours after placement for noninsulated repairs. If an insulation blanket is placed over the repair, the temperature difference may be as high as 40 to 60 ^{O}F [22 to 33 ^{O}C].

Thus, for rapid curing (particularly in cold weather) it is strongly recommended that insulation blankets be placed over repairs.(60) Polyethylene sheeting should be placed on the concrete surface under the insulation to prevent moisture loss. Wet burlap has also been used as a curing cover.

Water/cement ratio and admixtures also have a significant effect on strength development during the first few hours after placement. The shortest curing time can be obtained by using a combination of calcium chloride, superplasticizer and insulation blankets. Table 28 provides recommendations on early opening of full-depth concrete repairs.

5.6.7 Preparation of Plans and Specifications

It is recommended that when a substantial amount of repair work is needed, aerial photography be used to clearly delineate the repair locations and estimate quantities. The photographs of the roadway can be cut out and mounted on plan sheets where quantities and locations can be identified.

Diagrams of typical repairs and removal procedures should be included.

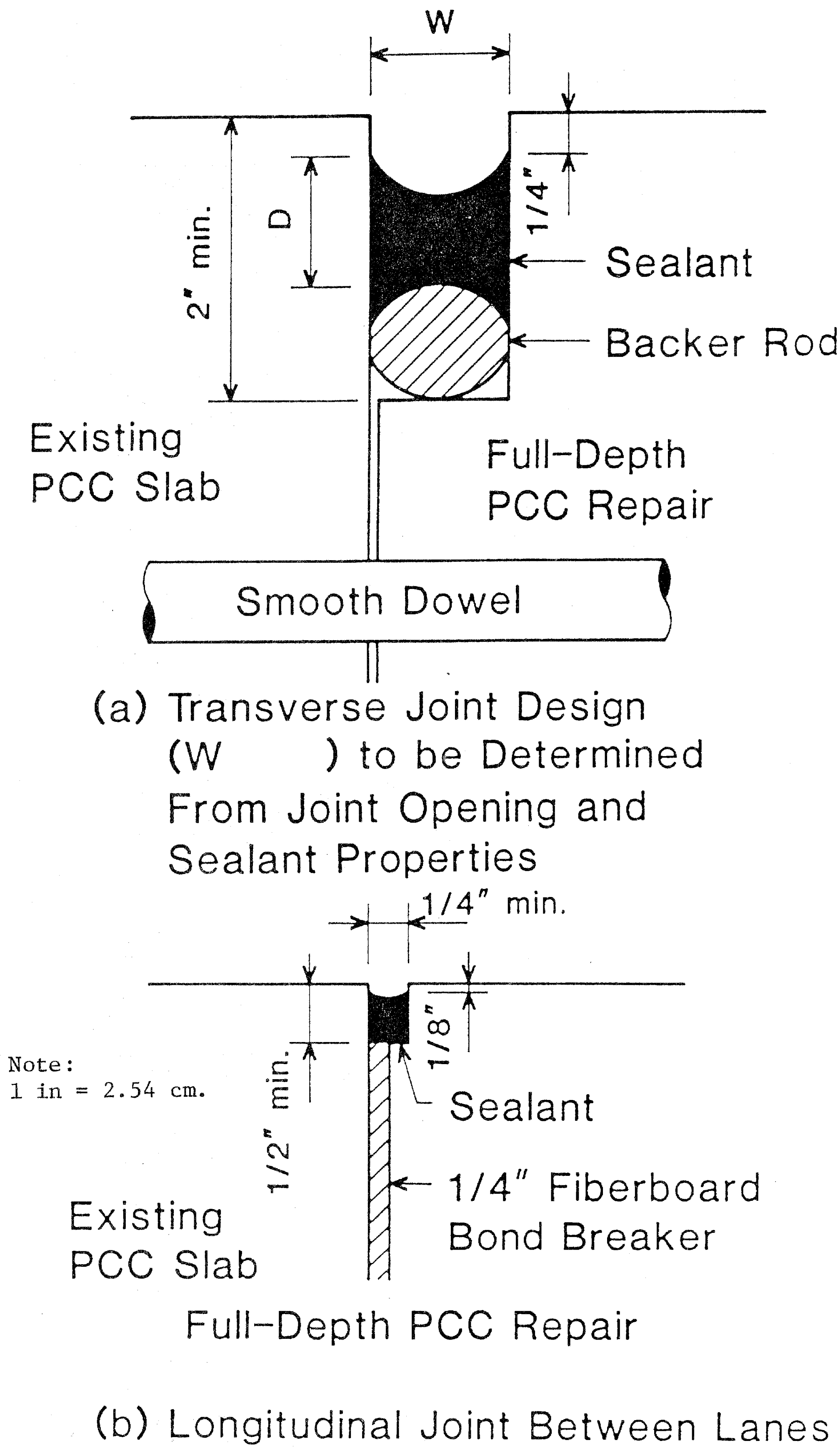


Figure 72. Transverse and longitudinal joint reservoir designs.(34)

Table 28. Early opening guidelines for full-depth repairs [1 in = 2.54 cm; $^{o}C = (^{o}F-32)5/9$].(60)

Slab Thickness (inches)	Ambient Temperature At Placement (^{o}F)	Full-Depth Repair Mixtures/Curing* (hours after placement) A	B	C	D	E	F
7	40	203	90	69	29	28	7
	50	125	60	41	21	20	5
	60	80	45	28	17	16	4
	70	60	38	21	14	13	3
	80	48	35	17	13	11	3
	90	40	30	13	13	9	3
8	40	145	59	55	24	24	6
	50	82	40	35	18	17	5
	60	58	31	24	13	13	4
	70	42	26	17	11	10	3
	80	35	23	13	10	9	3
	90	29	22	11	9	8	3
9	40	82	34	37	15	16	5
	50	51	25	23	12	13	3
	60	28	19	16	9	9	3
	70	25	16	12	8	7	3
	80	20	14	10	6	6	3
	90	17	12	8	5	5	3
10	40	45	18	23	9	9	3
	50	30	14	14	7	7	3
	60	20	10	9	5	5	3
	70	15	9	7	4	4	3
	80	12	7	5	4	4	3
	90	9	6	4	3	3	3

*All mixtures contain 650 pounds cement per cubic yard [386 kg per cubic meter] and 2% CaCl.

Mixture Characteristics:	A	B	C	D	E	F
water/cement ratio	0.42	0.42	0.35	0.42	0.35	0.35
cement type	I	I	I	III	I	III
superplasticizer	no	no	yes	no	yes	yes
fiberglass insulation	no	yes	no	yes	yes	yes

Note: These results are based on research done at the University of Illinois, Department of Civil Engineering, using a computer program written in the Microsoft BASIC language. They are intended as guidelines and should only be used after careful evaluation.(60)

Since small repairs generally have higher unit costs than large repairs, better overall bid prices may be solicited by estalishing pay items with consideration of repair size. For example, one agency has three different bid items for full-depth repairs: Type I, less than 5 square yards [4.2 square meters]; Type II, 5-15 square yards [4.2-12.5 square meters]; and Type III, greater than 15 square yards [12.5 square meters].

Either of two different methods may be used to specify when repairs can be opened to traffic.

- Specified minimum strength of beams or cylinders. The minimum required strength before a repair can be opened to traffic has not been fully established, and it varies widely among agencies. A modulus of rupture of 300 psi [2.1 MPa] for center-point loading, or 250 psi [1.7 MPa] for third-point loading, or 1000 to 2000 psi [6.9 to 13.8 MPa] for compressive strength of specimens cured similarly to the repair are fairly common specifications for opening to traffic.(6,58,59) The actual strength of the repair will be higher than the beams or cylinders because the temperature in the repair will be higher than that in the beam or cylinder.

  Several impact hammers are also available for determining the approximate in-place compressive strength of the full-depth repairs. They have been found to be accurate within 15 percent and provide quick readings in the field. However, they must be calibrated with cylinders and it is important that, once correlated, their testing be performed only on repairs with the same mix design as the cylinders. One such test method is described in detail in ASTM C805.

- Specified minimum time to opening. The agency may specify the mixture design and curing procedures, and then based on ambient temperature at placement and slab thickness, set the minimum time to opening to traffic. The recommendations provided in table 28 are based on analytical and field tests.(59,60) These recommendations should be carefully evaluated by an agency before adoption, and adjusted to local conditions where needed.

5.7 CONCLUSIONS AND RECOMMENDATIONS

5.7.1 Conclusions From Field Data Analysis

The following conclusions and observations were drawn from the analysis of the field data results and are supported in section 5.3 of this volume:

1. Transverse joint faulting is often "built-in" to full-depth repairs due to over- and underfilling of the repair.

2. Repair leave joint faulting is often much greater than repair approach joint faulting. This may be due to the rocking of the repair under passing wheel loads, which allows the moisture under the repair to continue moving forward beyond <u>both</u> of the repair joints. It is then ejected backwards from beneath the leave slab, depositing eroded materials under the repair leave joint and producing a fault. Since little of the moisture is ejected to deposit material under the approach slab, little faulting develops at the repair approach joint.

Additionally, repair approach joints were generally tighter than leave joints, possibly due to movement of the repair in the opposite direction of traffic under the turning action of passing wheels. Tighter joints typically exhibit better load transfer and are more resistent to faulting. Where the development of faulting occurs at both repair joints at equal rates and the repair is originally overfilled, the development of faulting serves to increase the built-in fault at the leave joint and decrease the negative built-in fault at the approach joint. In either case, the net effect is a larger leave joint fault than approach joint fault.

3. The following factors were determined to significantly affect the development of full-depth repair faulting:

Factor	Effect on Faulting
Strong Pavement Support Layers	Decrease
Pavement Drainage Systems	Decrease
Tied Concrete Shoulders	Decrease
Long Original Pavement Contraction Joint Spacing	Increase
Long Repair Length	Decrease
Sealed Repair Joints	Decrease
Cold Climates	Increase
Larger Dowel Diameter	Decrease
More Dowels per Wheel Path	Decrease

4. The most important factor in reducing the development of full-depth repair spalling is the installation of joint seals as soon after repair construction as possible. The use of wider repair joints also reduced the development of spalling. Heavy traffic resulted in increased incidences of spalling, presumably due to increased numbers and magnitudes of vertical joint movements. Other less important factors are listed below:

Factor	Effect on Spalling
Strong Pavement Support Layers	Decrease
Tied Concrete Shoulders	Decrease
Long Original Pavement Contraction Joint Spacing	Increase
Long Repair Length	Decrease
Unsound Aggregates	Increase

5. Only a small proportion of the surveyed repairs exhibited transverse cracks, but some significant relationships were observed:

Factor	Effect on Transverse Repair Cracking
Strong Pavement Support Layers	Increase
Long Original Slab Length	Decrease
Long Repair Length	Increase
Increased Heavy Truck Traffic	Increase

Most of these factors appear to be related to the curling and warping stresses that the repair experiences. Stronger support layers and longer repairs increase these stresses, which can become critical when combined with traffic stresses, resulting in fatigue cracking.

6. An even smaller proportion of the surveyed repairs exhibited longitudinal cracks, but some significant relationships were observed:

Factor	Effect on Longitudinal Repair Cracking
Good Joint Sealant Maintenance Practices	Decrease
Long Repair Length	Decrease
Strong Pavement Support Layers	Decrease
Increased Heavy Truck Traffic	No Real Effect
Undercut, Tied or Dowelled Repairs	Decrease

These factors point to nonload-related causes and imply that the key to reducing this distress is to prevent incompressible materials from entering the repair joints from the surface (through good joint sealant maintenance practices) and from beneath (through the use of stabilized materials and reduced pumping action through good load transfer).

5.7.2 Conclusions From Laboratory Data Analysis

The following conclusions and observations were drawn from the analysis of the laboratory experiment results and are supported in section 5.4 of this volume:

1. The epoxy mortar anchoring material was easier to use and produced more consistent results than the cement grout. Dowel deflections and computed "looseness" were lower when cement grout was properly installed (i.e., when no voids were present and uniform support was provided), but the potential of the cement grout was difficult to achieve because the consistency of the grout typically changed rapidly over very short periods of time.

2. The use of larger diameter dowels significantly reduces concrete bearing stresses and dowel deflections and "looseness" when all other factors are held constant.

3. Large annular gaps (radius of drilled hole - radius of dowel) improved the performance of dowels anchored in cement grout, apparently because better distribution of stiff grout could be achieved. Very fluid grouts performed poorly, regardless of the annular gap.

4. Small annular gaps generally improved the performance of dowels anchored in epoxy mortar because thinner supporting layers of epoxy mortar, which was softer than the concrete specimens, deformed less than thick layers. The consistency of the material was such that good support and filling of the voids was achieved regardless of the annular gap.

5. Reducing dowel embedment resulted in very small increases in dowel deflection and "looseness" when epoxy mortar was used. Even smaller increases resulted when good cement grout specimens were tested.

6. <u>The use of nylon or plastic grout retention disks are essential to achieve the potential performance of any anchored dowel installation</u>. The disk should fit the dowel snugly and have a "weep hole" to allow excess anchor material to escape. Excess anchor material should be used with the disks to allow filling of the spalls surrounding the drilled hole behind the disk.

7. The indicated effect of drill impact energy (increasing drill energy improves dowel performance) is the opposite of what was expected. The use of the grout retention disks to fill surface spalls may have masked the increases in deflection and looseness that would be expected to accompany the spalling associated with high-energy drills. Since the higher-energy drills were both mechanically guided and the low-energy drill was essentially hand-held, a somewhat larger hole may have been produced by the low energy drill, requiring more anchor material along the entire length of the dowel and resulting in slightly increased deflections.

8. Close-fitting holes probably offer promise when used with good anchor materials, quality control and grout retention disks. Cement grout and "no grout" applications may experience poor performance due to nonuniform support of the dowel and drill-induced spalling at the joint face. In any case, care must be taken to avoid forcing (hammering) the dowel into place, which may cause tensile failure in the concrete and dowel damage. A straight hole with a constant diameter must be achieved and the drill steel diameter must be checked often to ensure that it has not worn to a diameter less than that of the dowel.

9. The hollow stainless steel dowel performed adequately, although it did not bond with the epoxy mortar that was used. The deflection profile for this dowel fell somewhere between those obtained for 1.5 and 1.0 in [3.8 and 2.5 cm] dowels, all other factors held constant. Concurrent testing by the FHWA has demonstrated the need to fill hollow dowels with concrete or some other stiff material to deformation of the dowel at the joint face.

10. Based on the lab study results, it appears that the following design and construction parameters would provide excellent field performance on primary and Interstate installations:

    - 1.5-in [3.8 cm]-diameter corrosion-resistant solid steel dowels
    - 1.625-in [4.1 cm]-diameter (nominal) guided drills
    - 7-in [17.8 cm] or greater dowel embedment
    - Use rapid-curing, consistent, easy-to-use anchor material (reduce the emphasis on using the cheapest materials when they are difficult to install adequately).
    - Use grout retention disks during curing of the anchor materials.

    Field testing of these recommendations should be accomplished prior to widespread installation.

CHAPTER 6

PARTIAL-DEPTH REPAIR

6.0 RESEARCH APPROACH

Partial-depth repair is the correction of localized surface distress in concrete pavements by removal of deteriorated concrete and replacement with a suitable repair material. Partial-depth repair improves ride quality and may arrest further development of the distress addressed. It also restores a uniform, well defined joint sealant reservoir prior to joint resealing.

Partial-depth repair may have been performed on an experimental basis on some concrete pavements in the United States as early as 1968. However, the oldest CPR projects still in service which included partial-depth repair were performed about 1976. A recent survey conducted by FHWA identified 14 States which use partial-depth repair routinely, 21 which use it occasionally, and 13 which have developed guidelines for its design and/or construction. Recently conducted reviews of partial-depth repair performance on various projects throughout the United States are described in references 6, 34, 73, and 2.

To assess the performance of partial-depth repairs, it was necessary to develop an extensive database containing information on the design, traffic, climate, and condition of pavements on which this technique has been performed. To obtain all of the data items of interest, the following methods were utilized:

- Extensive field surveys were conducted to record distress, measure faulting, subjectively rate ride quality, observe drainage conditions, and document repair condition with photographs.
- Original pavement design, rehabilitation design and construction data were obtained from as-built plans and verbal communication with State DOT personnel.
- Environmental data were obtained from the National Oceanic and Atmospheric Administration.
- Estimates of average daily traffic and percent commercial trucks were obtained from State DOT personnel. FHWA W-4 tables with historical axle-load distribution data by State and pavement classification were used to compute cumulative ESALs since rehabilitation.

Physical test data (e.g., cores, material samples, deflections, etc.) were not collected. Cores through repaired and unrepaired joints on partial-depth repair projects would have provided a great deal of information on the causes and extent of joint deterioration, and the mechanisms of repair failure. In the absence of this type of information, reasonable assumptions about the reasons for placing partial-depth repairs on particular projects, and the reasons for their success or failure, were made on the basis of communication with State DOT personnel, published reports, and observations made during the field surveys.

Since partial-depth repairs are custom-constructed to the size of the deteriorated concrete area repaired, they vary widely in their horizontal dimensions and depths. They are placed at transverse and longitudinal joints, at cracks, adjacent to full-depth repairs, and even at midslab. Materials used for partial-depth repairs range from conventional Portland cement concrete to concretes made with polymers, epoxies, and special cements, to a multitude of proprietary materials. Finally, construction practices and quality control vary significantly from project to project.

Clearly, a great number of variables exist in the design and construction of partial-depth repairs which may influence their performance. Furthermore, so many partial-depth repairs have experienced early failure, due to inappropriate use or improper construction techniques and materials, that the remaining projects which have experienced good performance do not form a good experimental design upon which long-term performance can be analyzed quantitatively. Even among the best-performing projects, none are more than 10 years old, and many have yet to manifest any significant distress by which declining performance and "expected life" might be defined.

For these reasons, partial-depth repair performance is difficult to assess except in subjective terms. Therefore, the approach taken in this study was to make a case-by-case review of partial-depth repair projects in the database, and from this draw some insights into the causes of early failure versus long-term success of partial-depth repairs.

6.1 DATABASE AND DATA COLLECTION

In this study, partial-depth repairs were surveyed on 36 projects in 16 States. The projects are well distributed throughout the major climatic zones of the United States, and cover a wide range of traffic levels. The database includes JRCP and JPCP (with and without dowels), and several joint spacings, slab thicknesses, and pavement ages. The partial-depth repairs surveyed ranged in age from 1 to 9 years when surveyed (in 1985 and 1986). The oldest partial-depth repair projects were located in Virginia, Georgia, Minnesota, and South Dakota.

The projects in the database are those surveyed on which a significant number of partial-depth repairs were performed, either with or without other techniques, as part of conventional concrete pavement restoration (CPR) work. Occasionally a few partial-depth repairs were found on other projects surveyed, but these projects were not included in the database. The projects included in the database represent a majority of the CPR-type partial-depth repair projects in the United States.

The projects were surveyed between June 1985 and July 1986. In 6 States, projects were surveyed for a concurrent study on joint rehabilitation techniques for FHWA.(34) Figure 73 shows the number and locations of the partial-depth repair projects surveyed. As the map shows, the projects are well distributed throughout the climatic zones of the United States.

A detailed description of the field and office data collection procedures used is given in volume IV. A list of the partial-depth projects surveyed, along with concurrent work performed and traffic and age since rehabilitation, is given in table 29. Design data for the projects is given in table 30. Partial-depth repair construction data for the projects is given in table 31.

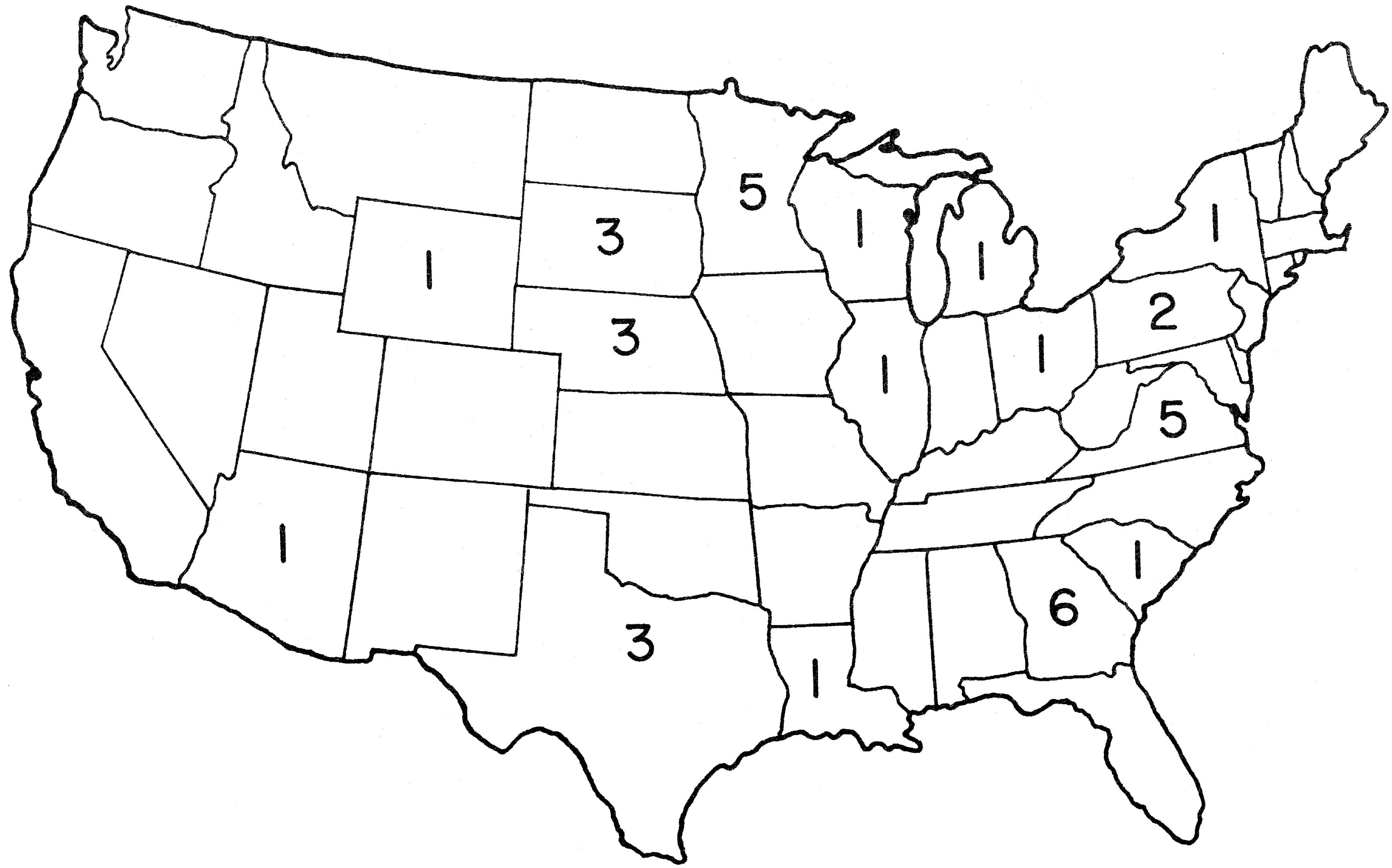


Figure 73. Location of partial-depth repair projects in the database.

Table 29. Summary of partial-depth repair projects.

PARTIAL-DEPTH REPAIR PROJECTS

STATE	ROUTE	LOCATION	MP	YEAR REHAB	PDR AGE years	CUMULATIVE ESAL OUTER	INNER	CONCURRENT WORK
NY	NSP	Northern St. Pkwy.		1980	5	0.06	0.02	none
PA	I-70	east of WV line	0	1984	1	1.19	0.28	FDR,JR
PA	I-70	east of WV line	2	1984	1	1.19	0.28	FDR,JR
VA	I-64	Richmond	202	1976	9	1.98	0.32	FDR,PRJ,JR
				1984	1	0.40	0.06	DRN,FDR
VA	I-64	Williamsburg	239	1984	1	0.35	0.05	FDR,PRJ
VA	I-81	Roanoke	148	1984	1	0.80	0.20	FDR,DG,LTR,SUB,DRN,JR
VA	I-95	Emporia	0	1983	2	1.20	0.25	none
				1984	1	0.60	0.10	FDR,JR
VA	SR 44	Norfolk	0	1976	9	2.50	0.90	JR,PRJ
				1984	1	0.50	0.10	FDR,JR,PRJ
SC	I-20	Augusta, GA	0	1984	2	0.36	0.06	FDR,DG,ES,SUB
GA	I-16	Dublin	39	1982	4	1.07	0.07	FDR,DG,JR
GA	I-75	Valdosta	22	1978	8	4.85	1.24	SUB,PRJ,JR
GA	I-75	Tifton	64	1978	8	3.14	0.82	FDR,SUB,JR,DRN
GA	I-75	Macon	142	1978	8	4.38	0.92	FDR,DG,JR
				1984	2	1.12	0.24	FDR,DG,JR,SUB,DRN
GA	I-75	Macon	165	1980	6	3.25	0.66	DG,JR,SUB
GA	I-85	Atlanta	58	1982	4	5.86	1.98	FDR,DG,JR,SUB
OH	I-77	Cambridge	53	1982	3	2.10	0.50	FDR,SUB,LTR,DG,JR
MI	M-47	Midland		1983	2	0.90	0.18	none
WI	US 61	Boscobel		1981	4	0.54		FDR,DG
MN	I-494	Minneapolis	13	1978	7	3.72	1.29	FDR,JR
MN	I-694	TH 65 to TH 49	39	1981	4	4.59	1.96	FDR,PRJ
MN	TH 23	St.Cloud		1983	2	2.10	0.74	FDR,DG
MN	US 61	St. Paul	119	1979	4	1.59	0.50	none
MN	US 61	Duluth	309	1979	6	0.84	0.13	FDR,JR
SD	I-29	Sioux City	0	1979	6	2.23	0.29	FDR,PRJ,JR
SD	I-29	Junction City	27	1979	6	1.48	0.16	FDR,PRJ,JR
SD	I-90	Chamberlain	265	1982	2	0.83	0.06	FDR,PRJ,DG,JR
IL	I-280	Moline	14	1984		0.50	0.25	FDR,DG,SUB,DRN,JR
NE	I-80	Kearney	279	1982	3	4.00	0.90	FDR,PRJ,JR
NE	I-80	Lincoln	382	1982	3	5.00	1.30	FDR,PRJ,JR
NE	I-80	Lincoln	404	1984	1	1.20	0.40	FDR,PRJ,JR
LA	I-10	Baton Rouge	151	1984	1	1.73	0.50	FDR,DG,LTR,SUB,JR,DRN
TX	I-40	Houston	731	1984	2	4.04	1.22	FDR
TX	I-40	Houston	741	1982	4	6.50	4.24	FDR
TX	US 59	Houston		1983	3	5.88	5.18	FDR
WY	I-80	Rawlins	210	1982	4	2.38	0.24	FDR,DG,SUB,JR
AZ	I-17	Phoenix	199	1976	5	4.50	3.50	DG

Table 30. Original pavement design data for partial-depth repair projects.

PARTIAL-DEPTH REPAIR PROJECTS

STATE	ROUTE	LOCATION	MP	YEAR REHAB	YEAR BUILT	PVT TYPE	THICK inches	JTSPACE feet	DOW DIA inches	UNITUBES
NY	NSP	Northern St. Pkwy.		1980	1942	JRCP	8	100	1.25	
PA	I-70	east of WV line	0	1984	1968	JRCP	10	61.5	1.25	
PA	I-70	east of WV line	2	1984	1963	JRCP	10	61.5	1.25	
VA	I-64	Richmond	202	1976 1984	1966	JRCP	9	61.5	1.25	yes
VA	I-64	Williamsburg	239	1984	1965	JRCP	9	61.5	1.25	yes
VA	I-81	Roanoke	148	1984	1964	JRCP	9	61.5	1.25	yes
VA	I-95	Emporia	0	1983 1984	1963	JPCP	9	20	1.25	yes
VA	SR 44	Norfolk	0	1976 1984	1967	JRCP	9	61.5	1.25	yes
SC	I-20	Augusta, GA	0	1984	1967	JPCP	9	25		
GA	I-16	Dublin	39	1982	1968	JPCP	10	30		yes
GA	I-75	Valdosta	22	1978	1961	JPCP	9	30		
GA	I-75	Tifton	64	1978	1961	JPCP	9	30		
GA	I-75	Macon	142	1978 1984	1966	JPCP	10	30		
GA	I-75	Macon	165	1980	1967	JPCP	10	30		
GA	I-85	Atlanta	58	1982	1968	JPCP	9	30		
OH	I-77	Cambridge	53	1982	1967	JRCP	9	60		
MI	M-47	Midland		1983	1966	JRCP	9	71	1.25	
WI	US 61	Boscobel		1981	1953	JPCP	8	20		
MN	I-494	Minneapolis	13	1978	1963	JRCP	9	40	1.25	
MN	I-694	TH 65 to TH 49	39	1981	1964	JRCP	9	40	1.25	
MN	TH 23	St.Cloud		1983	1964	JRCP	9	80	1.25	
MN	US 61	St. Paul	119	1979	1958	JRCP	9	40	1.00	
MN	US 61	Duluth	309	1979	1967	JRCP	8	40	1.00	
SD	I-29	Sioux City	0	1979	1961	JRCP	9	61.5	1.25	
SD	I-29	Junction City	27	1979	1961	JRCP	9	45	1.25	
SD	I-90	Chamberlain	265	1982	1965	JRCP	9	45	1.25	
IL	I-280	Moline	14	1984	1961	JRCP	10	100	1.25	
NE	I-80	Kearney	279	1982	1962	JRCP	9	46.5		
NE	I-80	Lincoln	382	1982	1962	JRCP	9	46.5		
NE	I-80	Lincoln	404	1984	1960	JPCP	10	16.3		
LA	I-10	Baton Rouge	151	1984	1971	JRCP	10	58.5		
TX	I-40	Houston	731	1984	1967	JRCP	10	60.5	1.25	
TX	I-40	Houston	741	1982	1966	JRCP	10	61.5	1.25	
TX	US 59	Houston		1983	1961	JRCP	10	61	1.25	
WY	I-80	Rawlins	210	1982	1964	JPCP	8	20		
AZ	I-17	Phoenix	199	1976	1961	JPCP	9	15		

Note: 1 in = 2.54 cm, 1 ft = 0.3048 m

Table 31. Partial-depth repair construction data.

PARTIAL-DEPTH REPAIR PROJECTS

STATE	ROUTE	MP	REPAIR LIMITS	CUTTING EQUIP	REPAIR DEPTH	REMOVAL METHOD	CLEAN METHOD	BOND AGENT	REPAIR MATL	JT FORM METHOD	CURE METHOD	CURE TIME hours
NY	NSP		sound	dia saw	3.5	airhammer	sand	none	Set 45	forms	none	2
PA	I-70	0	sound	dia saw	1.5	airhammer	sand	epoxy	pcc	styrofm	membrane	6
PA	I-70	2	sound	dia saw	1.5	airhammer	sand	epoxy	pcc	styrofm	membrane	6
VA	I-64	202	visual	dia saw	2	airhammer	air	ha cem*	hac*	saw	membrane	6
			visual	dia saw	2	airhammer	air	cement	pcc	saw	membrane	6
VA	I-64	239	visual	dia saw	3	airhammer	air	cement	pcc	saw	membrane	6
VA	I-81	148	visual	dia saw	2	airhammer	air	cement	pcc		membrane	6
VA	I-95	0	visual	dia saw	3	airhammer	air	cement	pcc		membrane	6
			visual	dia saw	3	airhammer	air	cement	pcc		membrane	6
VA	SR 44	0	visual	dia saw	2	airhammer	air	ha cem*	hac*	polyeth	membrane	6
			visual	dia saw	2	airhammer	air	cement	pcc	polyeth	membrane	6
SC	I-20	0	visual	dia saw	2	airhammer	sand	epoxy	pcc	styrofm	membrane	12
GA	I-16	39	sound	dia saw	1	airhammer	sand	epoxy	pcc	styrofm	membrane	
GA	I-75	22	visual	dia saw	2	airhammer	sand	none	propr*	saw	membrane	24
GA	I-75	64	visual	dia saw	2	airhammer	sand	none	propr*	saw	membrane	24
GA	I-75	142	sound	dia saw	4	airhammer	sand	epoxy	pcc	polyeth	burlap	24
			sound	dia saw	4	airhammer	sand	epoxy	pcc	polyeth	burlap	24
GA	I-75	165	sound	dia saw	4	airhammer	sand	epoxy	pcc	polyeth	burlap	24
GA	I-85	58	sound	dia saw	4	airhammer	sand	epoxy	pcc	polyeth	burlap	24
OH	I-77	53						cem,Sil*	pcc,poly*			
MI	M-47		visual	dia saw	1.5	airhammer	air	none	Set 45	styrofm	membrane	4
WI	US 61		visual	dia saw	2	airhammer	water	Acryl 60	pcc	forms	membrane	8
MN	I-494	13	visual	dia saw		airhammer	sand	cement	pcc	plastic	membrane	5
MN	I-694	39	visual	dia saw	1	airhammer	sand	cement	pcc	plastic	membrane	7
MN	TH 23		visual	dia saw	2		water		pcc	forms	membrane	8
MN	US 61	119	visual	cold mill	2	cold mill	sand	cement	pcc	fiberbrd	membrane	24
MN	US 61	309	visual	cold mill	2	cold mill	sand	cement	pcc		membrane	24
SD	I-29	0	sound	dia saw	2.5	airhammer	sand	epoxy	epoxy	fiberbrd	membrane	72
SD	I-29	27	sound	dia saw	1	airhammer	sand	epoxy	epoxy	fiberbrd	membrane	72
SD	I-90	265	sound	dia saw	3	airhammer	sand	epoxy	epoxy	fiberbrd	membrane	72
IL	I-280	14	visual	dia saw		airhammer	sand	epoxy	pcc		membrane	48
NE	I-80	279	visual	dia saw	2	airhammer		epoxy	epoxy			4
NE	I-80	382	visual	dia saw	2	airhammer		epoxy	epoxy			4
NE	I-80	404	visual	dia saw	2	airhammer		epoxy	epoxy			4
LA	I-10	151	visual	dia saw	4	airhammer	brush	cement	pcc	forms	none	24
TX	I-40	731	visual	dia saw	1	airhammer	sand	cement	pcc	forms	membrane	6
TX	I-40	741	visual	dia saw	1.5	airhammer	sand	cement	pcc	forms	membrane	6
TX	US 59		visual	dia saw	1	airhammer	sand	cement	pcc	polyeth		6
WY	I-80	210	sound	dia saw	2	airhammer	sand	epoxy	pcc	fiberbrd	membrane	24
AZ	I-17	199	sound	airhammer	2	airhammer	air	epoxy	epoxy	fiberbrd	membrane	

6.2 FIELD PERFORMANCE AND EVALUATION

This section presents a brief description of the performance of partial-depth repair projects in the database.

NY Northern State Parkway

8-in [20.3 cm], 100-ft [30.5 m] JRCP built in 1942. Partial-depth repairs placed in 1980 with Set 45. Performing well after 5 years, 0.06 million ESALs.

PA I-70 East of WV Line

10-in [25.4 cm], 61.5-ft [18.7 m] JRCP sections built in 1963 and 1968. Partial-depth repair (Type III PCC), full-depth repair, and joint resealing performed in 1984. Partial-depth repairs performing poorly after 1 year, 1.2 million ESAL. Greater number of partial-depth repairs, along with greater incidence of partial-depth repair deterioration, pumping, and poor joint sealant condition, observed on newer pavement section.

VA I-64 Richmond

9-in [22.9 cm], 61.5-ft [18.7 m] JRCP built in 1966. Unitubes used to form joints. Partial-depth repair performed in 1976 (calcium aluminate cement concrete), along with full-depth repair, pressure relief, and joint resealing. Additional partial-depth repair performed in 1984 (Type III PCC), along with full-depth repair and subdrainage improvement. Some 1976 repairs exhibit scaling and material loss, but replacement is not warranted. 1984 repairs are in good condition, no scaling or material loss.

VA I-64 Williamsburg

9-in [22.9 cm], 61.5-ft [18.7 m] JRCP built in 1965. Unitubes used to form joints. Partial-depth repair (Type III PCC), full-depth repair, and pressure relief performed in 1984. Almost all joints have full-depth repairs, many of which are narrow (e.g., 2 ft by 12 ft [0.6 m by 3.7 m]); many of these have small partial-depth repairs adjacent to them. Some cracking and spalling observed at narrow full-depth repairs and adjacent partial-depth repairs. Wider full-depth repairs (6 ft by 12 ft [1.8 m by 3.7 m]) and full-lane-width partial-depth repairs (2 ft by 12 ft [0.6 m by 3.7 m]) are performing well.

VA I-81 Roanoke

9-in [22.9 cm], 61.5-ft [18.7 m] JRCP built in 1965. Unitubes used to form joints. Surveyed partial-depth repair (Type III PCC) performed in 1984, along with full-depth repair, grinding, load transfer restoration, subdrainage improvement, subsealing, and joint resealing. Partial-depth repairs are in good condition after 1 year, 0.8 million ESALs.

VA I-95 Emporia

9-in [22.9 cm], 20-ft [6.1 m] dowelled JPCP built in 1963. Unitubes used to form joints. Partial-depth repairs placed in 1983 and 1984 with Type III PCC. Full-depth repair and joint resealing also performed in 1984. Nearly all joints in outer lane and 40 percent of joints in inner lane repaired. Both the 1983 (1.2 million ESALs) and 1984 (0.6 million ESALs) partial-depth repairs are in good condition. Full-depth repairs are not performing well.

VA SR 44 Norfolk

9-in [22.9 cm], 61.5-ft [18.7 m] JRCP built in 1967. Unitubes used to form joints. Partial-depth repairs placed in 1976 (calcium aluminate cement concrete) and 1984 (Type III PCC). Pressure relief and joint resealing also performed in 1976; full-depth repair, pressure relief, and joint resealing also performed in 1984.

Scaling, material loss, and adjacent slab spalling observed at 20 percent of 1976 partial-depth repairs. 1984 partial-depth repairs are in good condition.

SC I-20 East of GA Line

9-in [22.9 cm], 25-ft [7.6 m] undowelled JPCP built in 1967. Partial-depth repairs placed in 1984 with Type III PCC. Subsealing (outer lane only), retrofit PCC shoulders, diamond grinding, and joint resealing also performed. Many more partial-depth repairs in inner lane (35 percent of transverse joints) than in outer lane (5 percent of transverse joints). Most inner lane repairs are 1 ft by 4 ft [0.3 m by 1.2 m], and most are located at intersection of transverse joint and longitudinal centerline joint. Repairs in outer lane are smaller (1 sq ft [0.3 sq m]) and located mostly at outer slab corners. All repairs are in excellent condition.

GA I-16 Dublin

10-in [25.4 cm], 30-ft [9.1 m] undowelled JPCP built in 1961. Partial-depth repairs, full-depth repairs, grinding and joint resealing performed in 1982. Joint spalling caused by Unitube inserts. Partial-depth repairs (Type III PCC with calcium chloride accelerator, bonded with epoxy) placed at 85 percent of inner lane joints and 40 percent of outer lane joints between mileposts 39 and 51, fewer repairs (55 percent and 30 percent in inner and outer lanes respectively) between mileposts 51 and 67. Typically one saw cut made across full lane width 1 ft [0.3 m] from transverse joint, and partial-depth repairs placed in all or part of this area. Roughly one third of repairs are full lane width; others are 1 ft by 4 ft [0.3 m by 1.2 m] to 1 sq ft [0.3 sq m]. All repairs are in excellent condition after 4 years, 1.07 million ESALs.

GA I-75 Valdosta

9-in [22.9 cm], 30-ft [9.1 m] undowelled JPCP built in 1961. Partial-depth repair, subsealing, pressure relief, and joint resealing performed in 1978. Partial-depth repairs (proprietary material, type unknown) placed at 30 percent of inner lane joints and 7 percent of outer lane joints. Repairs are in very good condition after 8 years, 4.85 million ESALs. One corner repair failed (30 percent material loss). Low-severity spalling at several joints suggests need for additional partial-depth repair.

GA I-75 Tifton

9-in [22.9 cm], 30-ft [9.1 m] undowelled JPCP built in 1961. Partial-depth repair, full-depth repair, subsealing, subdrainage improvement, and joint resealing performed in 1978. Partial-depth repairs (proprietary material, type unknown) placed at 15 percent of inner lane joints, none in outer lane. Repairs are in excellent condition after 8 years, 3.14 million ESALs.

GA I-75 Macon, Milepost 142

10-in [25.4 cm], 30-ft [9.1 m] undowelled JPCP built in 1966. Partial-depth repair, full-depth repair, grinding and joint resealing performed in 1978, same techniques plus subsealing and subdrainage improvement performed in 1984. Partial-depth repairs (PCC bonded with epoxy) at 43 percent of inner lane joints, 18 percent of outer lane joints. Repairs are in very good condition overall; cracking and material loss observed on 2 repairs. Low-severity longitudinal joint spalling suggests need for additional partial-depth repair.

GA I-75 Macon, Milepost 165

10-in [25.4 cm], 30-ft [9.1 m] undowelled JPCP built in 1967. Partial-depth repair, subsealing, grinding and joint resealing performed in 1980. Partial-depth repairs (PCC bonded with epoxy) at 90 percent of inner lane joints, 33 percent of outer lane joints. Repairs are in very good condition overall; low-severity longitudinal cracking observed on one full-lane-width repair. Low-severity longitudinal cracking of unknown cause observed at 10 percent of outer lane transverse joints. Transverse joint sealant is in poor condition (adhesive failure), and typically sealant is absent in vicinity of partial-depth repairs.

GA I-85 Atlanta

9-in [22.9 cm], 30-ft [9.1 m] undowelled JPCP built in 1968. Partial-depth repair, full-depth repair, grinding, joint resealing, and subsealing performed in 1982. Partial-depth repairs (Type III PCC with calcium chloride accelerator, bonded with epoxy) at about 5 percent of joints. Repairs are in excellent condition after 4 years, 5.86 million ESALs.

OH I-77 Cambridge

9-in [22.9 cm], 60-ft [18.3 m] JRCP built in 1967. CPR work performed in 1982 for NCHRP 1-21 study included full-depth repair, partial-depth repair, subsealing, load transfer restoration, diamond grinding, and joint resealing (all in outer lane only).(6) Two materials and bonding agents were used for the partial-depth repairs: PCC bonded with cement grout, and polymer concrete bonded with a commercial primer, Silikal. Two of the three partial-depth repairs surveyed are in good condition after 3 years, 2.1 million ESAL. The third (material type unknown) has experienced some material loss.

MI M-47 Midland

9-in [22.9 cm], 71-ft [21.6 m] JRCP built in 1966. Partial-depth repairs (Set 45) placed in 1984 at almost 100 percent of transverse joints. After 2 years repairs are not performing well. Extensive cracking and crumbling of repair material observed on several repairs, particularly at working cracks.

WI US-61 Boscobel

8-in [20.3 cm], 20-ft [6.1 m] undowelled JPCP built in 1953. Partial-depth repair, full-depth repair, and grinding performed in 1981. Observed joint deterioration resembles D cracking or freeze-thaw damage of concrete. Medium-severity spalling of longitudinal centerline joint also noted. Joint sealant is absent throughout most of project. Diamond-shaped partial-depth repairs (PCC with Acryl 60 bonding agent) at about 50 percent of transverse/longitudinal joint intersections, and triangular repairs at several outer slab corners. Temporary AC patches at unrepaired joint intersections and corners. Partial-depth repairs in fair condition after 4 years, 0.54 million ESALs. Some cracking of repair material was observed, but not as much as might be expected considering joints were not reestablished through repairs.

MN I-494 Minneapolis

9-in [22.9 cm], 40-ft [12.2 m] JRCP built in 1963. Partial-depth repair, full-depth repair, and joint resealing performed in 1978. Partial-depth repairs placed at transverse joints, transverse cracks, and transverse/longitudinal joint intersections. No distress observed after 7 years, 3.72 million ESALs.

MN I-694 Between TH 65 and TH 49

9-in [22.9 cm], 40-ft [12.2 m] JRCP built in 1964. Partial-depth repair, full-depth repair, and pressure relief performed in 1981. Partial-depth repairs (PCC) placed at almost all joints in both lanes. In good condition overall after 4 years, although some repairs at working cracks exhibited cracking.

MN TH 23 St. Cloud

9-in [22.9 cm], 80-ft [24.4 m] JRCP built in 1964. Partial-depth repair, full-depth repair, and joint resealing performed in 1983. Partial-depth repair (PCC) at 5 percent of joints, in good condition after 2 years.

MN US-61 St. Paul

9-in [22.9 cm], 40-ft [12.2 m] JRCP built in 1958. Partial-depth repairs placed in 1981. Deteriorated concrete at all joints in both lanes removed by cold milling. Repairs (PCC) are in excellent condition after 4 years, 1.59 million ESALs.

MN US-61 Duluth

8-in [20.3 cm], 40-ft [12.2 m] JRCP built in 1967. Partial-depth repair, full-depth repair, and joint resealing performed in 1979. Deteriorated concrete at almost all joints in both lanes removed by cold milling. Partial-depth repairs (PCC) are in excellent condition after 6 years, 0.84 million ESALs.

SD I-29 Sioux City

9-in [22.9 cm], 61.5-ft [18.7 m] JRCP built in 1961. Partial-depth repairs, full-depth repairs, pressure relief, and joint resealing performed in 1979. Partial-depth repairs (epoxy concrete) placed at all joints. Repairs are in very good condition after 6 years, 2.23 million ESALs. A few repairs exhibited low-severity cracking. Poor silicone joint sealant condition (adhesive failure, intrusion of incompressibles) noted throughout project, as well as evidence of reactive aggregate expansion.

SD I-29 Junction City

9-in [22.9 cm], 45-ft [13.7 m] JRCP built in 1961. Partial-depth repairs, full-depth repairs, pressure relief, and joint resealing performed in 1979. Partial-depth repairs (epoxy concrete) placed at all joints. Repairs are in very good condition overall after 6 years, 1.48 million ESALs. Low-severity longitudinal cracking observed on a few repairs. Poor neoprene joint sealant condition (adhesive failure, oxidation,intrusion of incompressibles) noted throughout project, as well as evidence of reactive aggregate expansion.

SD I-90 Chamberlain

9-in [22.9 cm], 45-ft [13.7 m] JRCP built in 1965. Partial-depth repair, full-depth repair, pressure relief, grinding, and joint resealing performed in 1982. Partial-depth repairs (epoxy concrete) placed at roughly 30 percent of transverse joints and cracks. Repairs are in excellent condition after 2 years, 0.83 million ESALs.

IL I-280 Moline

10-in [25.4 cm], 100-ft [30.5 m] JRCP built in 1961. Partial-depth repair, full-depth repair, undersealing, diamond grinding, subdrains, and joint resealing performed in 1984. Full-depth repairs placed at 17 percent of outer lane transverse joints and 48 percent of inner lane joints to correct spalling and faulting. Partial-depth repairs (Type I PCC) at 20 percent of the transverse joints; all are in excellent condition. Medium-severity spalling and poor sealant condition observed at many unrepaired joints (47 percent in outer lane and 40 percent in inner lane) suggests need for additional partial-depth repair.

NE I-80 Kearney

9-in [22.9 cm], 46.5-ft [14.2 m] JRCP built in 1962, opened to traffic in 1963. Full-depth repairs, partial-depth repairs, pressure relief, and joint resealing performed in 1982. Full-depth repairs placed at 10 percent of transverse joints in outer lane, 4 percent of joints in inner lane, and major cracks, to correct spalling caused by mildly reactive aggregate. Localized spalls at joints and major cracks repaired by partial-depth repairs. Pressure relief joints installed at 2000-ft [610 m] intervals to reduce pressure build-up. Partial-depth repairs (epoxy concrete) are not performing well. Some were placed at working cracks without cracks being reestablished, and have experienced spalling and material loss (typically 5 to 15 percent of repair area).

NE I-80 Lincoln, Milepost 382

9-in [22.9 cm], 46.5-ft [14.2 m] JRCP built in 1962. Partial-depth repairs, full-depth repairs, pressure relief joints, and joint resealing performed in 1982. Full-depth repairs placed at 13 percent of joints in outer lane, 9 percent of joints in inner lane, and major cracks, to correct spalling caused by mildly reactive aggregate. Localized spalls at joints and major cracks were repaired by partial-depth repairs (epoxy concrete). Pressure relief joints installed at 1-mile [1.6 km] intervals to reduce pressure build-up. Minor material loss noted on partial-depth repairs, not enough to warrant replacement. Partial-depth repairs are performing well after 5 million ESALs.

NE I-80 Lincoln, Milepost 404

10-in [25.4 cm], 16.3-ft [5 m] JPCP built in 1960. Partial-depth repairs, full-depth repairs, and joint resealing performed in 1984. Partial-depth repairs (epoxy concrete) were placed to correct localized spalling at transverse and longitudinal joints. Partial-depth repairs placed at working cracks are not performing well. Some were placed without the cracks being reestablished and have experienced spalling and material loss (up to 25 percent of the repair area). Partial-depth repairs not placed at working cracks are performing well, after 1 year, 1.2 million ESALs.

LA I-10 Baton Rouge

10-in [25.4 cm], 58.5-ft [17.8 m] JRCP built in 1971. Rehabilitated in 1984 as part of FHWA Demonstration Project No. 69. Partial-depth repair, full-depth repair, subsealing, load transfer restoration, diamond grinding, joint resealing, crack repair and subdrainage improvement performed. Partial-depth repairs (Type I PCC with calcium chloride accelerator) placed at 60 percent of transverse joints to correct localized spalling at transverse/longitudinal joint intersections. Pre-rehab survey by LaDOT found longitudinal cracking and spalling at 97 percent of joint intersections, perhaps attributable to improper forming of centerline joint or dowel bar misalignment. In 1985 survey, 38 percent of repairs (mostly in outer lane) noted as exhibiting material loss (cracking and crumbling of the concrete) in the range of 10 percent to 20 percent of repair, maximum of 40 percent on a few repairs. Repair deterioration may have resulted from inappropriate use, i.e., improper joint construction and/or dowel bar misalignment that may have caused cracking through full depth of slab. Maximum repair placement depth was 4 in [10.2 cm], so it is also possible that repairs came into contact with and were damaged by movement of dowel bars.

TX I-40 Houston, Milepost 731

10-in [25.4 cm], 60.5-ft [18.4 m] JRCP built in 1967. Partial-depth repairs and full-depth repairs placed in 1984. Partial-depth repairs (PCC) placed at some transverse/longitudinal joint intersections. Many unrepaired joint intersections exhibited medium-severity spalling, suggestive of poor joint construction techniques. Joints were reestablished through repairs. Repairs are in excellent condition after 2 years, 4.04 million ESALs.

TX I-40 Houston, Milepost 741

10-in [25.4 cm], 61.5-ft [18.7 m] JRCP built in 1966. Partial-depth repairs and full-depth repairs placed in 1982. Fewer partial-depth repairs (PCC) than at section beginning at milepost 731, and much less spalling observed at transverse and longitudinal joints. Repairs are in excellent condition after 4 years, 6.50 million ESALs.

TX US-59 Houston

10-in [25.4 cm], 61-ft [18.6 m] JRCP built in 1961. Partial-depth repairs and full-depth repairs placed in 1983. Partial-depth repairs (PCC) at 5 percent of transverse joints, predominantly in outer lane at transverse/longitudinal joint intersections. Repairs are in excellent condition after 3 years, 5.88 million ESALs.

WY I-80 Rawlins

8-in [20.3 cm], 20-ft [6.1 m] JPCP built in 1964. Partial-depth repair, full-depth repair, subsealing, diamond grinding, and joint resealing performed in 1982. There is an average of one working mid-panel crack in each slab; most of these have one or two partial-depth repairs (PCC). Cracks have been carefully reestablished and sealed through repairs. Repairs are in excellent condition after 4 years, 2.38 million ESALs.

AZ I-17 Phoenix

9-in [22.9 cm], 15-ft [4.6 m] undowelled JPCP built in 1961. Partial-depth repairs and diamond grinding performed in 1981. Partial-depth repairs (epoxy concrete) placed at 17 percent of transverse joints. Repairs are in excellent condition after 5 years, 4.5 million ESAL. Poor joint sealant condition and joint spalling noted in field survey suggest a need for additional partial-depth repair and joint cleaning and resealing.

6.3 DESIGN AND CONSTRUCTION GUIDELINES

6.3.1 Introduction

These guidelines were originally prepared under NCHRP Project 1-21 and published in NCHRP Report No. 281, Transportation Research Board, 1985.(6) The guidelines were updated in early 1987 based upon the findings and results of the "Pressure Relief and Other Joint Rehabilitation Techniques" study conducted for the FHWA.(34) Further updates resulted from the research conducted for the "Determination of Rehabilitation Methods for Rigid Pavements" study conducted for the FHWA, which is described in this final report.

These guidelines cover permanent partial-depth repair of jointed portland cement concrete (PCC) pavements. Partial-depth repairs extend the life of PCC pavements by restoring ride quality to pavements that have spalled joints. Partial-depth repair of spalled areas also restores a well defined, uniform joint or crack sealant reservoir prior to joint or crack resealing. When properly placed with durable materials, these repairs can perform well for many years. In fact, several rehabilitation projects exist on which partial-depth repairs placed 10 years ago do not show any deterioration.

Partial-depth repair is an alternative to full-depth repair in areas where deterioration is located primarily in the upper third of the slab and the existing load transfer devices (if any) are still functional. When applied at appropriate locations, partial-depth repair can be more cost effective than full-depth repair. The cost of partial-depth repair is largely dependent upon the size, number, and location of repair areas, as well as the materials used. Lane closure time and traffic volume also affect production rates and costs.

6.3.2 Need for Partial-Depth Repair

Partial-depth repairs can be used to address spalling which is limited to the top few inches of the slab. Spalls are often caused by infiltration of incompressible materials into joints. This type of spalling is common on pavements with long joint spacing, where larger joint movements occur.

Transverse joint spalling in some States has been caused by the use of metal joint forming inserts (Unitubes) in areas where aggregate hardness makes sawing of joints difficult and expensive. These inserts often corrode and entrap incompressibles, resulting in joint spalling.

Other sources of spalling and scaling include reactive aggregate distress and "D" cracking, high reinforcing steel, and overfinishing.

6.3.3 Effectiveness of Partial-Depth Repair

The performance of partial-depth repairs has been excellent on many projects where their use was appropriate and where inspection and quality control are stringent. However, high rates of partial-depth repair failure have been observed on other projects. These failures are commonly caused by:

- Inappropriate use of partial-depth repairs (e.g., where full-depth repairs are needed).
- Poor construction techniques (failure to remove all deteriorated materials, failure to provide vertical saw cuts at the repair boundaries, failure to provide a compressible material in joints and cracks adjacent to or within the patch area, inadequate surface preparation and bonding provisions, insufficient repair material consolidation).
- Compression failures (caused by repair material entering working cracks and joints, thereby restricting slab expansion).
- Use of inappropriate, thermally incompatible or variable-quality repair material.

6.3.4 Limitations of Partial-Depth Repair

Partial-depth repairs are not suitable for spalls that extend deeper than one third of the slab thickness, because the removal of deteriorated concrete below this depth and proper reforming of the joint are often hampered by the presence of reinforcing steel and dowels. Furthermore, sound concrete at the bottom of the repair is more easily damaged as the depth of removal increases.

Partial-depth repairs are not suitable for working cracks or joints unless the crack or joint is reestablished through the repair directly above the discontinuity in the underlying slab. Full-depth repairs or load transfer restoration should be considered at working cracks.

If several spalls are present on one joint, it may be more economical to place a full-depth repair along the entire length of the joint than to repair individual spalls. Very small spall areas along joints (less than 6 in [15.2 cm] long and 1.5 in [3.8 cm] wide) generally do not need to be repaired unless the joint is to be resealed with a preformed compression seal.

6.3.5 Concurrent Work

Slab stabilization should be performed prior to placing partial-depth repairs so that any spalls which might develop from accidental lifting or movement of the slabs can be repaired.

Full-depth repairs should be placed concurrently with or after the placement of partial-depth repairs so that locations of deep deterioration can be identified and repaired full-depth.

Diamond grinding should be accomplished after the completion of all activities which might increase the roughness of the pavement surface (including slab stabilization and partial- and full-depth repairs).

Joint cleaning and resealing should be accomplished last to prevent damage of the new sealant by repair and grinding operations and to obtain the proper shape factor and recession of the sealant within the reservoir.

6.3.6 Partial-Depth Repair Materials

Material Selection

Repair material selection depends on available curing time, ambient temperature, available funds, and the size and depth of the repairs. Portland cement concrete is generally accepted as the most universally compatible repair material. Typical mixes combine Type I, II, or III portland cement concrete with coarse aggregate not greater than one half the minimum repair thickness (3/8-in [0.95 cm] maximum size is often used).

The concrete should have a minimum compressive strength of 3,000 psi [20.7 MPa] at the time of opening to traffic. When early opening is required, such as within 24 hours, this accelerated strength gain can be obtained by using not more than 8 bags of Type III cement per cubic yard and calcium chloride in an amount not to exceed 2 percent by weight of the cement, or by using other accelerating admixtures.

Type III cement, with or without admixtures, has been used for repair mixtures longer and more widely than most other materials because of its relatively low cost, availability, and ease of use. Rich mixtures (up to 8 bags) gain strength rapidly in warm weather, although the rate of strength gain may be too slow to permit quick opening to traffic in cool weather. Insulating layers can be used to retain the heat of hydration and reduce curing time.

Many projects require that repairs be opened to traffic within a few hours. To meet this challenge, a wide variety of rapid-setting and/or high-early-strength materials, such as epoxy resin mortars and concretes, have been developed.(62,67,69,70) Many of these products are very sensitive to construction procedures or may be used only within very narrow temperature ranges. The manufacturer's directions regarding handling, mixing, placement, consolidation, screeding, and curing must be followed exactly. The durability of such materials under local climatic conditions must be carefully evaluated. These materials must also be thermally compatible with the concrete in the pavement. Significant differences in coefficients of thermal expansion can cause premature repair failure.

Partial-depth repair failure is frequently caused by shrinkage of the repair material, which weakens the repair and initiates progressive deterioration. Some agencies have successfully minimized shrinkage by using expansive (e.g., high gypsum content) mortars for large repairs.

Epoxy resin mortars and concretes have also been used. Available epoxy resins have a wide range of setting times. The epoxy concrete mix design must be compatible with the concrete in the pavement. Differing coefficients of thermal expansion can cause repair failures. Deep epoxy repairs must frequently be placed in lifts to control heat development.

When using a proprietary patching material, it is essential that the manufacturer's recommendations are followed closely. Handling, mixing, placement, consolidation, screeding and curing of the repair material must be in accordance with the manufacturer's written instructions. The specifying agency should investigate the various repair materials available to determine their suitability for application and environment.(67) Other valuable information on repair material performance can be obtained from agencies that have used the material.(67) The working tolerances of some of the proprietary repair materials are too tight for most repair projects (i.e., ambient temperature range for placement and curing, exact measurements of quantities, etc.).

Calcium aluminate cement (also called high-alumina cement) has been used by some agencies for partial-depth repairs where high early strength and/or sulfate resistance was desired, but has generally not provided good performance.(72) This is attributed to a chemical conversion which occurs in calcium aluminate cement which can cause substantial strength loss. This conversion occurs rapidly at temperatures greater than 86 ^{O}F [30 ^{O}C], but also occurs, albeit more slowly, even at temperatures below 68 ^{O}F [20 ^{O}C]. If the temperature of concrete made with calcium aluminate cement exceeds 77 ^{O}F [25 ^{O}C] <u>at any time</u> in its life, conversion and subsequent strength loss may occur. Temperatures in excess of 77 ^{O}F [25 ^{O}C] for even a few hours during initial curing can cause substantial strength loss resulting in failure of the repair. The repair's sulfate resistance is also substantially diminished by this loss in strength. For these reasons, calcium aluminate cement is prohibited for structural use in many countries. Calcium aluminate cement is not recommended for partial-depth repairs.

<u>Bonding Agents</u>

Sand/cement grouts have proven adequate when used as bonding agents with PCC repair material, provided the repairs are protected from traffic for 24 to 72 hours. Excellent results have been obtained with 7-sack Type III mixes using a sand-cement grout bonding agent, with a cure period of 72 hours before opening to traffic.

Epoxy bonding agents have been used successfully with both PCC and proprietary repair materials to reduce required curing time to 6 hours or less.

6.3.7 Preparation of the Repair Area

<u>Location of Repair Boundaries</u>

The actual extent of deterioration in the concrete may be greater than is visible at the surface. In early stages of spall formation, weakened planes often exist in the slab with no signs of deterioration visible at the surface. The extent of deterioration can be determined by "sounding" the concrete with a solid steel rod, chains, or a ball peen hammer. Areas yielding a clear ringing sound are judged to be acceptable while those emitting a dull sound are considered weak. Sophisticated sounding equipment (e.g., the Delam-Tech) is also commercially available.

All weak concrete must be located and removed if the repair operation is to be effective. The area marked for sawing should be 3 to 4 in [7.6 to 10.2 cm] outside the visibly distressed area.

Sawing Repair Boundaries

A vertical saw cut 1 to 2 in [2.5 to 5.1 cm] deep should be made beyond the boundary of the unsound area to be removed (see figure 74). The cut boundary should be straight and vertical to provide a vertical face and square corners. Cutting repair boundaries with jackhammers results in "scalloped" boundaries into which repair materials must be "feathered." Vertical boundaries reduce the spalling associated with thin or "feathered" concrete along the repair perimeter.

Removal of Deteriorated Concrete

The partial-depth removal of unsound concrete is usually accomplished with jackhammers. The initial breakup can be done with hammers weighing up to 30 pounds [13.6 kg]. Removal begins near the center of the area to be removed and proceeds towards (but not to) the edges. Care must be taken to avoid fracturing the sound concrete below the repair and undercutting or spalling repair boundaries. Removal near the repair boundaries must be completed with lighter (10- to 20-pound [4.5 to 9.1 kg]) hammers, particularly in the areas of the repair boundaries. Even hammers of this size fitted with gouge bits can damage sound concrete. Carefully operated small hammers with spade bits have been used successfully to remove unsound concrete without fracturing the underlying sound concrete.

The surface of the area to be removed may be sawed in a shallow crisscross or waffle pattern to facilitate concrete removal. Pneumatic scarifiers can also be used to break up the area between the saw cuts. Carbide-tipped cold milling machines and diamond blade grinding machines have been used for larger areas, such as for full-lane-width repairs.

After removal, the bottom of the repair area is checked by "sounding" or other specified methods to ensure that all deteriorated concrete has been removed. Any remaining areas of unsound concrete must be removed.

The typical depth of concrete removal varies from 1 to 4 in [2.5 to 10.2 cm]. The removal method should provide a very irregular surface to provide a high degree of mechanical interlock between the repair material and the existing slab.

If sound concrete cannot be reached (e.g., the area is unsound through the depth of the slab or unsound material cannot be removed because of reinforcing or load transfer devices) a full-depth repair is required. Small areas of full-depth repair have been combined with partial-depth repairs, but these generally do not perform as well as regular full-depth repairs.

Joint Preparations

Partial-depth repairs placed adjacent to transverse, centerline, or shoulder joints require special construction preparations. Partial-depth repairs placed at the centerline joint directly in contact with the adjacent lane frequently develop spalling because of curling and differential movement of the slabs. This can be prevented by placing a polyethylene strip (or other thin bond-breaker material) along the centerline joint just prior to placement of the repair material.

The most frequent cause of failure of partial-depth repairs placed directly across transverse joints or cracks is crushing by the compressive forces created when the slabs expand. This must be prevented by placing a strip of Styrofoam or asphalt-impregnated fiberboard between the new concrete and the adjoining slab (see figures 74 and 75). This material must be placed so as to prevent intrusion of the repair material into the opening. Failure to do so can result in compressive

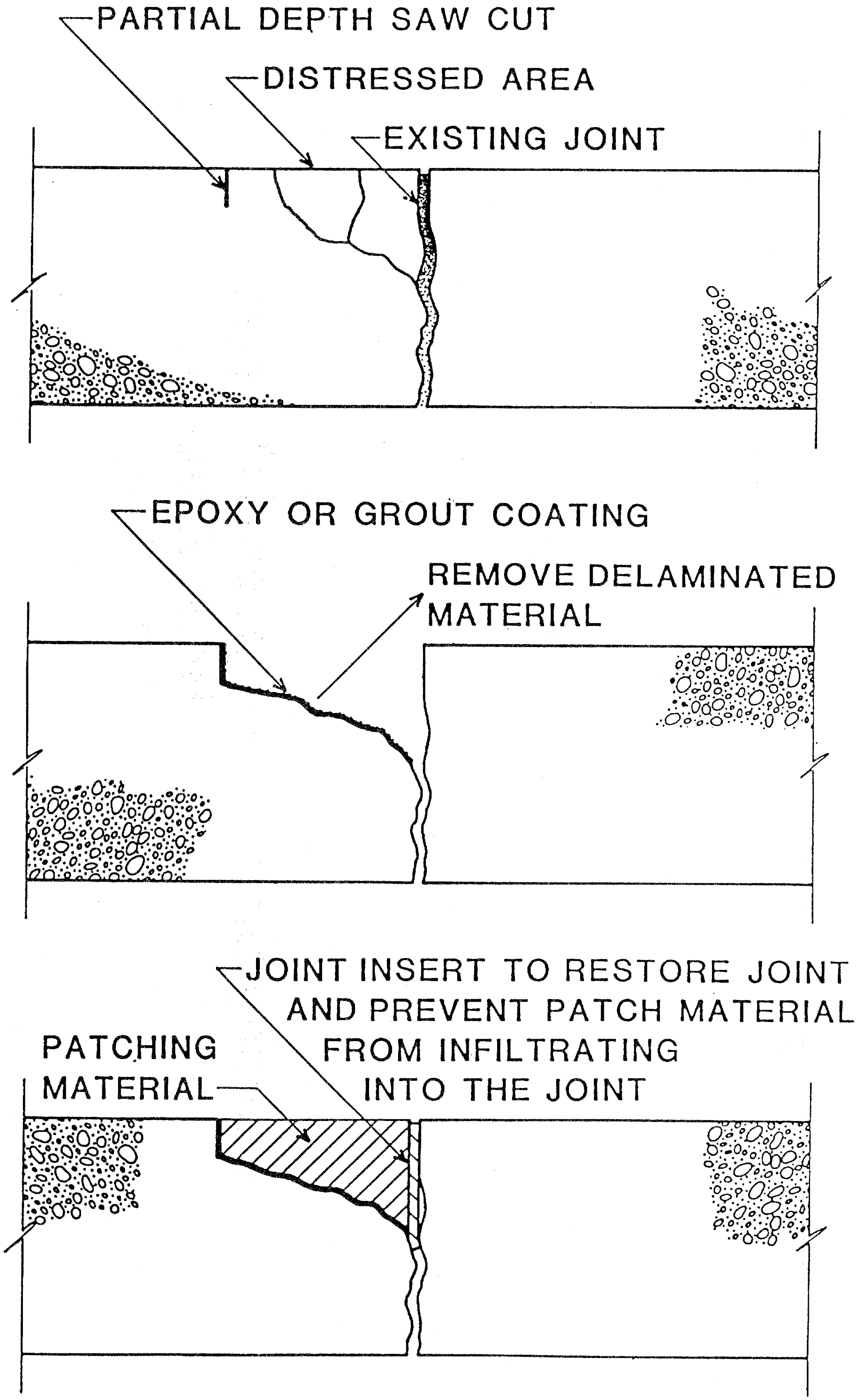
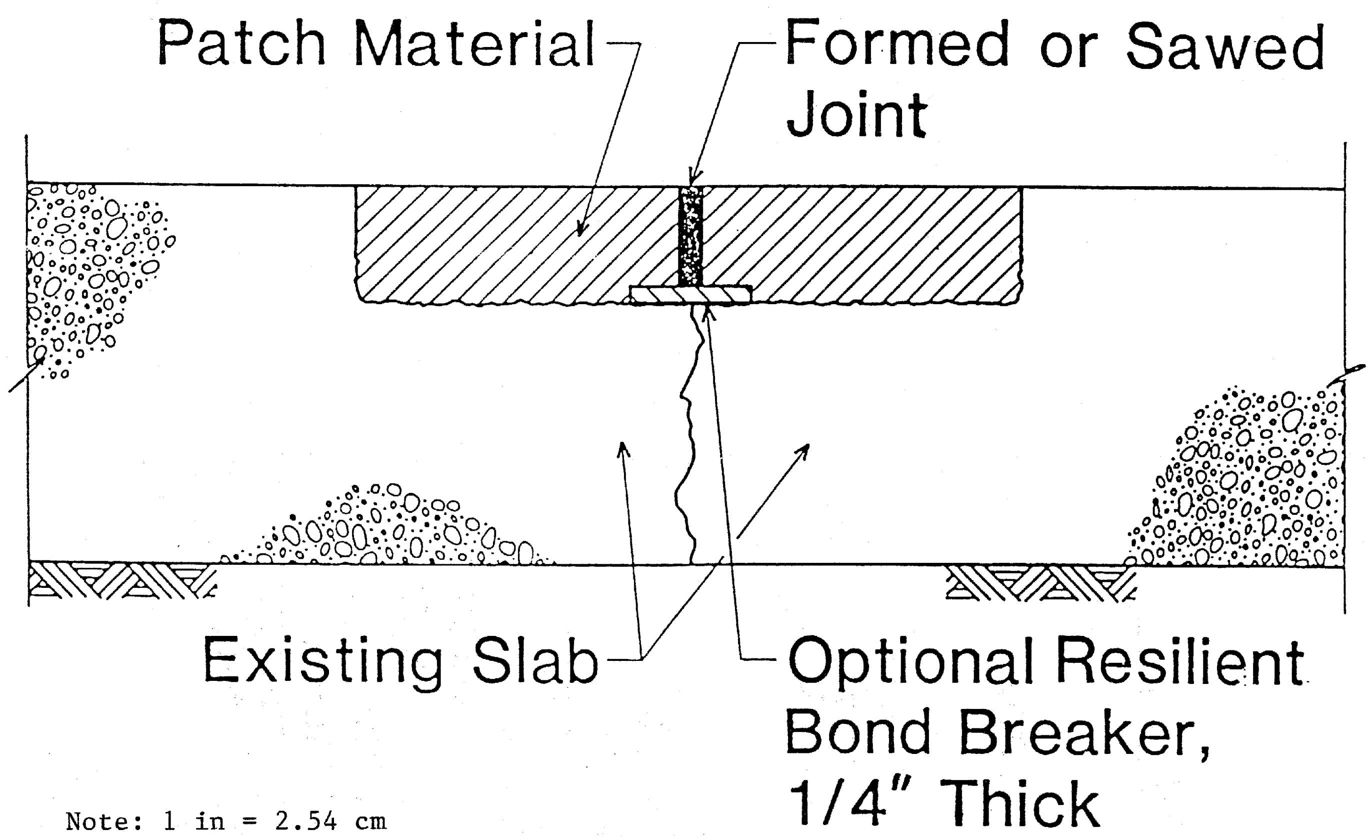


Figure 74. Steps in partial-depth repair operation.(6)



Note: 1 in = 2.54 cm

Figure 75. Placement of partial-depth repair over a joint or crack.(6)

stresses at lower depths that will damage the repair. This material will also guard against damage due to deflection of the joint under traffic.

Where spalling has been caused by a metal insert such as Unitube, the spalls usually start at the bottom fin of the insert about 2.5 in [6.3 cm] below the surface. When repairing this type of spall, it is recommended that the insert be sawed out along the entire length of the joint to avoid further deterioration. The joint can then be repaired and resealed.

If a repair is to be placed along the outer edge of a lane, it must be formed along the lane/shoulder joint. If the repair material is allowed to flow into the shoulder, it may form a "key" which will restrict longitudinal movement of the slab and damage the repair.

All existing joint sealing or expansion joint materials should be removed to prevent contamination of the repair material. Sandblasting is an acceptable means of accomplishing this removal; solvents must never be used.

<u>Cleaning the Repair Area</u>

Following removal of the concrete, the surface of the repair area must be cleaned. If jackhammers were used to remove concrete, dry sweeping, sandblasting and compressed air blasting are normally required to provide a clean surface. The compressed air must be free of oil, since contamination of the surface will prevent bonding. This can be checked by placing a rag over the nozzle and visually inspecting for oil.

Sandblasting is highly recommended for cleaning the surface. Sandblasting removes dirt, oil, thin layers of unsound concrete, and laitance. High-pressure water may also be used to remove contaminants, but sandblasting usually produces better results.

With all methods, the prepared surface must be checked prior to placing the new material. Any contamination of the surface will reduce the bond between the new material and the existing concrete. If the fingers pick up material (dust, etc.) when rubbed across the prepared surface, the surface must be cleaned again.

<u>Application of Bonding Agent</u>

After the surface of the existing concrete has been prepared, and just prior to placement of the repair material, it should be coated with a bonding agent to ensure complete bonding of the repair material to the surrounding concrete (figure 74). A saturated, surface-dry condition is desirable for application of cement grouts. When epoxies or other manufactured grouts are being used, the manufacturer's directions must be followed closely.

Thorough coating of the bottom and sides of the repair area is essential. This may be accomplished by brushing the grout onto the concrete. Spraying may be appropriate for large repair areas. The grout should not be allowed to puddle.

The grout should be placed immediately before the repair material is placed so that the grout does not set before it comes into contact with the repair material.

Cement grout requires a minimum of 72 hours of curing prior to opening. Repairs that must be opened to traffic in less than 72 hours must use an epoxy bonding agent. Many epoxy bonding agents require only 6 hours of curing prior to opening.

6.3.8 Repair Placement and Finishing

<u>Repair Material Mixing</u>

The volume of material required for a partial-depth repair is usually small (0.5 to 2.0 cu ft [0.014 to 0.057 cu m]). Ready-mix trucks and other large equipment cannot efficiently produce such small quantities since maximum mixing times for a given temperature would be easily exceeded, resulting in waste of material. Small drum or paddle-type mixers with capacities of up to 2 cubic feet [0.057 cu m] are often used. Based on trial batches, repair materials may be weighed and bagged in advance to facilitate the batching process. Continuous feed mixers are also popular.

<u>Placement and Consolidation of Material</u>

PCC repairs should not be placed when air or pavement temperatures are below 40 ^{o}F [4.4 ^{o}C]. At temperatures below 55 ^{o}F [12.8 ^{o}C] substantially longer curing times may be required, although the use of insulation will shorten curing times.

The repair material must be consolidated during placement. Failure to do so may result in poor repair durability, spalling, and rapid deterioration. For example, voids located at the interface between the repair material and existing pavement can result in total debonding and loss of repair material.

The purpose of consolidation is to release trapped air from the fresh mix. Three common methods of accomplishing this are:

- Use of internal vibrators with small heads (less than 1 in [2.5 cm] in diameter).
- Use of vibrating screeds.
- Rodding or tamping and cutting with a trowel or other hand tool.

The internal vibrator and the vibrating screed give the most consistent results. The internal vibrator is often more readily available and is used most often, although very small repairs may require the use of hand tools.

The placement and consolidation procedure begins by slightly overfilling the repair with repair material to allow for a reduction in volume during consolidation. The vibrator is held at a slight angle (15 to 30 degrees) from the horizontal and is moved through the concrete in such a way as to vibrate the entire repair area. The vibrator should not be used to move material from one place to another within the repair as this may result in segregation. Adequate consolidation of the mix is achieved when the mix stops settling, air bubbles no longer emerge, and a smooth layer of mortar appears at the surface.

On very small repairs, the mix can be consolidated using hand tools. Cutting with a trowel seems to give better results than rodding or tamping. The tools used should be small enough to easily work in the area being repaired.

<u>Screeding and Finishing</u>

Partial-depth repairs are usually small enough so that a stiff board resting on the adjacent pavement can be used as a screed. The materials should be worked against the grade (if any exists) to prevent downflow. This also pulls the material against the face of the original pavement, which enhances bonding. Screeding generally requires at least two passes to ensure a smooth repair surface.

The repair surface must be hand-trowelled to remove any remaining minor irregularities. The edge of a repair located adjacent to a transverse joint should be tooled to provide a good reservoir for joint sealant. Excess mortar from trowelling can be used to fill any saw cuts extending into the adjacent pavement at repair corners.

Partial-depth repairs typically cover only a small percentage of the pavement surface and have little effect on skid resistance. However, the surface of the repair should match that of the surrounding slab as much as possible.

<u>Curing</u>

Curing is as important for partial-depth repairs as it is for full-depth repairs. Since partial-depth repairs often have large surface areas with respect to their volumes, moisture can be lost quickly. Inadequate attention to curing can result in the development of shrinkage cracks that may cause the repair to fail prematurely.

All of the standard curing methods used for full-depth repairs may be considered for partial-depth repairs as well. The most effective curing procedure in hot weather is to apply a white-pigmented curing compound as soon as water has evaporated from the repair surface. This will reflect radiant heat while allowing the heat of hydration to escape, and will provide protection for several days. Moist burlap and polyethylene can also be used, but they must be removed when the roadway is opened to traffic. In cold weather, insulating blankets or tarps can be used to provide more rapid curing and opening to traffic. The required repair curing time should be stated in the project plans and specifications. Epoxy and proprietary repair materials should be cured as recommended by their manufacturers.

6.3.9 Preparation of Plans and Specifications

Partial-depth repair costs are highly dependent upon the size, number and location of repair areas. Since there is typically some delay between the time that a project is selected for repair and the time that the repair work is actually performed, during which the deterioration of the pavement may progress significantly, it is essential that the required repair quantities be verified by a detailed condition survey prior to preparation of plans and specifications. Cost overruns exceeding 500 percent have occurred on partial-depth repair projects where the actual amount of distress needing repair was greatly underestimated.(2) It is also recommended that coring be performed at a representative number of spalled joints and/or cracks to determine the depth of deterioration and differentiate on the plans between areas which should be partial-depth repaired and areas which should be full-depth repaired.

6.4 CONCLUSIONS AND RECOMMENDATIONS

Partial-depth repair is the correction of localized surface distress in concrete pavements by removal of deteriorated concrete and replacement with a suitable repair material. Partial-depth repair improves ride quality and may arrest further development of the distress addressed. It also restores a uniform, well defined joint sealant reservoir prior to joint resealing.

In this study, partial-depth repairs were surveyed on 40 projects in 16 States. The projects are well distributed throughout the major climatic zones of the United States, and cover a wide range of traffic levels. The database includes JRCP and JPCP (with and without dowels), and several joint spacings, slab thicknesses, and pavement ages. The partial-depth repairs surveyed ranged in age from 1 to 10 years. The oldest partial-depth repair projects were located in Virginia, Georgia, and Minnesota, and South Dakota.

Partial-depth repairs have performed poorly on some projects; but they have performed well on the majority. This suggests strongly that, although good long-term performance is achievable with partial-depth repair, how this performance can be achieved is not well understood by the practicing engineer. The most significant factors influencing the success of a partial-depth repair application are:

- Appropriateness of partial-depth repair to repair the distress present.
- Adequacy of construction techniques, materials, and quality control.

The most important reason that partial-depth repairs fail is that they are used in places where they are not appropriate, e.g., where full-depth repairs are needed. Even when used appropriately, partial-depth repairs often fail if constructed with poor techniques or unsuitable materials. Unless partial-depth repairs are used only for surface spalling and constructed well with durable materials, failure within as little as 1 year is virtually guaranteed. The majority of the projects surveyed represent examples of how not to perform partial-depth repair.

However, a number of examples of excellent partial-depth repair performance also exist. These can be found on projects in several different States, with different traffic levels, pavement designs, repair materials, construction techniques, and concurrent restoration techniques. In fact, on the projects where partial-depth repairs have been successful, they are generally in such good condition that the long-term effects of traffic and climate are not readily apparent. Furthermore, the oldest projects in the database were only 10 years old. There is not sufficient long-term performance data available at this time to prove or disprove that the expected life of a partial-depth repair can exceed 10 years, nor to model its long-term performance quantitatively.

The poor performance of partial-depth repairs on many projects should not deter agencies from their use, since good performance is certainly achievable. Indeed, these projects provide valuable information on the appropriate use and successful construction of partial-depth repairs. A better understanding of long-term partial-depth repair performance will develop as more successful projects are evaluated.

6.4.1 Appropriate Use of Partial-Depth Repairs

Partial-depth repair is strictly removal and replacement of small, shallow areas of deteriorated concrete with a suitable repair material, i.e., one which is comparable in strength and volume stability to the concrete in the existing slab. Ideally, the repair material bonds to sound concrete and becomes an integral part of the slab. Partial-depth repair is appropriate for certain types of concrete pavement distress which are confined to the top few inches of the slab. Distresses which have been successfully corrected with partial-depth repair include:

- Spalling caused by intrusion of incompressible materials into transverse joints.
- Spalling caused by use of metal (Unitube) joint forming inserts.
- Scaling due to high reinforcing steel, overfinishing, or weak concrete.
- Early stages of "D" cracking or reactive aggregate distress.

Partial-depth repairs replace concrete only, and cannot accommodate the movements of working joints and cracks, load transfer devices, or reinforcing steel without experiencing high stresses and material damage. Therefore, they should only be used to correct distress which does not extend through more than one half of the slab thickness nor to the depth of any reinforcing steel or dowel bars present. A more conservative limit of one third of the slab thickness has even been suggested.

The distresses listed above are not always limited to the upper few inches of the slab and so may not always be corrected by partial-depth repair. Incompressibles may infiltrate transverse joints from the bottom of the slab as well as the top, and cause spalling which is not visible at the surface. Spalling associated with use of Unitubes often results from entrapment of incompressibles in the fins, which typically spalls the top 2 to 3 in [5.1 to 7.6 cm] of the joint, but may extend deeper. Scaling can only be repaired partial-depth if the concrete is not deteriorated to the depth of the reinforcing steel. "D" cracking may occur at the slab surface only, but more often it begins at the slab/base interface where moisture accumulates, and is not visible at the surface until deterioration of the bottom of the slab is already extensive. In most cases, partial-depth repair cannot be considered a permanent solution to the problem of "D" cracking deterioration. Scaling and map cracking caused by reactive aggregate can be corrected partial-depth, but it should be recognized as the result of fracturing of the cement matrix and probable structural degradation of the concrete. Furthermore, partial-depth repair cannot halt or effectively repair cracking and joint damage caused by expansion and subsequent compressive stress buildup in reactive aggregate pavements.

Other types of concrete pavement distress which are <u>not</u> likely to be correctable by partial-depth repair include:

- Cracking and joint spalling caused by compressive stress buildup in long-jointed pavements.

- Spalling caused by dowel bar misalignment or lockup.

- Transverse or longitudinal cracking caused by improper joint construction techniques (late sawing, inadequate saw cut depth, or inadequate insert placement depth).

- Working transverse or longitudinal cracks caused by shrinkage, fatigue, or foundation movement.

On any project where partial-depth repair is being considered, it is highly recommended that coring be performed at representative joints to determine the depth of deterioration, and assess the appropriateness of partial-depth repair in accordance with the above guidelines.

6.4.2 Construction Techniques and Materials

The procedure for partial-depth repair construction involves the following steps:

1. Locating repair boundaries.
2. Sawing repair boundaries.
3. Removing deteriorated concrete.
4. Placing a form or insert to maintain the working joint.
5. Cleaning the repair area.

6. Applying the bonding agent.
7. Mixing the repair material.
8. Placing and consolidating the repair material.
9. Screeding and finishing the repair.
10. Curing.

The construction guidelines in section 6.3 provide detailed information on the successful performance of these steps. The construction steps most significantly affecting success of the repair include the following:

1. Removal of deteriorated concrete: All deteriorated concrete must be removed and sound concrete exposed to which the repair material can bond. Light jackhammers and hand tools must be used to remove the existing deteriorated concrete without damaging the underlying sound concrete. Deterioration found to extend beyond the top few inches of the slab or to the depth of the dowel bars or reinforcing steel should be corrected by full-depth repair.

2. Reestablishment of the joint: It is essential that the joint be maintained with a form or insert, or reestablished by sawing, and that repair material not be allowed to flow down into the joint. Crushing of repairs has occurred on projects where repair material infiltrated the joints and caused compressive stress buildup in the repairs when the joints closed. This is particularly true of repairs placed in cold weather.

3. Cleaning the repair surface: Unless all loose concrete and debris is removed, the repair material will not achieve good bond with the existing concrete. Sandblasting is recommended to achieve a clean surface; waterblasting and airblasting have also been used successfully.

4. Mixing, placing, and curing: Conventional practices (for PCC), practices verified by testing (for polymer concretes and other special concretes), or manufacturer's instructions (for proprietary materials) should be observed. Repairs should not be placed at ambient temperatures too low for them to attain adequate strength prior to opening to traffic, nor at temperatures so high that they experience excessive shrinkage.

A suitable repair material is one that is comparable in strength and thermal expansion to the existing concrete, achieves adequate strength gain to meet opening-to-traffic time requirements, has good durability, and is safe and convenient to use, in terms of mixing time, ambient temperature range, and heat liberation. Cost considerations will also influence material selection. Materials that have been used successfully for partial-depth repair include Type III PCC with or without an accelerating admixture, proprietary-rapid setting materials, and epoxy concrete. Partial-depth repairs constructed using calcium aluminate cement concrete have performed poorly, experiencing significant scaling, shrinkage, and debonding.

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