

NATIONWIDE PERFORMANCE OF CONCRETE PAVEMENT REHABILITATION

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INTRODUCTION

ABSTRACT

This paper summarizes the results of extensive field, laboratory, and analytical studies conducted into the evaluation and rehabilitation of concrete pavements. The field studies included 349 concrete pavement rehabilitation projects throughout the United States, the construction of a large load transfer restoration experiment on I-10 in Florida, and a full-depth repair experiment on I-74 in Illinois. A major laboratory study was conducted to investigate anchoring of dowels into existing concrete slabs for full-depth repair load transfer.

The rehabilitation techniques studied were diamond grinding, full-depth repair, partial-depth repair, load transfer restoration, edge support restoration, bonded concrete overlay, unbonded concrete overlay and crack and seat/AC overlay. Detailed condition data, traffic data, design and construction data and climate data were collected for each project. Analyses of the field and laboratory data were conducted to identify significant factors influencing the performance of rehabilitated concrete pavements, and to develop predictive models for distresses observed on rehabilitated concrete pavements, including faulting, cracking, joint deterioration, rutting, and reflection cracking. These models may be used to predict the performance of rehabilitated concrete pavements under various traffic, design, and climate conditions.

The major findings and predictive models obtained from this study were incorporated into a computer system for evaluation and rehabilitation of jointed plain, jointed reinforced, and continuously reinforced concrete pavements. This practical, user-friendly system developed for use on a personal computer is intended to assist engineers in project evaluation and selection of cost-effective rehabilitation techniques.

Study Objective

The study "Determination of Rehabilitation Techniques for Rigid Pavements" was conducted by the University of Illinois for the Federal Highway Administration between 1985 and 1988. The objective of the study was to improve procedures for evaluating and rehabilitating concrete pavements. This was accomplished through extensive field, laboratory and analytical studies which have provided new knowledge and understanding of the performance of rehabilitated concrete pavements. The results of this study are fully reported in References 1 through 4 and 6.

Field Studies

The field studies involved surveys of 349 rehabilitated sections of jointed plain and reinforced concrete pavement located in 24 states. A breakdown of projects by technique and state is given in Table 1. Eight rehabilitation techniques were selected for detailed study:

1. Diamond grinding,
2. Load transfer restoration,
3. Edge support restoration,
4. Full-depth repair,
5. Partial-depth repair,
6. Bonded concrete overlay,
7. Unbonded concrete overlay, and
8. Crack and seat and AC overlay.

Candidate projects were identified by state transportation departments. The states provided detailed information on each project's original design and construction, subgrade conditions, traffic history, and rehabilitation design and construction. As-built plans and special provisions were obtained for original construction and rehabilitation, and research reports documenting performance of the rehabilitation projects were obtained where possible. Consultation with state personnel was a vital part of the data collection process.

Historical traffic data (average annual daily traffic and percent commercial trucks) were also obtained from the states. Federal Highway Administration W-4 tables were used to obtain truck axle

Table 1. Breakdown of rehabilitation projects by technique and state.

STATE	FDR	PDR	DGD	LTR	CAS	UBOL	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

FDR = Full-depth repair

PDR = Partial-depth repair

DGD = Diamond grinding

LTR = Load transfer restoration

CAS = Crack and seat and AC overlay

UBOL = Unbonded PCC overlay

BOL = Bonded PCC overlay

ES = Edge support restoration

load distributions and compute truck factors for the projects. This information was used to compute accumulated 18-kip equivalent single-axle loads (ESALs) on the projects from the time of construction to rehabilitation, and from rehabilitation to the time of survey. Climatic data, including annual and monthly average temperature and precipitation, were obtained from the National Oceanic and Atmospheric Administration. Data from the weather station located closest to each project were used.

A standard field condition survey was performed on each uniform section by a trained two-person team. A uniform section was defined as a section of pavement with uniform characteristics along its length with respect to structural design, joint design and spacing, traffic, subgrade conditions, distress, and rehabilitation design. Two or more randomly located 1000-foot sample units were surveyed in each uniform section. The distress identification manual and condition survey procedures developed under NCHRP 1-19 [5] were used in the field surveys. Observed distresses were carefully mapped and photographed. Measurements of joint, crack, and full-depth repair joint faulting and width were made on restoration and concrete overlay projects, and wheelpath rutting measurements were made on crack and seat projects. Ride quality was rated by the survey team in one pass over the project in each lane at the posted speed limit. Drainage conditions were also noted.

Laboratory Study

The field study of full-depth repair performance was accompanied by a comprehensive laboratory study of full-depth repair dowel installation procedures and designs. Repeated shear loading of dowels was conducted for up to one million load repetitions, using specimens cut from an in-service Interstate highway (I-70 in Illinois). A factorial experimental design was developed to determine the effects of five full-depth repair dowel installation variables on deflection response: dowel diameter, annular gap (the difference between drilled hole radius and dowel radius), anchor material, embedment length, and drill type. The results of the laboratory study are summarized in this paper and provided in full in Reference 6.

Rehabilitation Performance Prediction Models

The data collected in the field study were analyzed to develop prediction models for deterioration of rehabilitated pavements, so that the service lives of the different rehabilitation techniques could be estimated. The twelve distress models developed for rehabilitated concrete pavement performance are described in this paper.

Evaluation and Rehabilitation System

The rehabilitation performance models were incorporated in a comprehensive system for evaluation and rehabilitation of jointed plain, jointed

reinforced and continuously reinforced concrete pavements. [3] The system (EXPEAR) was implemented in a set of three computer programs which operate on an IBM-compatible PC.

PERFORMANCE OF REHABILITATION TECHNIQUES

Diamond Grinding

Diamond grinding of jointed concrete pavements has been part of experimental and routine restoration work since 1965. A total of 76 grinding projects in 19 states were surveyed in this study. The projects ranged in age from 1 to 9 years, and had carried from 0.22 to 7.8 million ESALs since rehabilitation.

Diamond grinding has been used to improve the ride quality and extend the service lives of jointed concrete pavements. The rideability of a newly ground pavement is typically as good as or better than that of a newly constructed pavement. However, if no other restoration work except grinding is performed, faulting after grinding recurs at a faster rate than for newly constructed pavements, as illustrated in Figure 1. This is due to loss of aggregate interlock, elongation of dowel sockets, and voids beneath the pavement. The life extension provided by grinding may be very short on a pavement with poor joint load transfer unless grinding is accompanied by other restoration work.

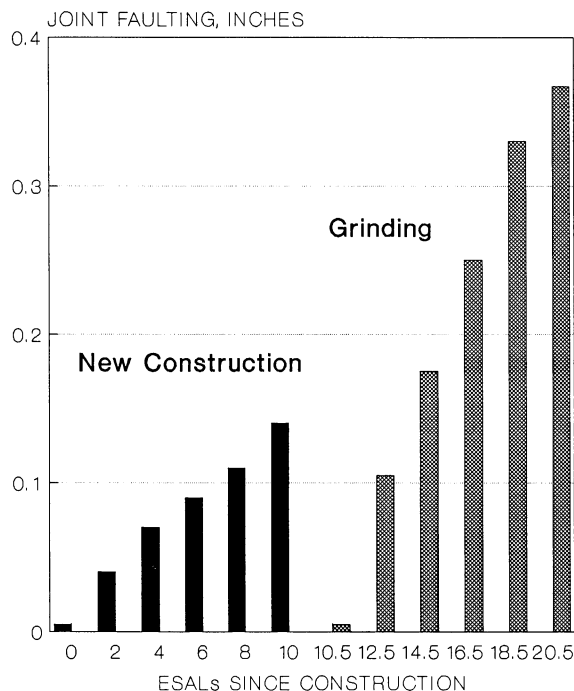


Figure 1. Projected faulting for JRCP new construction and after grinding.

The model developed for joint faulting after grinding is:

$$\begin{aligned} \text{FAULT} = & -5.62 (\text{ESAL} + \text{AGE})^{0.54} * \\ & [5.85 (\text{DRAIN} + \text{SOILCRS} + 1)^{0.0529} \\ & - 3.8 * 10^{-9} (\text{FI} / 100)^{6.29} \\ & + 0.484 (\text{THICK} + \text{PCCSH})^{0.335} \\ & + 0.1554 \text{BASE} - 7.163 \text{JTSPACE}^{0.0137} \\ & + 0.1136 \text{LDTR}] / 100 \end{aligned}$$

where:

FAULT = mean transverse joint faulting after grinding, inches

ESAL = accumulated 18-kip ESALs since grinding, millions

AGE = time since grinding, years

DRAIN = new or existing longitudinal subdrains
= 0, if subdrains not present or present but not functional
= 1, if subdrains present and functional

SOILCRS = subgrade soil classification
= 0, if fine grained (A-4 to A-7)
= 1, if coarse grained (A-1 to A-3)

FI = mean Freezing Index, Fahrenheit degree-days

THICK = thickness of PCC slab, inches

PCCSH = new or existing tied PCC shoulder
= 0, if not present
= 1, if present

BASE = type of base under PCC slab
= 0, if granular
= 1, if stabilized

JTSPACE = transverse joint spacing, feet

LDTR = load transfer restoration done by retrofitting dowel bars
= 0, if not done
= 1, if done

$R^2 = 0.38$

$n = 114$ pavement sections

SEE = 0.03 inch

Faulting after grinding is reduced if other restoration work is performed concurrently. Figure 2 illustrates the effects of load transfer restoration and edge support restoration in reducing faulting. Improving drainage conditions was also shown to reduce the rate at which faulting recurs after grinding, as shown in Figure 3.

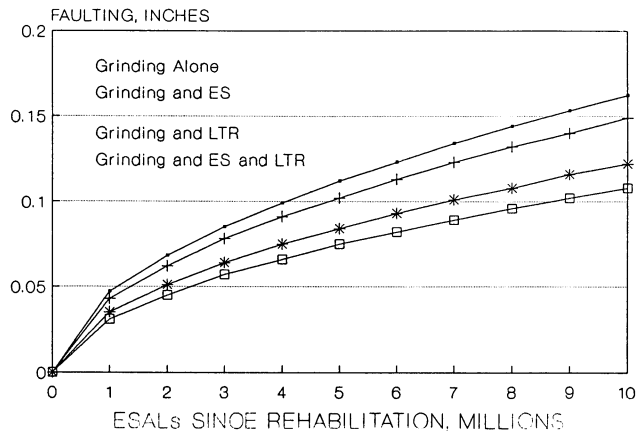


Figure 2. Predicted effects of restored load transfer and edge support on JPCP faulting after grinding.

The full benefit of diamond grinding in terms of pavement life extension is not likely to be realized when grinding is performed on a pavement with little remaining structural life. Twenty percent of the grinding projects surveyed in this study had large amounts of cracking (more than 1000 feet of cracking per lane mile). Pavements such as these are likely to require major structural rehabilitation long before the recurrence of joint faulting after grinding becomes significant.

Load Transfer Restoration

The ability of a joint or crack to transfer load is a major factor in its structural performance. Poor load transfer permits high deflections and leads to pumping, faulting, spalling, slab cracking, and corner breaks.

Load transfer across a joint or crack is tested by placing a load on one side of the joint or crack and measuring the deflections on the loaded and unloaded sides. Load transfer efficiency is defined as the ratio of the deflection of the unloaded side to the deflection of the loaded side, multiplied by 100 and expressed as a percentage. When the deflections on both sides are equal, the joint or crack is said to have 100 percent load transfer. Conversely, when the unloaded side does not deflect at all and the loaded side operates as a free edge, the joint or crack has no load transfer capability.

Load transfer efficiency is affected by many factors, including joint/crack spacing, temperature variation, joint/crack width, the quality of aggregate interlock, and the presence and dowel bars. Dowelled joints in relatively new pavements tend to have good load transfer, in the range of 70 to 100 percent. However, repeated heavy traffic loads can cause elongation of the dowel sockets and looseness of the dowels, which reduces load transfer. The load transfer efficiency at cracks between joints tends to be lower since it is entirely dependent upon aggregate interlock, which degrades rapidly under heavy traffic loads.

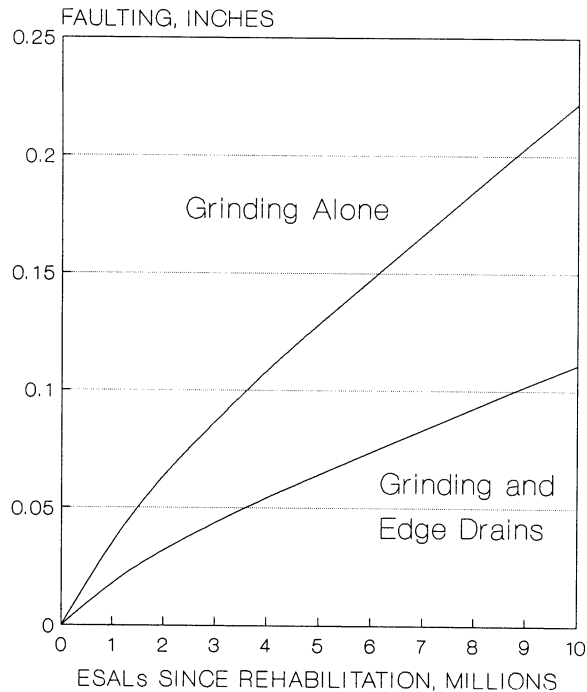


Figure 3. Predicted JPCP faulting after grinding alone versus grinding with edge drains.

Load transfer restoration typically increases load transfer efficiency to 90 to 100 percent. The benefit of such an increase is particularly great for joints and cracks with very poor load transfer. Increasing load transfer efficiency from 0 to 100 percent reduces slab deflections and stresses by 50 percent, which substantially reduces pumping, faulting, spalling, and cracking. Attempts to improve load transfer by retrofitting dowels or other mechanical devices have been employed by several highway agencies.

The performance of load transfer restoration was evaluated in this study on 13 projects in 9 states. The projects ranged in age from 1 to 9 years, and had carried from 0.22 to 7.8 million ESALs since rehabilitation. Four different types of load transfer devices were studied:

1. Retrofit dowels,
2. Double-vee devices,
3. Figure-eight devices, and
4. Miniature I-beams.

Load transfer device installations were observed at regular contraction joints spaced 15 to 100 feet apart, at full-depth repair approach and leave joints, at pressure relief joints, and at transverse cracks.

Most of the devices were placed in the outer traffic lane. As few as one or as many as eight devices were installed at any given joint or crack.

Performance ratings were taken on 1,525 individual load transfer devices. The retrofit dowel bars and miniature I-beams performed very similarly; 98 and 99 percent, respectively, showed no visible signs of deterioration. The double-vee shear devices and figure-8 devices performed similarly and more poorly than the other two device types; only 72 percent of the double-vee devices and 75 percent of the figure-8 devices showed no visible distress. It should be noted that the double-vee shear device design and installation has been improved on load transfer restoration projects more recent than the ones included in this study. The construction procedure now includes grooving of the core walls and precompression of the device. These new installation procedures were used on the Florida I-10 experimental load transfer project.

Faulting measurements were taken on a total of 369 restored joints. A model to predict the effect of load transfer restoration on faulting was developed using only those projects in the database which had zero faulting at the time of rehabilitation, i.e., projects on which diamond grinding was performed in conjunction with load transfer restoration. The model developed is similar to the grinding model, except that the individual effects of load transfer device types are included. Retrofit dowels were found to significantly reduce faulting, while double-vee shear devices, figure-eight devices, and I-beams had very little effect on faulting.

In addition to the field surveys of in-service projects, an experimental load transfer restoration project was designed and constructed on Interstate 10 in cooperation with the Florida Department of Transportation. The installations consisted of 1152 dowels in 8 different configurations and 432 double-vee shear devices in 6 different configurations.

Early performance results show that both device types are effective at reducing faulting, although the retrofit dowels are performing measurably better. One year after installation, faulting at joints with retrofit dowels was 33 percent lower on the average than at control joints. Longitudinal drains were placed at the same time. The most effective dowel configuration used five 1.5-inch-diameter, 18-inch-long dowels per wheelpath, although considerably less costly configurations with only three dowels per wheelpath performed almost equally well. Joints with shear devices faulted 20 to 25 percent less than control joints. The best shear device configuration used two devices in each wheelpath.

Individual device performance on the I-10 project was significantly better with the dowels than with the shear devices. Within a few months after installation, 26 percent of the double-vee devices showed some distress, while nearly all of the dowels were in excellent condition.

A similar experimental project in Georgia, which is currently 10 years old and has carried some 6 million ESALs since rehabilitation, better reflects the long-term effectiveness of the various load transfer restoration devices: faulting less

than 0.1 inch was measured on 99 percent of the joints restored with retrofit dowels, 87 percent of the joints with double-vee shear devices, and 88 percent of the joints with figure-eight devices.

The performance of the backfill material is critical to the performance of any load transfer device. Backfill material failure was either the most prominent or second most prominent failure mode for all four device types.

Edge Support Restoration

While some field evidence exists to show that tied PCC shoulders are beneficial to the performance of new pavements, almost no data has been available to show the effect of retrofit PCC shoulders. Edge support restoration (tied PCC shoulder or edge beam) was evaluated in this study on 13 projects in 9 states. The projects ranged in age from 1 to 21 years and had carried from 0.4 to 2.8 million ESALs since rehabilitation.

The oldest of the surveyed retrofit edge support projects, on Route 116 in Illinois, is a good example of the technique's benefit. A PCC shoulder was retrofit on the outer lane in the early 1960's, and 22 years later faulting and crack deterioration were both significantly higher in the inner lane than the outer lane (faulting of 0.11 inches versus 0.05 inches and cracking of 412 feet/mile versus 317 feet/mile, respectively), despite the fact that the outer lane carried roughly ten times more traffic over this time.

Three different types of edge support installations were observed on the projects surveyed:

1. Retrofit PCC edge beam,
2. Retrofit PCC shoulder, and
3. PCC shoulder placed with concrete overlay.

In all cases but one, the PCC beam or shoulder was tied to the existing pavement or overlay with deformed rebar (hook bolts were used on one project). The mean and range of values for edge support design variables are shown below:

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
Joint spacing (ft)	JPCP 17.4	13.5 - 25
	JRCP 70.5	41 - 100

Since retrofit edge support was performed concurrently with either diamond grinding or a PCC overlay on most of the edge support projects surveyed, the effect of edge support was included in the predictive models for these techniques. Edge support was shown to have a positive effect in reducing faulting, but cannot be relied upon to prevent faulting if poor subdrainage or poor load transfer exists. In addition to reducing lane edge deflections, tied PCC shoulders and edge beams also provide longitudinal lane/shoulder joints which are considerably easier to seal and maintain than typical PCC/AC junctions. Thus, edge support

restoration may improve pavement performance in a second, indirect way, by significantly reducing water infiltration into the pavement structure through the lane/shoulder joint.

The selection of a PCC edge beam or full-width PCC shoulder is a matter of material costs, condition of the existing shoulder, and geometry. A full PCC shoulder may be preferred when the existing shoulder is badly deteriorated or when it is necessary to put traffic on the shoulder during rehabilitation of the mainline pavement.

Jointed plain concrete is recommended for a PCC edge beam or shoulder. For a full shoulder, the thickness should match the thickness of concrete in the outer traffic lane, although the shoulder may be tapered to a lesser thickness at the outer edge. The shoulder should be at least 6 inches thick at the outer edge, and thicker if heavy truck parking on the shoulder is expected. Edge beams require a somewhat more substantial thickness to provide the same structural benefit as full shoulders. A PCC edge beam should be at least as thick as the traffic lane slab, and at least 24 inches wide. Weakened-plane transverse joints without dowels are recommended for both PCC shoulders and edge beams. All transverse joints in the existing traffic lane should be matched with joints in the shoulder, and additional joints should be placed in the shoulder as needed so that the shoulder joint spacing does not exceed 20 feet.

The retrofit edge should be securely tied to the existing pavement by deformed rebars grouted or epoxied into drilled holes in the slab face. The embedment length must be sufficient to develop the full yield strength of the bar. Tie bars 30 inches long, 0.625 inches in diameter, and spaced 24 inches apart performed well on projects in this study. A corrosion-resistant coating should be used on tie bars in areas of deicing salt use.

Full-Depth Repair

Full-depth repair of joints and cracks consumes a large portion of the funds set aside for concrete pavement rehabilitation. Unfortunately, full-depth repair performance to date has generally been poor. Failures such as settlement, heaving, rocking, faulting, cracking, spalling, and pumping have often been observed within as little as a year after construction. Extensive field and laboratory studies were conducted to investigate the design and construction factors critical to achieving better performance from this important rehabilitation technique.

Design and performance data were collected on 2001 individual full-depth repairs on 125 in-service rehabilitation projects in 22 states. The projects ranged in age from 1 to 15 years and had carried from 0.27 to 7.68 million ESALs since rehabilitation. A variety of different repair designs and joint load transfer designs were represented in the database. A second field study consisted of monitoring an experimental project constructed on Interstate 70 by the Illinois Department of Transportation in 1984, which incorporated a variety of full-depth repair joint designs.

From the field data, observations of typical full-depth repair performance were made and predictive models for full-depth repair faulting and spalling were developed. Faulting was observed to typically be larger at repair leave joints than at approach joints. The following model for full-depth repair leave joint faulting (for diamond-ground projects only) was developed:

$$\text{FDRFAULT} = \text{ESAL}^{0.74} (0.0364 - 0.0292 \text{ BASE}) + 0.275 (\text{AGE} * \text{FI})^{0.019} - 0.283$$

where:

FDRFAULT = full-depth repair leave joint faulting, inches

ESAL = accumulated 18-kip ESALs since placement of repairs, millions

BASE = type of base under PCC slab
= 0, if granular
= 1, if stabilized

AGE = time since placement of repairs, years

FI = mean Freezing Index, Fahrenheit degree-days

$R^2 = 0.41$

n = 113

SEE = 0.048 inch

Full-depth repair approach joints were observed to be generally tighter than leave joints, which may be due to the repairs being shoved backwards by tire traction. Repair approach joints may fault less as a result of being tighter, but the trade-off is that they tend to spall more. The model for full-depth repair approach joint spalling is shown below:

$$\begin{aligned} \text{FDRSPALL} = & \text{ESAL}^{0.071} (0.666 - 0.457 \text{ SEAL}) \\ & + 0.000686 \text{ JTSPACE}^{1.20} + 0.131 \text{ BADAGG} \\ & - 0.227 \text{ JTWIDTH}^{0.463} + 0.0554 \text{ DAMAGE} \\ & + 0.943 \end{aligned}$$

where:

FDRSPALL = repair approach joint spalling
= low severity, if between 0.0 and 1.0
= medium severity, if between 1.0 and 2.0
= high severity, if between 2.0 and 3.0

SEAL = full-depth repair joint sealant
= 0, if repair joint not originally sealed
= 1, if repair joint originally sealed

JTSPACE = original transverse contraction joint spacing, feet

BADAGG = presence of "D" cracking or reactive aggregates in pavement
= 0, if not present
= 1, if present

JTWIDTH = approach joint width measured during survey, inches

DAMAGE = approach joint sealant condition
 = 0, if present and in good condition
 = 1, if missing, failed, or infiltrated by incompressibles

$R^2 = 0.37$
 $n = 1102$ joints
 $SEE = 0.19$

Much of the poor performance of in-service full-depth repairs observed can be attributed to inadequate design and/or construction of the dowel load transfer system. A large laboratory study was conducted to fully investigate factors affecting the performance of dowel load transfer systems for full-depth repairs. [6] Cyclic shear loads were applied to dowels anchored in holes drilled in concrete specimens retrieved from an in-service Interstate pavement, and load and deflection data were recorded at selected intervals during the testing of each dowel.

The effects of five design and construction factor on dowel deflection response were investigated: dowel diameter, annular gap, anchor material, embedment length, and drill type.

A half-fractional factorial experiment design was developed to investigate the individual effects and interactions of the above factors. Nearly 90 test specimens were included in the experiment. A number of special tests were also conducted, including tests on two specimens with dowels cast in place in the lab, two specimens with dowel turned on a lathe to nearly the same size as the drilled holes to obtain a very tight fit, and one specimen with a large-diameter hollow stainless steel dowel.

The loading applied to the dowels was a continuous sinusoidal form with peak magnitudes of 3000 pounds in each direction and a frequency of 6 cycles per second. This loading form was selected to simulate, as much as possible within the mechanical limitations of the testing apparatus, the loading applied to a dowel in an in-service full-depth repair under a 36-kip tandem axle dual-wheel load at highway speed. Most of the specimens were subjected to approximately 600,000 load cycles, although a few received two million or more load cycles. Complete details of the laboratory study procedures and findings are given in Reference 6. Some of the key findings are summarized below.

Epoxy mortar was an easier anchoring material to use and produced more consistent results than the cement grout. When properly installed at the right consistency so that the dowel is fully supported, cement grout produced lower deflections and less dowel looseness than epoxy. However, proper installation with the cement grout was difficult to achieve, even in the laboratory, due to rapid change in the grout's consistency within very short periods of time.

Larger-diameter dowels significantly reduced concrete bearing stresses, as well as dowel deflections and looseness. Five of the 1-inch dowel bars failed in fatigue during testing, before the equivalent of even one year of heavy traffic was applied. In each of these cases, the dowel broke 0.75 to 1.5 inches in from the face of the specimen, which coincides with the theoretical location of maximum moment. Some of the dowels failed after as few as 40,000 load cycles, while others failed after about 600,000 load cycles. Uniformity of support around the dowel at the face of the concrete was critical to the performance of the smaller-diameter dowels: four of the five failed dowels were anchored with cement grout, and three of these had voids visible above the dowel.

Inexpensive reusable nylon or plastic grout retention disks, illustrated in Figure 4, proved very effective in preventing anchor material from flowing out of the drilled hole and providing uniform support around the entire dowel at the face of the concrete. One of these disks was placed on each dowel so that when the dowel was inserted in the hole and the anchor material was forced forward around the dowel, the grout retention disk was

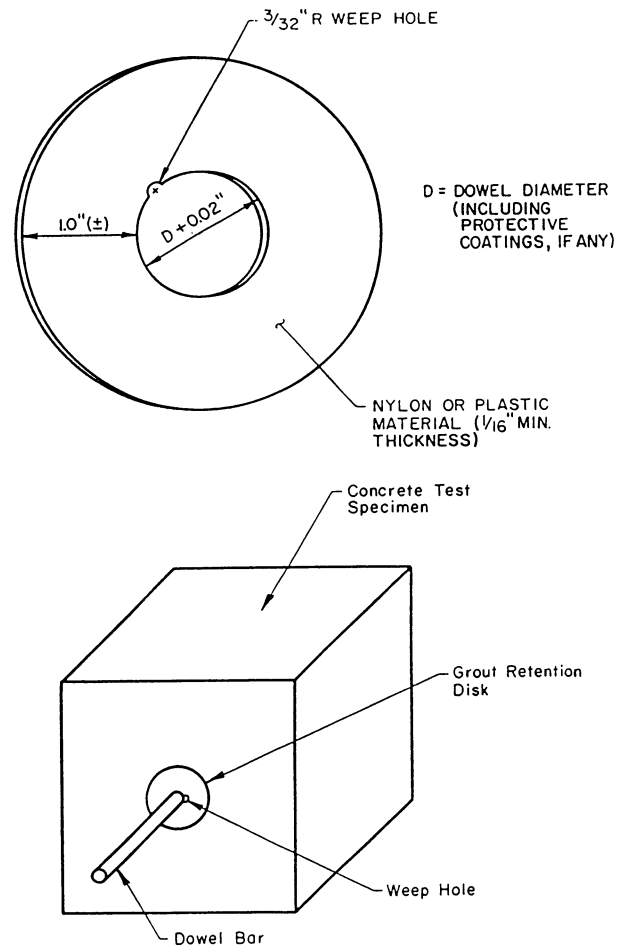


Figure 4. Grout retention disk recommended for full-depth repair dowel installation.

flush with the face of the specimen. The disk was removed once the anchor material had set. It cannot be overemphasized that the use of a grout retention disk is essential to achieving full support of the dowel during full-depth repair installation and maximizing the load transfer performance of the dowel in service.

A larger annular gap worked better with cement grout, perhaps because a larger gap permits movement of the grout around the dowel even when the grout consistency becomes stiffer than ideal. In contrast, a smaller annular gap worked better when epoxy was used as the anchor material. This may be due to epoxy being a much softer material than either the dowel or concrete. A large annular gap creates a thicker layer of epoxy around the dowel, and compression of this layer under loading permits greater dowel deflection and apparent looseness.

Embedment length did not appear to significantly affect dowel deflection and looseness. Only slight increases in deflection were observed with the shorter 7-inch embedment. Higher drill impact energy levels produced more spalling at the faces of the specimens, but did not necessarily result in higher dowel deflections. This may be due to the use of the grout retention disks, which effectively retained the anchor material at the faces of the specimens, filling in spalls caused by the high-impact drills.

The findings of the field and laboratory studies were combined to develop a design procedure for dowel load transfer systems for full-depth repairs. The procedure, which is described in Reference 6, permits a design engineer to determine the number and diameter of dowels needed to limit full-depth repair faulting to a specified maximum level after a specified number of anticipated traffic load applications. Sensitivity analyses of the design procedure indicate that most repair load transfer designs in use today would be quite satisfactory if the dowels were provided good uniform support by the anchor materials. The fact that performance of full-depth repairs to date has been far less than consistently satisfactory suggests that installation methods must be improved to achieve the potential performance of this rehabilitation technique.

Partial-Depth Repair

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. In this study, partial-depth repairs were surveyed on 40 projects in 16 states. The projects ranged in age from 1 to 10 years and had carried from 0.06 to 6.50 million ESALs since rehabilitation. The oldest partial-depth repair projects were located in Virginia, Georgia, Minnesota, and South Dakota.

Partial-depth repairs have performed poorly on many projects; they have performed well on many others. This suggests that, although good long-term performance is achievable with partial-depth, how this performance can be achieved is not well understood. The primary reason that partial-depth

repairs often fail is that they are often used where they are not appropriate. Even when used appropriately, partial-depth repairs may fail if constructed with poor techniques or unsuitable materials. Unless partial-depth repairs are used only for surface distress and constructed well with durable materials, failure within as little as one year is virtually guaranteed. The majority of the project surveyed represent examples of how not to perform partial-depth repair.

A number of examples of excellent partial-depth repair performance were also found. These can be found on projects in several different states, with different traffic levels, pavement designs, repair materials, construction techniques, and concurrent restoration work. In fact, on the projects where partial-depth repairs have been successful, the repairs are generally in such good condition that no long-term effects of traffic or climate are apparent. However, the oldest projects in the database were 10 years old. Although it appears that the lives of the repairs on these successful projects will substantially exceed 10 years, there is not sufficient long-term performance data available to model partial-depth repair performance quantitatively in terms of traffic, climate, and design and construction variables.

Partial-depth repair is strictly removal of small, shallow areas of deteriorated concrete and replacement with a suitable repair material, i.e., a material which is comparable to the concrete in the existing slab in terms of strength, durability, and volume stability. The repair material must bond to sound concrete and become an integral part of the slab.

Partial-depth repairs have been successfully used to repair certain types of concrete pavement distress which are confined to the top few inches of the slab, including:

- Transverse joint spalling caused by intrusion of incompressibles,
- Spalling caused by metal (Unitube) joint forming inserts,
- Scaling due to high reinforcing steel or overfinishing, and
- Early stages of "D" cracking and reactive aggregate distress.

The distresses listed above are not always limited to the upper few inches of the slab and so may not always be correctable by partial-depth repair. Incompressibles may infiltrate transverse joints from the bottom of the slab as well as the top, causing spalling which is not visible at the surface. Spalling caused by Unitubes often results from incompressibles entrapped in the fins, and typically occurs in the top 2 to 3 inches of the joint, though it may extend deeper. Scaling can only be corrected by partial-depth repair 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 water accumulates, and may not be visible at the surface until the bottom of the slab is already severely deteriorated. On pavements with scaling and map cracking caused by reactive aggregates, partial-depth repair may achieve short-term ride quality improvements and provide well formed reservoirs for joint resealing, but it should be recognized that these distresses are the result of progressive fracturing of the cement matrix and probable structural degradation of the concrete. Partial-depth repair cannot halt or effectively correct the cracking and joint damage caused by expansion and consequent compressive stress buildup in reactive aggregate pavements.

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 half of the slab thickness nor to the depth of any reinforcing steel or dowels present. A more conservative limit of one third of the slab thickness has even been suggested. Types of concrete pavement distress which are not likely to be correctable by partial-depth repair include:

- Cracking and joint spalling caused by compressive stress buildup in long-jointed or reactive aggregate pavements,
- Spalling caused by dowel bar misalignment or lockup,
- Transverse or longitudinal cracking caused by poor joint construction (late sawing, inadequate sawcut depth, or inadequate insert placement depth), and
- Working transverse or longitudinal cracks caused by shrinkage, fatigue, or foundation movement.

On any rehabilitation project where partial-depth repair is being considered, coring representative joints and cracks to determine the depth of deterioration is important. Partial-depth repair should not be performed in place of full-depth repair for full-depth distresses.

A wide variety of materials are available for use in partial-depth repair. Materials that have been used most successfully include Type III Portland cement with or without an accelerating admixture, proprietary rapid-setting materials, and epoxy concrete. The only partial-depth repair material observed in this study to perform poorly was high alumina cement concrete, which experienced significant scaling, shrinkage, and debonding. Factors influencing material selection include strength, durability, volume stability, safety and convenience of use, strength gain, past performance record, and cost.

Bonded Concrete Overlay

Bonded concrete overlays have been used as part of experimental and routine resurfacing for many years, particularly for airports. Within the last ten years, however, the practice of resurfacing concrete highway pavements with bonded PCC has increased in popularity. Nineteen bonded concrete overlay projects in 6 states were included in this study. The majority of the projects were located in Iowa. The projects ranged in age from 1 to 12 years and had carried from 0.06 to 4.52 million ESALs. The projects included in the database represent most of the bonded concrete overlay highway projects performed in the United States since 1973.

A bonded concrete overlay may be an effective rehabilitation technique for a pavement in need of a structural improvement or improved ride quality. Many of the sections surveyed in this study performed well. However, several sections performed poorly, mainly due to the extent of deterioration of the existing pavement prior to overlay. Working cracks in the existing pavement which are not repaired prior to overlay will typically reflect through the overlay during the first two years of its life, and eventually progress to about the same severity in the overlay as in the underlying slab.

Slab cracking from structural fatigue after the overlay was placed was not identified on any of the projects except on Route C-17 in Clayton County, Iowa, one of the oldest experimental bonded overlays constructed, on original slabs only 6 inches thick. A bonded overlay normally increases total concrete thickness beyond the point where normal highway traffic loads can cause appreciable fatigue damage.

The predictive model for cracking on bonded overlays is shown below:

$$\begin{aligned} \text{CRACKS} = & 11.328 \text{ ESAL}^{0.07546} \{ 21.426 [\text{AGE} \\ & * (\text{FI} + 1) / 1000]^{0.66876} \} \\ & + \text{ESAL}^{0.002} (378.5 \text{ INDEXM} \\ & + 1257.1 \text{ INDEXH}) \end{aligned}$$

where:

CRACKS = total length of medium- and high-severity deteriorated cracks after bonded overlay, ft/mile

ESAL = accumulated 18-kip ESALs, millions

AGE = time since overlay, years

FI = mean Freezing Index, Fahrenheit degree-days

INDEXM = index for quantity of medium- and high-severity cracking on original pavement prior to overlay:

[JPCP] = 0, cracking $\leq$ 100 or $>$ 500 ft/mile
 = 1, cracking between 101 and 500 ft/mile

[JRCP] = 0, cracking ≤ 200 or >1000 feet/mile
 = 1, cracking between 201 and 1000 ft/mile

INDEXH = index for quantity of medium- and high-severity cracking on original pavement prior to overlay

[JPCP] = 0, cracking ≤ 500 ft/mile
 = 1, cracking > 500 ft/mile

[JRCP] = 0, cracking ≤ 1000 ft/mile
 = 1, if cracking > 1000 ft/mile

$$R^2 = 0.75$$

$$n = 13$$

$$SEE = 326 \text{ ft/mile}$$

Figure 5 shows that cracking of the overlay, as predicted by the model, begins almost immediately after placement and increases rapidly for about a year before it levels off. The most influential factor in occurrence of cracking was the extent of cracking of the original pavement prior to overlay (where the extent of unrepaired cracking in the underlying pavement is indicated by INDEXH, INDEXM, and INDEXL). These results indicate the importance of proper full-depth repair of working cracks and forming joints in the overlay to match the full-depth repair joints.

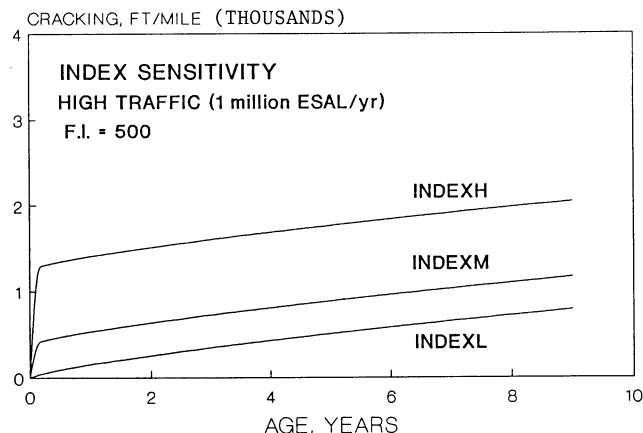


Figure 5. Sensitivity of bonded overlay cracking model to existing cracking, and age/traffic.

Faulting of transverse joints in bonded overlays was not a significant problem on the projects surveyed. The largest average faulting measured on any overlay project was 0.10 inches. The predictive model for bonded overlay faulting is shown below:

$$\begin{aligned} \text{FAULT} = & 0.0015897 \text{ ESAL}^{0.233} [-10.942 \\ & - 30.657 \text{ BASE} + 0.0005652 (\text{FI} + 1)^{2.299} \\ & + 33.322 (\text{DOWEL} + 1)^{-0.8477}] \end{aligned}$$

where:

FAULT = mean transverse joint faulting after overlay, inches

ESAL = accumulated 18-kip ESALs after overlay, millions

BASE = type of base under original PCC slab
 = 0, if granular base
 = 1, if stabilized base

FI = mean Freezing Index, Fahrenheit degree-days

DOWEL = diameter of dowels in original PCC slab, inches (0 if no dowel bars used in original pavement)

$$R^2 = 0.54$$

$$n = 27$$

$$SEE = 0.02 \text{ inch}$$

The bonded overlay projects included in this study were fairly young and had carried only a moderate amount of traffic (less than 4.5 million ESALs to date). Figure 6 shows that the shape of the faulting curve is similar to that expected for new pavements, i.e., rapid initial faulting which levels off. Dowels in the existing pavement reduce faulting of the overlay. Faulting on a bonded overlay is expected to be somewhat greater than on a new pavement, probably due to some loss of load transfer of the existing pavement joints. Thus, when placing a bonded overlay over an existing pavement that has significant faulting, consideration should be given to improving subdrainage and performing load transfer restoration.

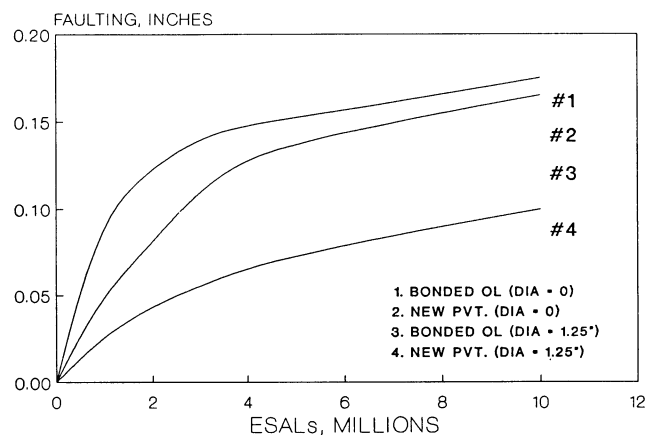


Figure 6. Predicted bonded overlay faulting versus new pavement faulting, with and without dowels in the existing pavement.

Secondary joint cracking (from reflection of joints in the underlying slab) is a significant distress on bonded concrete overlays. However, the many projects surveyed that had no secondary joint cracking are evidence that construction techniques are available to prevent this distress. Critical factors in preventing secondary joint cracking are determining the proper sawcut location and sawing completely through the overlay as soon as possible after concrete placement.

Debonding testing was not performed, but corner cracking on a few projects indicates that some debonding has occurred. This is a potentially

serious problem and must be considered in the design and construction phase. One project that had serious "D" cracking prior to overlay had developed a significant amount of cracking in the overlay, indicating probable disintegration of the underlying slab and debonding of the overlay.

Unbonded Concrete Overlay

Unbonded concrete overlays achieve a separation between the new overlay and the existing slab to eliminate reflection cracking. This type of overlay has been a part of routine resurfacing work for many years. Fourteen unbonded concrete overlay projects in 6 states were surveyed in this study. The projects ranged in age from 1 to 10 years and had carried from 0.38 to 6.7 million ESALs. The performance of all of these sections has generally been very good. Faulting, pumping, cracking, and joint deterioration were all very low.

Thermal curling stresses are significant in unbonded overlays because the temperature gradient through the overlay slab may be very large during many days and nights of the year, and also because of the stiff support of the underlying slab. No predictive model for unbonded overlay cracking could be developed since no cracking was observed. Data from one project indicated that an overlay joint spacing of 30 feet resulted in transverse cracking, but a joint spacing of 15 feet did not. Several projects with joint spacings of 15 feet or less had no cracking.

Faulting of unbonded overlays surveyed was very low. The following predictive model was developed for unbonded overlay faulting:

$$\text{FAULT} = 0.28615 \text{ ESAL}^{0.39654} [0.0987 \\ * (\text{DOWEL} + 1)^{-0.51083}]$$

where:

FAULT = mean transverse joint faulting after overlay, inches

ESAL = accumulated 18-kip ESALs since overlay, millions

DOWEL = diameter of dowels in overlay, inches (0 if no dowel bars used)

$$R^2 = 0.51$$

n = 23 sample units

SEE = 0.02 inch

Faulting of an unbonded overlay without dowels was found to be substantially less than for new pavements without dowels, as shown in Figure 7. Dowelled unbonded overlays fault about as much as new pavements with dowels (very little). Dowels are believed to be required for short-jointed JPCP unbonded overlays under heavy traffic (more than 10 million design ESALs), and also must be used in long-jointed JRCP unbonded overlays.

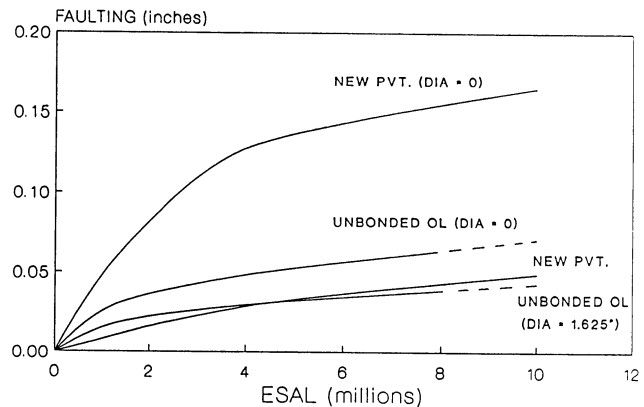


Figure 7. Predicted unbonded overlay faulting versus new pavement faulting.

USE OF MODELS FOR ESTIMATING REHABILITATION LIFE

The predictive models developed in this study can be used to estimate the approximate life of any project under design. The EXPEAR computer program was developed specifically for this purpose, but the models could also easily be entered into a spreadsheet. As an example, a 0.9-mile section of JRCP located on I-74 in Illinois was under consideration for restoration. The pavement was 29 years old when surveyed in 1986. Design, construction and traffic data needed in the predictive models were collected, and a distress survey was conducted. The outer lane pavement was in considerably worse condition than the inner lane, as shown below:

Distress/PSR	Outer Lane	Inner Lane
Pumping	Low	None
Faulting	0.27 in	0.13 in
Det. Cracks	760 ft/mile	707 ft/mile
Det. Joints	21 per mile	21 per mile
PSR	2.6	3.1

Based upon the condition, the following restoration techniques and quantities were selected for the traffic lanes and shoulders (0.9 miles):

Repair Type	Outer Lane	Inner Lane	Shoulders
Full-depth repair	608 sy	576 sy	8448 sy PCC
Reseal center jt	4752 ft		
Reseal trans jts	342 ft	342 ft	
Reseal lane/sh jts			9504 ft
Subseal jts	78 cf		
Level-up settlement	267 sy	267 sy	
Install subdrains	4752 ft	4752 ft	
Diamond grinding	6336 sy	6336 sy	

The models were used to predict the performance of these restoration techniques, and the results are shown in Table 2. The results show that if the pavement had been restored in 1986, distress and roughness would have reached unacceptable levels again within 9 years (PSR = 2.6, crack deterioration = 1073 feet/mile, and full-depth repair

joint faulting = 0.21 inch). These results are not too surprising, since the pavement would be 38 years old in 1995. A cost analysis could be performed to compare this strategy with other alternatives, such as overlay or reconstruction.

Table 2. Predicted performance of I-74 example after restoration.

YEAR	AGE	CUMULATIVE ESALs	JOINT FAULTING	FDR FAULTING	CRACKING (NEW)	JOINT DETERIOR.	PUMPING	PSR
1986	29	13.00	0.00	0.00	0.0	0.0	0.0	4.5
1987	30	13.76	0.04	0.05	82.1	0.9	0.2	4.1
1988	31	14.55	0.05	0.08	171.6	1.7	0.4	3.9
1989	32	15.37	0.07	0.10	269.2	2.7	0.5	3.7
1990	33	16.23	0.08	0.12	378.6	3.7	0.6	3.5
1991	34	17.11	0.09	0.14	494.5	4.7	0.7	3.3
1992	35	18.04	0.10	0.15	621.0	5.8	0.8	3.2
1993	36	19.00	0.11	0.17	758.8	6.9	0.9	3.0
1994	37	20.00	0.12	0.19	909.2	8.0	1.0	2.8
1995	38	21.04	0.12	0.21	1073.2	9.2	1.1	2.6
1996	39	22.12	0.13	0.22	1252.0	10.5	1.2	2.4
1997	40	23.24	0.14	0.24	1447.1	11.8	1.3	2.3
1998	41	24.41	0.15	0.26	1659.8	13.2	1.4	2.1
1999	42	25.63	0.16	0.28	1989.7	14.6	1.5	1.8
2000	43	26.89	0.16	0.29	2243.3	16.0	1.6	1.6
2001	44	28.21	0.17	0.31	2519.9	17.5	1.7	1.4
2002	45	29.58	0.18	0.33	2821.5	19.1	1.8	1.2
2003	46	31.00	0.18	0.35	3150.5	20.7	1.9	0.9
2004	47	32.48	0.19	0.37	3509.3	22.4	2.0	0.7
2005	48	34.02	0.20	0.39	3900.5	24.1	2.2	0.4

18-kip
millions

Inches

Inches

Feet
per
mile

Joints
per
mile

CONCLUSIONS AND RECOMMENDATIONS

This paper describes the results of the most comprehensive evaluation of in-service concrete pavement rehabilitation techniques ever conducted in the United States. The findings concerning the performance of each technique are summarized as follows.

1. Diamond grinding has been successful in improving ride quality and extending the service life of jointed concrete pavements. About 20 percent of the sections surveyed had structural deficiencies indicated by extensive cracking, and were probably not suitable candidates for restoration. Grinding does not stop faulting. The rate of recurrence of faulting after grinding is higher than the rate of faulting on new pavements, unless steps are taken to improve subdrainage, edge support and/or load transfer of the joints.

2. Load transfer restoration is a promising technique for improving joint performance. Retrofit dowel bars are very reliable and by far the most effective of the several types of devices available for load transfer restoration. A great need exists for more efficient installation procedures to reduce the cost of this technique.

3. Edge support restoration reduced transverse joint faulting about nine percent. One project in service for 22 years showed a dramatic reduction in faulting due to retrofit shoulders. A good tie system between the shoulder and traffic lane is essential to achieving the full benefit of edge support restoration. PCC shoulders and edge beams themselves perform very well for up to 20 years with little deterioration.

4. Full-depth repair is perhaps the most critical rehabilitation technique, and its performance to date has been inconsistent, largely due to inadequacies in design and construction of the

joint load transfer system. Results of the field and laboratory studies have produced guidelines for improving full-depth repair dowel installation procedures. Key recommendations include dowels of larger diameter (1.25 to 1.5 inches), at least 4 to 5 dowels per wheel path, proper anchoring in the existing slab using a grout retention disk, and sawing a reservoir and sealing the transverse repair joints.

5. Partial-depth repair performance has also been inconsistent. Two key factors influencing its success are appropriateness for the distress present and adequacy of construction techniques, materials and quality control. Partial-depth repairs of many different material types have provided excellent performance when used to repair surface distresses that do not extend below the top third of the slab depth, but have often failed in as little as one year when applied to working cracks and other major distresses that should have been corrected by full-depth repair.

6. Bonded concrete overlay performance depends primarily upon two factors: extent of cracking in the existing slab, and adequacy of the bond achieved. Working cracks must be full-depth repaired and joints must be formed in the PCC overlay over the repair joints. A full bond must be achieved through proper cleaning, bonding material application and curing procedures. When this is accomplished, bonded overlays show very good performance.

7. Unbonded concrete overlays have given very good performance on practically all projects surveyed. They typically exhibited very little distress even after a large number of traffic loadings. The high initial cost of an unbonded overlay may be in many cases more than offset by the low-maintenance performance of this technique. Transverse joint spacing for plain overlays must be short, 15 feet or less. Faulting was minimal for unbonded overlays.

Selection of appropriate rehabilitation techniques require a careful and thorough evaluation of the pavement and identification of the mechanisms causing distress. The life of a rehabilitation strategy such as restoration depends on the performance of each rehabilitation technique that is included (e.g., full-depth repairs, grinding, partial-depth patching). Each technique must be properly designed and constructed. Maximum cost-effectiveness of pavement rehabilitation can only be achieved by application of appropriate rehabilitation measures.

Advances constantly being made in the design and construction of specific techniques will advance the state of the art of pavement rehabilitation and benefit the pavement management efforts of state transportation departments. The Design and Construction Guidelines provided in References 1 and 2 include detailed information on the use of the techniques described in this paper.

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The results reported in this paper concerning the field and laboratory analyses of full-depth repair design and performance are based on the Ph.D. thesis research of Mark B. Snyder. The bonded and unbonded concrete overlay performance findings are based on the M.S. thesis research of Gerald F. Voigt. The edge support performance findings are based on the M.S. thesis research of Michael J. Reiter.

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