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The contents of this paper reflect the view of the authors who are responsible for the facts and the accuracy of the data presented herein.

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Performance Evaluation of Jointed Concrete Pavement Rehabilitation Without Resurfacing

David L. Lippert

The results of a research project that was conducted to evaluate the effectiveness of concrete pavement restoration (CPR) on jointed portland cement concrete pavement are described. The CPR methods evaluated were pavement grinding, grout undersealing, installing underdrains, retrofitting double-vee load transfer devices, and pavement patching. Five construction sections, located on Interstates in Illinois, were selected for evaluation. The original pavement sections were constructed between 1960 and 1963, then rehabilitated in 1983 and 1984. All pavements were of the same design: a 10-in. slab over a 6-in. granular base and a joint spacing of 100 ft. The evaluation began just before rehabilitation of each section and continued until May 1986. Evaluation was done using crack surveys, destructive testing, and nondestructive testing. Performance factors monitored were faulting, pavement cracking, pavement roughness, skid resistance, deflection, load transfer, void development, and drainage. A great deal of emphasis was placed on grout undersealing and doweled patching in laboratory and field experiments. Effectiveness of undersealing was determined by deflection testing using a Dynatest 8002 falling weight deflectometer and a Road Rater 2008. Another field experiment was conducted to investigate the effects of dowel bar size and number of dowels on full-depth patch performance. Several different techniques for dowel bar grouting were tested in the laboratory to establish grouting procedures. The findings of this research resulted in improvements in full-depth patch design, improved construction procedures, and proper use of undersealing.

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The state of Illinois has many miles of highways composed of jointed portland cement concrete (JPCC). Many of these pavements have approached or are approaching the end of their design life and are in need of major rehabilitation.

Resurfacing with asphalt concrete (AC) is one of the primary methods of rehabilitation used in Illinois. However, asphalt concrete overlays may not be cost-effective for a JPCC pavement that has faulted joints, transverse cracks, and possibly some spalling but is otherwise sound. On this type of pavement, it is possible that rehabilitation without resurfacing can be much less expensive and more cost-effective.

The main objective of this study was to determine whether pavements with faulted joints and transverse cracks or general joint deterioration can be restored by grinding, pressure grouting, placement of underdrains, retrofitting with load transfer devices, or replacement of the joints more economically over the long run than by resurfacing. Five rehabilitation projects were monitored for performance by studying such indicators as ride quality, faulting, and pavement deflection.

Initial poor performance of doweled patches resulted in a pavement patching experiment. Such variables as number of dowel bars, size of dowel bars, and grout type were investigated. Also included in the experiment were sawed and sealed joints at the patch-pavement interface. Patching performance was monitored by measuring pavement deflection, faulting, and patch distress.

Also of interest was the effectiveness of grout undersealing. Two areas of undersealing were investigated. First, undersealing of joints and cracks to fill voids (*1*) and, second, the filling of voids to stabilize rocking and pumping patches. Undersealing was evaluated using Road Rater deflection testing and void detection procedures (2) using a falling weight deflectometer (FWD).

In the laboratory the problem of proper dowel bar grouting was investigated. This was done by using several methods of applying the grout at different consistencies. The methods were then rated subjectively with respect to grout coverage, ease of use, and cost.

PAVEMENT EVALUATION SECTIONS

Five projects, which were located on Interstate routes 55, 70, 80, and 280-74 were evaluated. All pavement sections consist of 10-in.-thick jointed reinforced concrete pavement on a 6-in. granular subbase. Joints are spaced 100 ft apart and include $1^{1}/_{4}$ in. × 18 in. dowel bars spaced at 12 in. for load transfer. Figure 1 shows the general location of the projects.

Each rehabilitation project was designed to address such problems as ride quality, joint deterioration, filling of voids, and restoring load transfer. Ride quality was improved by diamond grinding of faults, partial-depth pavement patching, and full-depth pavement patching of deteriorated joints. Voids were filled by grout undersealing and load transfer was restored by use of the Double Vee load transfer devices developed at the University of Illinois. The Double Vee device used is shown in Figure 2.

Rehabilitation features such as experimental undersealing, experimental patching, load transfer devices, and drainage mats are not included in the economic analysis because of the smaller quantities associated with these experiments. Summaries of the original pavement designs and rehabilitation techniques used are given in Tables 1 and 2, respectively.

Project 1

This project is located on I-55 near Springfield, Illinois, between mileposts 98.00 and 102.36 and has two lanes in both directions. Traffic on this project averages 11,500 vehicles per day with 26 percent trucks. The pavement has a low to medium D-cracking aggregate, which was evident at joints and cracks at the time of rehabilitation. The main distress types were medium-severity joint faulting, medium- and high-severity joint deterioration (due to D-cracking), and poor ride due to faulting and joint distress.

Rehabilitation first took place in 1983 and again in 1984. The 1983 rehabilitation was in the form of full-depth patching using a 3-2 patch design, as shown in Figure 3, to replace deteriorated joints. Pipe underdrains were installed to facilitate removal of water that may have been causing faulting. In 1984 a number of poorly performing patches needed to be replaced as well as a few joints that had further deteriorated since 1983. Pavement patching in 1984 used the 3-2 patch design in the northbound lanes and the 3-3 patch design in the southbound lanes, as detailed in Figure 3. Also part of the 1984 rehabilitation was blanket undersealing of existing patches.

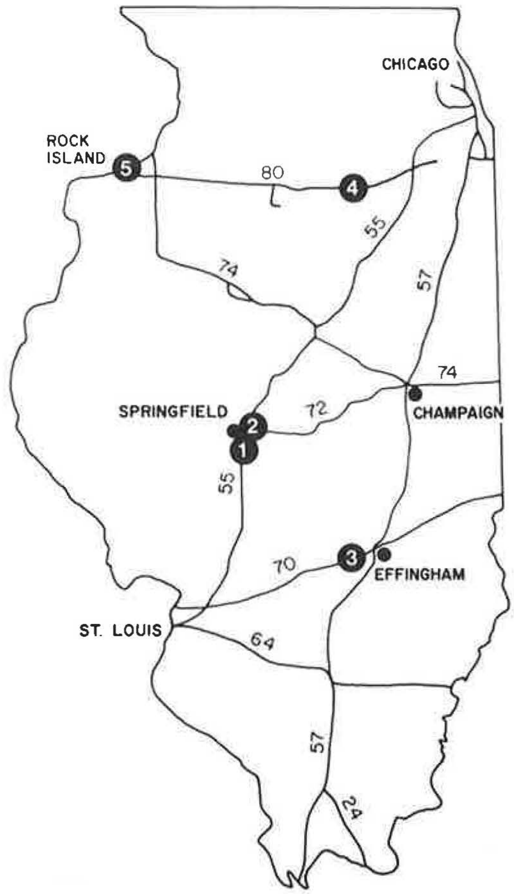

FIGURE 1 Project location map.

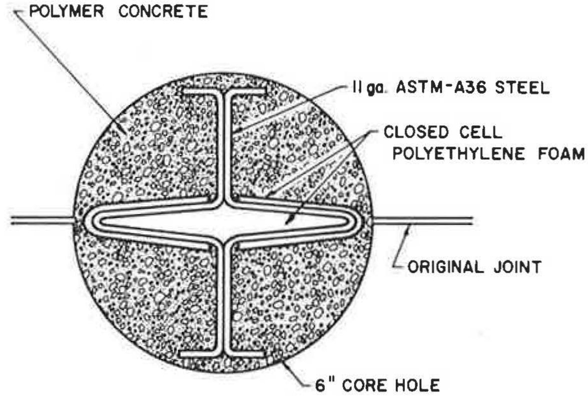

FIGURE 2 Double Vee load transfer device.

TABLE 1 ORIGINAL PAVEMENT DESIGN

Project	Route and Milepost Location	Year Constructed	PCC Pavement Thickness (in.)	Joint Spacing (ft)	PCC Aggregate Quality	Subbase Thickness and Type	Shoulder Design	Pavement Distress[a] at Time of Rehabilitation
1	I-55 98.00–102.36	1962	10.0	100	Fair (moderate D-cracking)	6 in. granular	0.5-in. surface treatment on granular base[b]	MS joint faulting MS & HS joint deterioration Rough ride
2	I-55 102.36–105.52	1962	10.0	100	Very good	6 in. granular	0.5-in. surface treatment on granular base[b]	HS joint faulting Rough ride MS high reinforcing steel spalling Limited MS joint deterioration
3	I-70 74.28–82.23	1960–1963	10.0	100	Good (low D-cracking)	6 in. granular	3-in. bituminous concrete on granular base	MS joint deterioration Moderate joint faulting Rough ride
4	I-80 105.30–111.70	1960	10.0	100	Very good	6 in. granular	3-in. bituminous concrete on granular base	HS joint faulting MS joint deterioration Rough ride
5	I-280 14.70–18.0 I-74 5.00–9.64	1962	10.0	100	Very good	6 in. granular	0.5-in. surface treatment on granular base	LS joint faulting Poor skid properties MS joint spalling

[a]Distress ratings: LS = low severity, MS = medium severity, HS = high severity.
[b]Surface treatment replaced with 3-in. bituminous concrete in early 1970s.

Project 2

This project is also located on I-55 near Springfield, Illinois, and is immediately north of Project 1. This section is between mileposts 102.36 and 105.52 and has two lanes in the northbound direction and three lanes in the southbound direction that taper into two lanes near the southern end of the project. The traffic is the same as that on Project 1.

The pavement showed no signs of a D-cracking aggregate at the time of rehabilitation; there were only localized distress and spalling at a limited number of joints. The main distress types were high-severity joint faulting, limited medium-severity joint spalling, limited low-severity joint deterioration, medium-severity spalling and scaling due to high reinforcing steel, and poor ride quality due to joint faulting.

The main feature of this rehabilitation project was removal of joint faulting by diamond grinding to improve the ride of the pavement. Other features were full-depth patching using a 3-2 patch design, partial-depth patching to remove areas of spalling and scaling due to high reinforcing steel, and installation of pipe underdrains. Included in the rehabilitation was a grout undersealing experiment (*1*). Because of the experimental nature of the undersealing, it was not included in the economic analysis. A more detailed account of the undersealing experiment is given in the section on Undersealing.

Project 3

This project is located on I-70 near St. Elmo and Altamont, Illinois, from mileposts 74.28 to 82.23. This section of highway consists of two lanes in each direction. Traffic on this project

TABLE 2 REHABILITATION TECHNIQUES USED ON PROJECTS

Project	Route	Year of Rehabilitation	Full-Depth Patching	Partial-Depth Patching	Joint Sealing	Blanket Under-sealing	Retrofit Load Transfer Devices	Pavement Grinding	Under-drains
1	I-55	1983	X	–	–	–	–	–	X
		1984	X	–	–	X	–	–	–
2	I-55	1983	X	X	–	X[a]	–	X	X
3	I-70	1983	X	–	–	–	–	–	–
		1984	X[a]	–	X[a]	X	–	–	–
4	I-80	1983	X	–	–	X	X[a]	X	X
		1985	X	–	X[b]	–	–	–	–
5	I-280 I-74	1984	X	X	X	X	–	X	X

NOTE: Dash means technique not used.
[a]Experimental (not included in economic analysis).
[b]Patches only (not included in economic analysis).

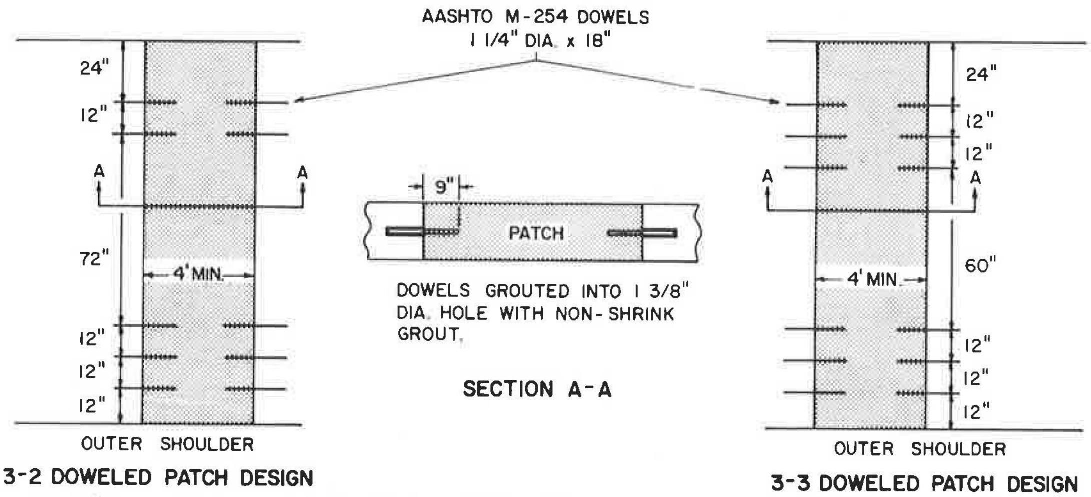

FIGURE 3 Patch design details of 3-2 and 3-3 patches.

averages 10,500 vehicles per day with 36 percent trucks. The aggregate in this pavement has low-severity D-cracking, which was evident at joints before rehabilitation. The main types of distress in the pavement at the time of rehabilitation were medium-severity joint deterioration, medium-severity joint faulting, and poor ride.

In 1983 the pavement was first rehabilitated by full-depth patching of deteriorated joints and cracks using the 3-2 doweled patch design. The following spring a number of these patches began to experience deep spalling and pumping. In a few cases, the spalling was so deep that the dowel bars were exposed. As a result, blanket undersealing of existing patches was undertaken in 1984. Several poor, fair, and excellent performing patches were removed to investigate the cause of the deep spalling and why it did not occur in all patches. It was found that the concrete was being overstressed; this was evidenced by large egg-shaped holes around the dowel bars. No evidence of the nonshrink grout used to install the dowel bars could be found in the dowel holes. As a result, a doweled patching experiment was conducted to investigate the impact of dowel bar size, bar arrangement, grout type, and joint sealing on patch performance. This study is detailed in the section on Full-Depth Patching.

Project 4

This project is located near Morris, Illinois, on I-80 between mileposts 105.30 and 111.70. This section of highway has two lanes in each direction. Traffic on this project averages 13,300 vehicles per day with 32 percent trucks. At the time of rehabilitation, the pavement showed no signs of a D-cracking aggregate. The main distresses were medium-severity joint spalling, medium-severity localized distress, and high-severity faulting. Because of the faulting, this section of pavement had a rough ride.

In 1983 this section underwent an ambitious rehabilitation that included full-depth patching using a 3-2 patch design, blanket undersealing of cracks and joints, retrofitted Double Vee load transfer devices, pavement grinding, and installation of underdrains. Another experimental feature of this project, in addition to the load transfer devices, was the use of a drainage mat developed by Monsanto in the place of standard pipe underdrains on a portion of the project.

As a result of poor patch performance, similar to that of Projects 1 and 3, several patches were replaced in 1985 using 10 dowel bars in each joint rather than the 5 or 6 used previously.

Project 5

This project is located on I-280 and I-74 near Rock Island, Illinois, between mileposts 14.70 and 18.00 on I-280 and mileposts 5.00 and 9.64 on I-74. Traffic on the I-280 section averaged 16,400 vehicles per day with 19 percent trucks, and the I-74 section averaged 8,700 vehicles per day with 25 percent trucks. This section of highway has two lanes in each direction and showed no sign of a D-cracking aggregate. Distress before rehabilitation consisted of medium-severity joint spalling, medium-severity localized distress, and low-severity faulting.

The rehabilitation of this section consisted of full-depth patching using a 3-3 patch design, partial-depth patching, joint sealing, blanket undersealing, pavement grinding, and installation of underdrains.

PERFORMANCE OF REHABILITATION TECHNIQUES

Pavement Grinding

Projects 2, 4, and 5 used pavement grinding to improve the riding qualities of the pavement. To remove the severe faulting in Projects 2 and 4, the pavement was ground in the opposite direction of traffic. This was done to better utilize the leveling properties of the grinding machine and resulted in a smoother profile. Project 5 was ground in the direction of traffic because faulting on this project was minor. A limited amount of friction and faulting data was collected before and after rehabilitation.

TABLE 3 FRICTION NUMBER HISTORY OF GRINDING PROJECTS

		Friction Number in						
Project	Route	1980	1981	1982	1983	1984	1985	1986
2	I-55	–	44	–	–	46[a]	41	–
4	I-80	35	–	–	48[a]	40	–	40
5	I-280 and I-74	–	–	–	–	–	42[a]	–

NOTE: Dash means data not available.
[a]After grinding.

The friction and faulting data are given in Tables 3 and 4, respectively. Roadmeter readings were taken before rehabilitation and annually thereafter. The roadmeter data are given in Table 5. From the roadmeter data, predictions of the roughness index (RI) were made by plotting the data on log-log paper and extending a best fit line through the points. The results are shown in Figure 4. It is expected that diamond grinding will provide a smooth ride for about 5 years after construction and an acceptable ride for another 5 years.

In isolated areas of Project 2, high reinforcing steel caused spalling and scaling of the pavement surface. Known areas of high steel distress were partial-depth patched in an effort to eliminate the problem. The grinding process removed only a fraction of an inch from these areas but may have caused the spalling to accelerate in areas not patched. Although grinding a pavement with a few isolated areas of high steel should not present much of a problem, caution should be used in grinding a pavement with uniform high steel.

Pavements with D-cracking aggregate should not be ground because this opens the aggregate to water intrusion and can greatly accelerate the D-cracking process. Unfortunately, in Illinois, many miles of pavement have low to moderate D-cracking aggregates and, therefore, care must be taken in selecting pavements to be diamond ground.

Retrofit Double Vee Load Transfer Devices

Project 4 was the only project to use the retrofit load transfer devices shown in Figure 2. Several problems were encountered in the installation of these devices. First, the contract did not call for grooving or roughing the inside of the core hole before installing the device. The manufacturer of the load transfer device provided a tool that applied several grooves to the inside of the core hole. Grooving of the holes provided more surface area for the polymer concrete to bond. Because the contract did not require the grooving, only a small number of the core holes were grooved. These locations were noted for comparison.

The second problem was the polymer concrete used. Improper mixing resulted in a mixture that did not take a final set for several days. When mixed properly, the polymer concrete was almost unworkable. Prewetting the coarse aggregate resulted in an acceptable product.

TABLE 4 AVERAGE JOINT FAULT HISTORY OF GRINDING PROJECTS (in.)

		Year Measured			
Project	Route	1983	1984	1985	1986
2	I-55	0.31	0.00[a]	0.03	0.06
4	I-80	0.18	0.00[a]	0.04	–
5	I-280 and I-74	–	0.00[a]	0.01	–

NOTE: Dash means data not available.
[a]After grinding.

TABLE 5 ROADMETER HISTORY OF REHABILITATION PROJECTS (in./mi)

		Year Tested				
Project	Route	1982	1983	1984	1985	1986
1	I-55	–	129[a]	–	–	113
2	I-55	–	165[a]	73	66	78
3	I-70	104[a]	–	–	–	112
4	I-80	151[a]	51	73	64	82
5	I-280 and I-74	–	–	82[a]	40	47

NOTE: Dash means not tested. The following ranges are used to rate ride quality:

RI (in./mi)	*Quality*
Under 75	Very smooth
76–90	Smooth
91–125	Slightly rough
126–170	Rough
Over 171	Very rough

[a]Before rehabilitation.

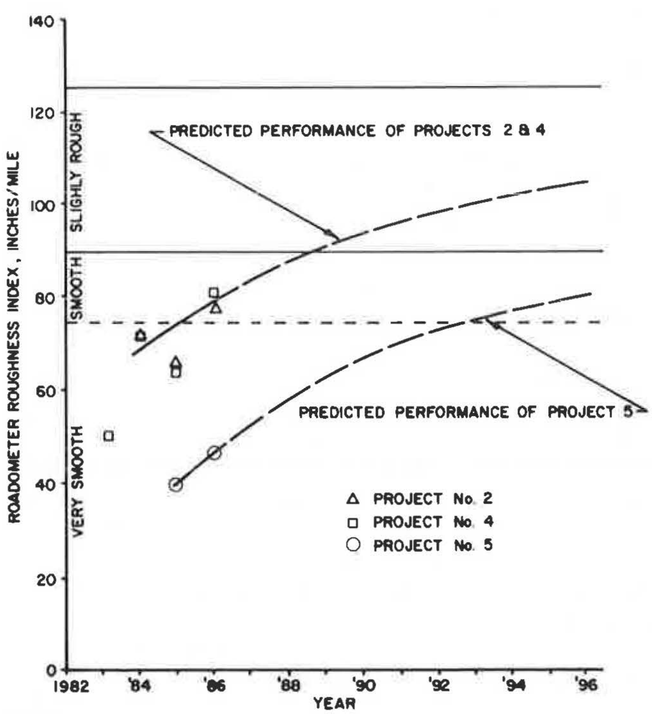

FIGURE 4 Predicted roadmeter roughness index.

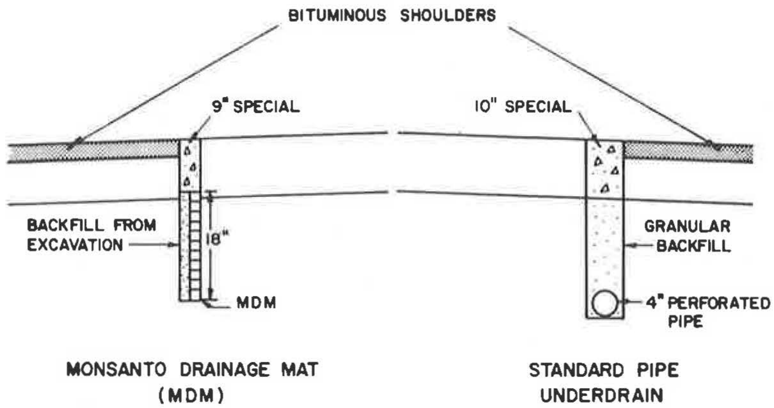

FIGURE 5 Details of Monsanto drainage mat and standard pipe underdrains.

Within a few months of installation, more than 50 percent of the devices debonded at the pavement-polymer concrete interface. There appeared to be no pattern to the debonding in that it occurred on either or both sides of the joint or crack. The grooving of the core hole had no influence on debonding. By 1985 all 671 devices installed have failed by debonding.

Underdrains

Underdrains were installed on all projects except Project 3, in which underdrains had been installed at the pavement edge in previous construction.

On Project 4, a new drainage product was included, the Monsanto drainage mat (MDM). Figure 5 shows a comparison of the design of standard pipe underdrains and the MDM. Sections of the pipe underdrains and MDM were monitored by use of a tipping bucket device that measures outflow with time. Figure 6 shows the typical outflow characteristics of the two underdrains.

The outflow characteristics of the MDM drain showed a desirable improvement in two areas. First, the MDM removed 1.11 to 1.87 times the water of the standard pipe underdrain. Second, the MDM removed the water from the pavement faster, whereas the standard drain continued to flow for several days.

Since first installed on this project, it has been found that a 12-in. mat will perform as well as the standard pipe underdrains. The reduction in mat size, along with the elimination of the granular backfill, has made the MDM quite competitive with pipe underdrains in cost. More long-term research is planned to determine the effectiveness of these and other types of underdrains.

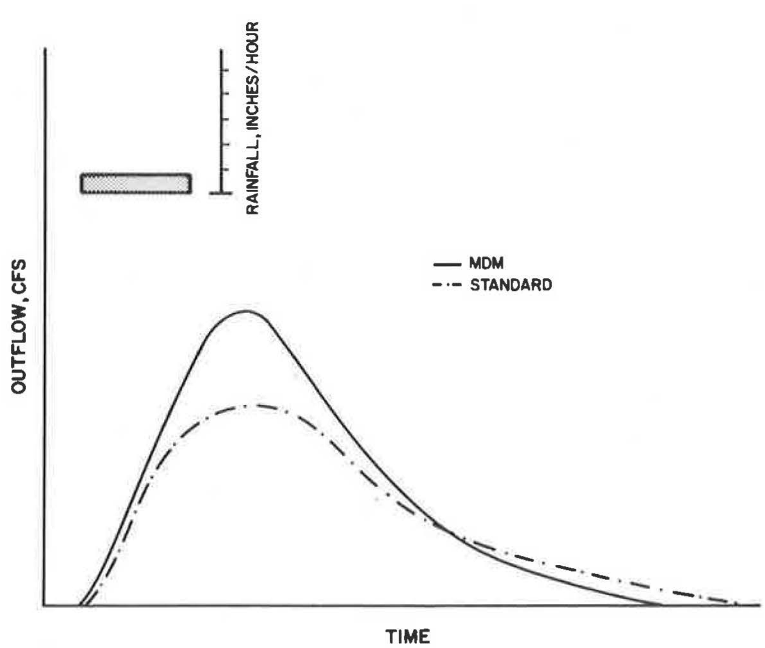

FIGURE 6 Typical outflow characteristics of Monsanto drainage mat and standard pipe underdrains.

Partial-Depth Patching

Projects 2 and 5 used partial-depth patching to repair spalls at joints, and in Project 2 it was used to repair areas of high steel that were exposed. Patches on both projects are performing exceptionally well and are expected to last the life of the pavement. The only problem found was that more areas should have been repaired using partial-depth patching on both projects. Since rehabilitation, several areas of spalls at joints on Project 5 and spalls due to high steel in Project 2 have shown up. Because these areas were outside the condition surveys, it is not certain if any visual evidence of distress was present at the time of rehabilitation. If these areas showed no visual distress, perhaps delamination detection techniques such as those used on bridge decks could be used to detect delaminations at joints.

Full-Depth Patching

All projects included full-depth patching with the percentage of patching ranging from 0.2 to 4.5 percent of the total pavement area. Projects 1, 2, 3, and 4 were patched in 1983 using a 3-2 patch design as detailed in Figure 3. The following spring (1984), Projects 1 and 3 experienced severe spalling of a number of patches. Along with the spalling, evidence of pumping could be seen. When heavy trucks passed over the patches, a rocking motion could be visually detected.

Detailed surveys of patch performance on Project 3 showed that about an equal number of patches fell in the good, fair, and poor categories. There was no apparent correlation between cut, fill, well-drained or poorly drained areas and patch performance. The spalling was limited to the new patch and rarely occurred in the original pavement. Several concrete cores were taken in good, fair, and poorly performing patches, but testing showed no significant strength differences in the samples.

The survey showed that the approach side of the patches had the greatest amount of spalling. On closer investigation, it was found that the approach joint of the patch (first joint to be crossed by traffic) was typically tight and, in spalled areas, the joints were closed. The opposite or leave joint would be 1/16 to 1/8 in. wide with little or no spalling.

As a result of problems with the 1983 3-2 patches, an extra dowel bar was placed in the inner wheelpath for 1984 construction as detailed by the 3-3 patch design in Figure 3. Project 5 used a 3-3 patch design and also included a joint seal at all joints. The 3-3 patch design was also used on Project 1 (southbound lanes only) in 1984 when a few badly distressed patches were replaced.

To better understand the effects of dowel bar arrangement, size, and grout used in dowel bar holes, a patching experiment was incorporated in Project 3 in 1984. The experiment called for removing 28 patches from good, fair, and poor performing groups. All patches were selected in the driving lanes of the roadway; 8 patches were in the westbound driving lane and 20 patches were in the eastbound driving lane.

Because the patches that were to be removed were 4 ft long, sawing requirements were such that the new patch would have to be 6 ft long. Patches were removed using the lift-out method. As patches were removed, the dowel bars were found to be lying loose in an egg-shaped hole. The holes measured about 1 3/8 in. horizontally, as drilled, but would be larger in the vertical direction. The egg-shaped hole occurred in both the old pavement and the patch. No evidence of the nonshrink grout could be found in any of the patches. Also, the epoxy coating used on dowel bars to prevent corrosion was chipped off or debonded for several inches near the center of the dowel bars. All evidence indicates that the concrete was being overstressed at the dowel bars in 3-2 patches with 1 1/4-in. dowel bars.

Variables in the experiment were selected such that some patch designs would be underdesigned and some overdesigned. The patching experiment investigated two dowel bar sizes, three dowel bar arrangements, tied joints, nonshrink grout, and an epoxy grout. The experimental features are given in Table 6 by number of patches constructed in each design variable. Another feature of the experiment was transverse joint seals that were included on all patches, as well as undersealing the old pavement near the patch joint. Design details of experimental patching are shown in Figure 7.

Evaluation of the patches was by visual inspection and deflection testing using an FWD. The visual inspection indicated that all of the experimental patches were in excellent condition, and there was no indication of spalling or any other distress. Deflection testing results are present in terms of a deflection due to a 9-kip load and load transfer across the patch joints in the outer wheelpath. Load transfer is measured by dividing the deflection on the unloaded side of a joint by the deflection on the loaded side and is reported as a percentage. Testing was conducted before the experimental patches were undersealed and again after undersealing before the roadway was opened to traffic. After the roadway was opened to traffic the patches were tested periodically. Deflection and load transfer results for dowel bar size are shown in Figures 8 and 9, and

TABLE 6 NUMBER OF PATCHES CONSTRUCTED BY VARIABLE IN PATCHING EXPERIMENT

	Dowel Design					
	3-3	4-4		5-5		Tied[a]
Dowel Size (in.)	Nonshrink Grout	Nonshrink Grout	Epoxy Grout	Nonshrink Grout	Epoxy Grout	Nonshrink Grout
1.25	4	3	1	4	1	3
1.50	4	4	–	4	–	–

NOTE: Dashes mean not constructed. All patch lengths are 6 ft.

[a]Tied patch design consists of No. 8 tie bar on approach joint and 1.25-in. dowel bar on leave side of patch.

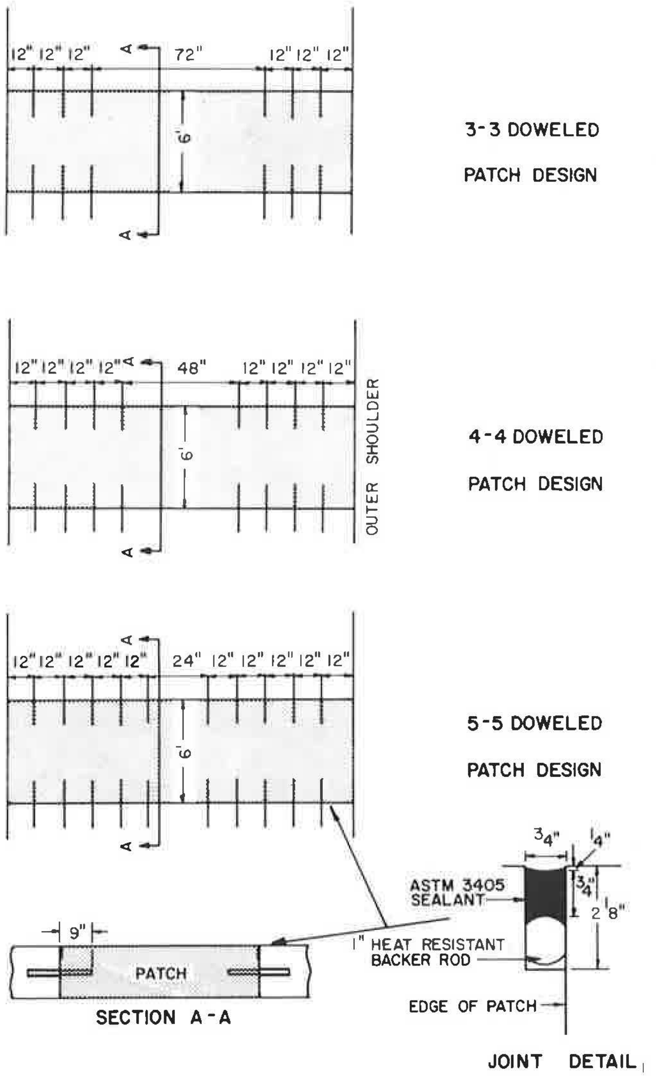

FIGURE 7 Experimental patch details.

results for dowel bar arrangement are shown in Figures 10 and 11. Also shown on the deflection plots are center of slab tests, which are taken at least 6 ft away from any joints or cracks and give an indication of subgrade support. At the end of the evaluation, all deflections and load transfers indicated that the patches were performing outstandingly.

The tied patches showed the best performance with respect to deflection and load transfer. Patches with 1.5-in. dowel bars and a 5-5 dowel bar arrangement performed second best. Although patches with 1.25-in. dowel bars and a 3-3 dowel bar arrangement performed the worst relative to the other designs, the deflection measurements and load transfer percentages were still good. Grout type appeared to have no influence on deflections or performance. It is thought that the sealed transverse joint is very important for a successful patch, not to keep water out of the patch but to prevent spalling. When 1.5-in. dowel bar, a 5-5 dowel bar arrangement, a sawed and sealed joint, and a 6-ft minimum patch are used, a low maintenance life of 10 or more years can be expected compared with 1 to 3 years for the original 3-2 patch design.

Joint Sealing

Project 5 was the only project to incorporate the complete sealing of all joints and cracks. Projects 3 and 4 used joint sealing but only at newly constructed patches, mainly for spall prevention. All joint sealing was done using a hot poured joint sealant in accordance with ASTM 3405.

In Project 5 existing cracks and joints were routed to a depth of 1 in. and a width of approximately ⅝ in. The crack or joint was then sandblasted and blown clean of dust before sealing. No backer rod or tape was required by the contract.

After the first winter it was noted that the sealant had failed by pulling away from one side of the concrete joint in full 100-ft panels. In areas where joints had been patched, the sealant

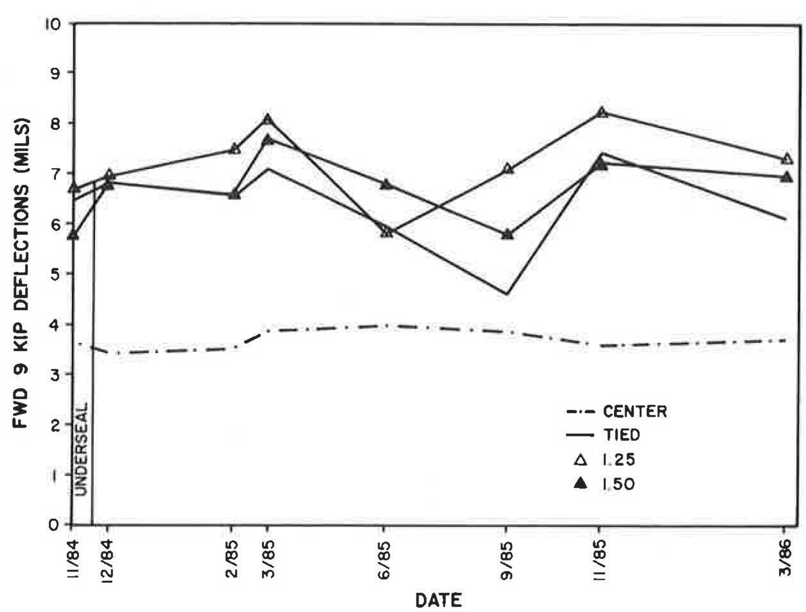

FIGURE 8 Deflection history of dowel bar size.

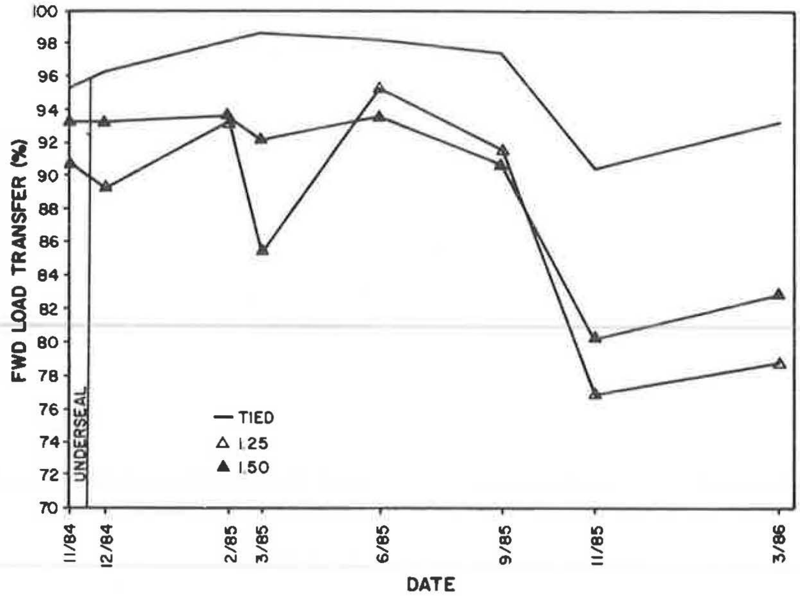

FIGURE 9 Load transfer history of dowel bar size.

was intact and performing well. This indicates that the joint reservoir did not have a properly designed shape factor for the amount of contraction in 100-ft panels.

Blanket Undersealing of Joints and Cracks

Blanket undersealing was part of the original rehabilitation on Projects 4 and 5. An undersealing experiment (*1*) on Project 2 compared grouts using fly ash and limestone as an aggregate as well as the effects of admixtures such as superplasticizers and water-reducers. Also investigated was the use of different pumping pressures, namely 10, 20, and 30 psi.

From the experiment, little difference was found between 20- and 30-psi pumping pressures, but the time required for injection at 10 psi was considerably longer. Fly ash grouts were found to be stronger and did a better job of reducing pavement deflection.

In several cases undersealing with limestone grouts actually increased pavement deflection. When fly ash grouts were used in areas of initial low deflection, minimal improvement in deflection was noted. Removal of four slabs after undersealing verified the increased flowability of fly ash grouts. Fly ash grouts spread out and covered significantly more void space than did the limestone grouts.

Blanket undersealing on Projects 4 and 5 was monitored by

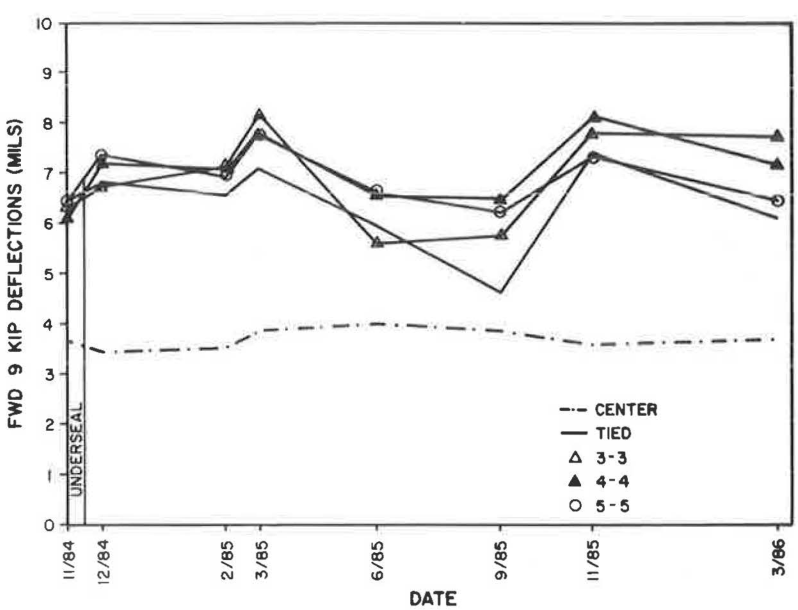

FIGURE 10 Deflection history of dowel bar arrangement.

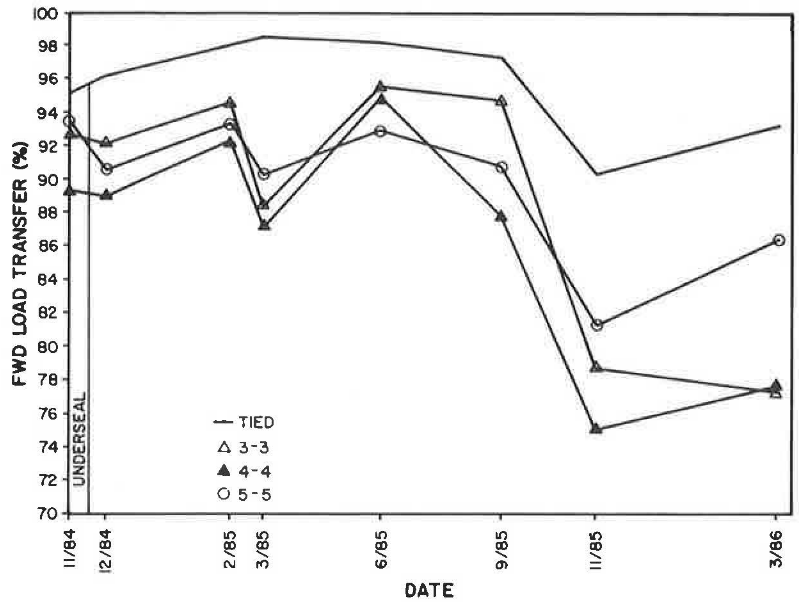

FIGURE 11 Load transfer history of dowel bar arrangement.

deflection testing before undersealing, after undersealing, and then periodically. On Project 4, the department's Road Rater 2008 was used to apply an oscillating 8-kip peak-to-peak load at a frequency of 15 Hz to the pavement. The deflection histories of outer wheelpath deflection and center of slab deflection of Project 4 are shown in Figure 12. The figure shows a reduction in deflection after undersealing, but the reduction is so small that it is doubtful whether the undersealing produced any benefits. The effects of subgrade support, which are reflected in the center deflection, appear to have more influence on deflections than does undersealing in this case.

On Project 5, a Dynatest 8002 FWD was used along with void detection procedures (*2*). In brief, the procedures for void detection are:

1. Three load ranges are applied to the approach and leave sides of joints and cracks. (Typically, these loads are 4, 8, and 12 kips or 6, 9, and 12 kips.)
2. Plots of load versus deflection are made for each test site.
3. A best-fit line is drawn through the points and extended to the axis.

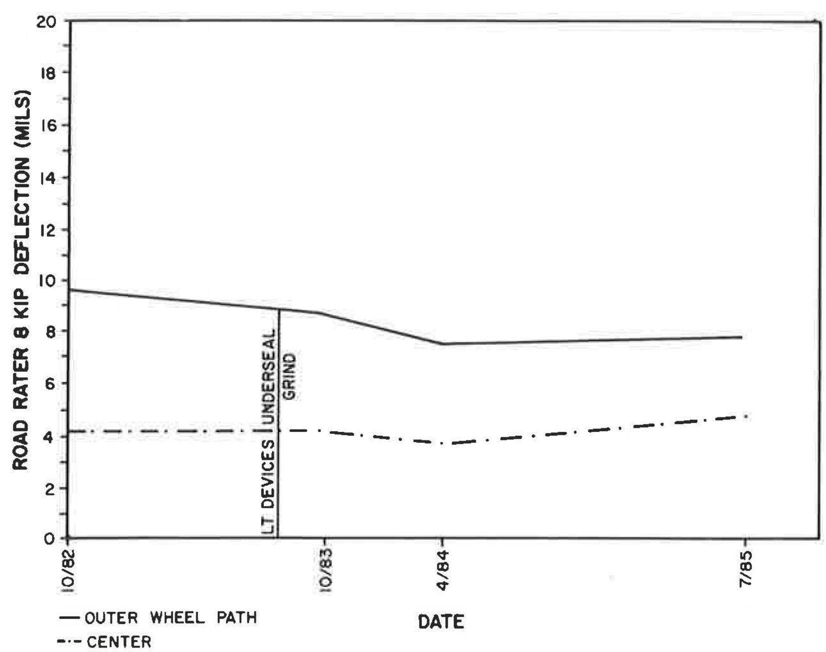

FIGURE 12 Deflection history of undersealing on Project 4.

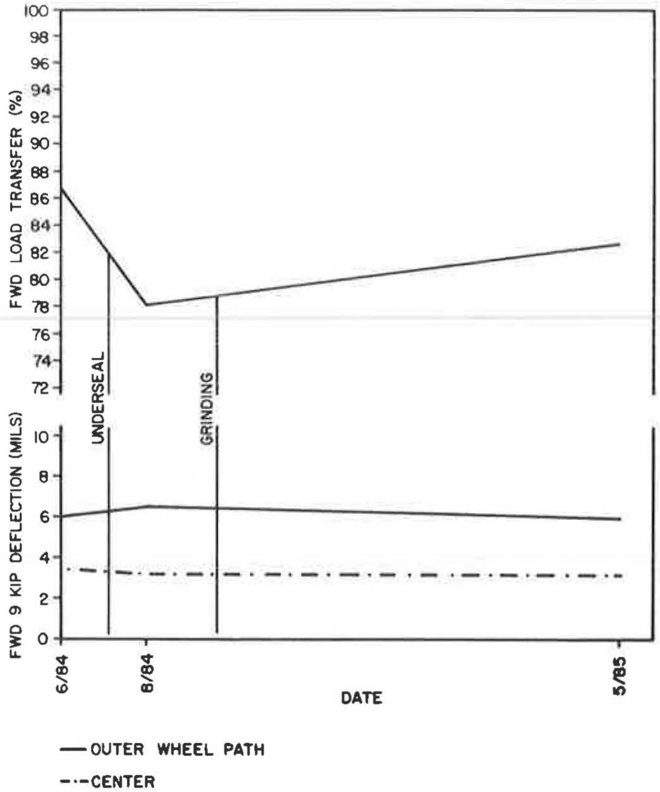

FIGURE 13 Deflection and load transfer history of undersealing on Project 5.

4. If the line intersects the deflection axis at a number greater than 0-1 mil, a void is present.

Also compared in the procedure are load transfer efficiency, 9-kip deflection, and center of slab deflections. By using these procedures, only 2 voids were found in the 53 joints and cracks tested before undersealing. Tests after undersealing and the following year showed that the voids were filled and remained stable. The deflection and load transfer histories are shown in Figure 13.

From the deflection graphs for Projects 4 and 5, it can be seen that little benefit was gained from blanket undersealing. To be effective, undersealing must be done on a selective basis only at locations of known voids.

Blanket Undersealing of Pumping Patches

In Project 1, nine patches, which were pumping and rocking under traffic, were selected for undersealing in conjunction with rehabilitation on Project 2 in 1983. Deflection testing, using the Road Rater, was conducted before and after undersealing. Deflections were greatly reduced, but continued testing showed that within 1 year deflections had nearly reached preundersealing levels. These patches, along with all other patches in Project 1, were undersealed again in the fall of 1984. The Road Rater deflection history is shown in Figure 14.

Project 3 was also undersealed in the fall of 1984 at all patch locations to arrest pumping and spalling of the patches. Before undersealing, a group of 21 patches was selected for evaluation. Deflection testing results using the FWD are shown in Figure 15, which again shows that patch undersealing was effective for about 1 year.

DOWEL BAR GROUTING EXPERIMENT

A laboratory experiment was conducted to resolve problems encountered when grouting in dowel bars. To evaluate the different techniques, a number of "hole" specimens were made by casting a polyvinyl chloride (PVC) pipe, with a $1^5/_8$-in. outside diameter, into a concrete cylinder 6 in. in diameter and 12 in. high. A bond breaker was also cast into each specimen in order that the cylinder might be split in half along the dowel bar

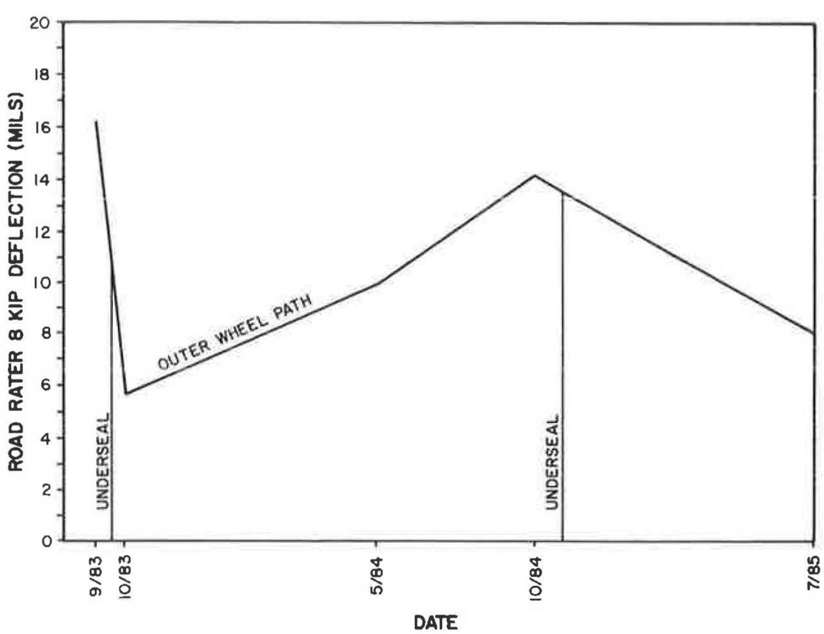

FIGURE 14 Deflection history of patch undersealing on Project 1.

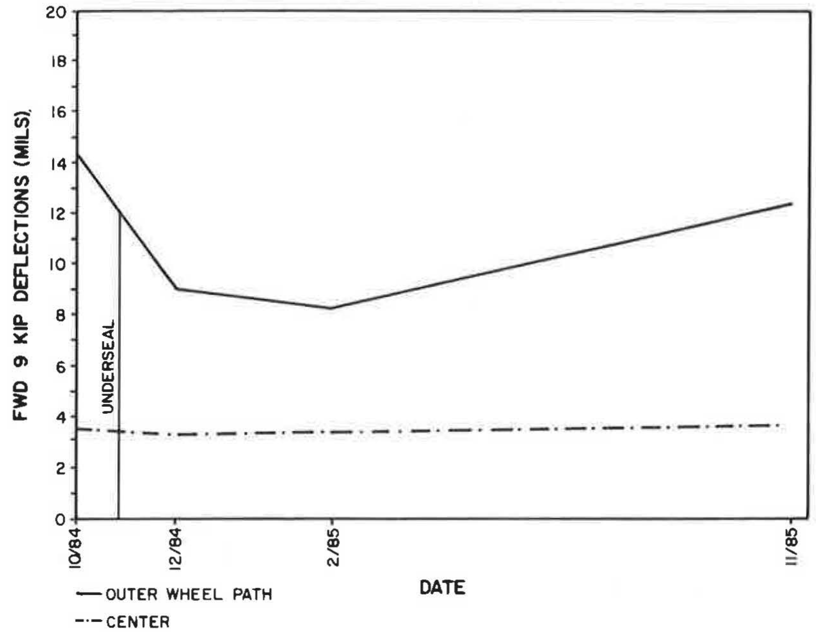

FIGURE 15 Deflection history of patch undersealing on Project 3.

after it had been grouted and cured. Cylinders were laid on their sides and different grouting techniques, along with different grout types and consistencies, were used to grout in 1½-in. dowel bars.

The following techniques and grouts were used:

- PVC pipe sleeve: Consists of a 9-in. section of 1⅝-in. inside diameter PVC pipe. The dowel bar is inserted about 3 in. into the pipe, then the pipe is filled with grout. The sleeve is then held against the pavement over the hole and the dowel bar is inserted through the sleeve into the hole, pushing the grout ahead of the bar. The sleeve is then pulled off the end of the dowel bar.
- Grout bag: This consisted of a vinyl bag similar to a pastry bag except that, in the bottom, a ¾-in.-diameter PVC pipe, 12 in. long with a reducer on the end, was used to extrude the grout. The pipe was inserted and grout deposited at the rear of the hole. The dowel bar was then inserted.
- Grout gun: This is a commercially available device that is similar to a caulking gun, except that the nozzle was extended with a 9-in. length of tubing. The nozzel was inserted and grout was deposited at the rear of the hole. The dowel bar was then inserted.
- Push rod and half sleeve: The push rod was made from a thread rod on which a 1.5-in. rubber "washer" was secured by metal washers and nuts. The half sleeve was made from a piece of sheet metal fastened to fit into the 1⅝-in. hole. The sleeve was filled with grout, inserted into the hole about an inch, and then the rod was used to push the grout to the rear of the hole. Last, the dowel bar was inserted.
- Grout pump: Grout is mechanically pumped through a hose into dowel bar hole by means of a commercially available grout pump. An on-off switch at end of the applicator allows high production. A mixer is also available that will mix grout and pour it into the pump. After the grout is applied, the dowel bar is inserted.
- Hilti Hit C-10 resin system: A two-component polyester resin is packaged in a caulking gun type of applicator that mixes components while applying resin in the rear of the hole. After application, the dowel bar is inserted. The resin has rapid strength gain.

Sand and silica nonshrinking grouts were evaluated for their ability to remain in the hole without excessive run out. The sand grout used worked well over a wide range of water contents. The silica grout was quite flowable, even at extremely low water contents, and continually ran out of the hole. Because of the run out problem, only sand grouts are recommended for dowel bar grouting. The sand grout was then used to evaluate the various grouting techniques.

When dowel bars had been grouted in and allowed to cure overnight, the specimens were split in half, the dowel bar removed, and a visual inspection made of the quality of coverage. Good coverage of grout was achieved with techniques that deposited the grout at the rear of the hole before the dowel bar was inserted. When using the PVC pipe sleeve, which uses the dowel bar to push the grout to the rear of the hole, small air voids were trapped on top of the dowel bar. This resulted in a fair to poor quality of coverage. The results of the experiment are given in terms of quality of coverage, production, workability, and material and equipment cost in Table 7.

As a result of the laboratory experiment, it was found that grout should be deposited at the rear of the hole before dowel bar insertion. Dowel bars should not be oiled before insertion. When bars are oiled, the cement is displaced or retarded, or both, around the dowel bar. The thickness of the grout should be such that it does not run out of the hole. It is important to use a back-and-forth twisting motion while inserting the dowel because this will eliminate any air voids. The back-and-forth twisting motion allowed dowels to be inserted easily, even when very dry grouts were used. There is no need to "drive" the dowel bar into the hole with a hammer unless the hole is out

TABLE 7 DOWEL BAR GROUTING TECHNIQUE RESULTS

Method	Quality of Coverage	Production	Workability	Approximate Nonshrink Grout Cost ($/hole)	Approximate Equipment Cost ($)
PVC pipe sleeve	Fair	Good	Good	0.10	0.30
Grout bag	Very good	Good	Good	0.10	5.00
Grout gun	Very good	Poor	Poor	0.10	40.00
Push rod and half round	Very good	Good	Good	0.10	5.00
Grout pump with mixer	Very good	Excellent	Excellent	0.10–0.15	1,500–2,600
	Very good	Excellent	Excellent	0.10–0.15	5,000
Hilti Hit C-10 resin	Very good	Good	Good	4.00[a]	0.00

[a]Two-component polyester resin.

of alignment or very dirty. Nonshrink grout properties are such that initial set does not take place for 2 hr and final set takes about 7 hr. For early-open patches, those to be opened to traffic in less than 24 hr, a high early strength resin or epoxy should be used.

COST-EFFECTIVENESS OF REHABILITATION

The actual bid prices for the five projects were converted to cost per two-lane mile and are given in Table 8. For patching, the percentage of pavement patched is also reported. Costs of experimental features were not included because these costs were not representative because of the small amount of work involved or work inexperience. The cost of the Double Vee load transfer devices was included only for informational purposes and is not included in the project total.

One form of rehabilitation that is used on Interstates in Illinois is to resurface with 3 in. of asphalt concrete. This consists of applying a prime or tack coat, prime coat aggregate, 1.5 in. of binder course, and 1.5 in. of surface course. The cost of 1 mi of resurfacing on 6 ft of inside shoulder, 24 ft of pavement, and 10 ft of outside shoulder averaged $124,500 in 1983, the year most of the projects were first rehabilitated. With traffic control and mobilization, the total cost for resurfacing 1 mi of Interstate was $137,000. Whether the pavement is resurfaced or not, full-depth patching and undersealing would be about the same in quality and cost because the procedures for determining patching and undersealing needs are the same. Underdrains are installed on all Interstate projects that have not had underdrains installed previously, so this cost would also be the same.

Items that are unique to CPR are partial-depth patching, joint sealing, and pavement grinding. When the cost of these techniques is compared with that of resurfacing, it is seen that the cost of CPR is about 50 percent of that of resurfacing.

CONCLUSIONS

Jointed concrete pavement can be rehabilitated cost-effectively. Pavement grinding of the projects evaluated is expected to provide good ride quality for 10 or more years. Improved full-depth patch design and procedures have resulted in greatly improved performance. Important features of the improved design are the use of 10 dowel bars, which are 1.5 in. in diameter, per joint; the use of a sawed and sealed joint; and a minimum patch length of 6 ft. The improved design is expected to give about 10 years of service. Another important requirement of full-depth patching is close inspection of dowel bar grouting. Grouting techniques that deposit grout at the rear of the drilled hole are best. Partial-depth patching has performed exceptionally well, but there is a need to locate and patch potential areas of spalling or delaminations to reduce pavement maintenance cost after rehabilitation. Underdrains will remove water from the pavement. More research is planned to deter-

TABLE 8 REHABILITATION TECHNIQUE COST ($/lane-mile)

Project	Route	Year of Rehabilitation	Full-Depth Patching	Partial-Depth Patching	Joint Sealing	Blanket Under-sealing	Retrofit Load Transfer Devices	Pavement Grinding	Pipe Under-drains	Traffic Control and Mobilization	Project Total
1	I-55	1983	42,910/4.5[a]	–	–	–	–	–	12,700	9,750	65,360
		1984	3,540/0.2	–	–	13,140	–	–	–	3,330	20,010
2	I-55	1983	4,340/0.3	8,740/0.4[a]	–	–	–	43,110	9,930	8,710	74,830
3	I-70	1983	38,300/3.9	–	–	–	–	–	–	6,600	44,900
		1984	–	–	–	7,210	–	–	–	1,820	9,030
4	I-80	1983	8,080/0.7	–	–	15,290	25,540[b]	36,700	20,220	6,690	86,980
		1985	10,260[c]/0.7	–	–	–	–	–	–	2,710	12,970
5	I-280	1984	26,070/2.4	2,570/0.2	5,870	16,220	–	45,590	33,010	18,930	148,260

[a]Percentage of pavement patched.
[b]Based on two devices per wheelpath and average spacing of 100 ft; presented for information only and not included in total.
[c]5-5 doweled patch design.

mine if underdrains are cost-effective. In the future, undersealing should only be done on a selective basis where voids are known to exist at cracks and joints. Undersealing of distressed patches is not cost-effective. Debonding failures of load transfer devices in Illinois and other states (*3*) indicate the need for more laboratory research in this area. When joints are resealed, the reservoir shape must be designed properly to prevent debonding of the sealant.

RECOMMENDATIONS

- Only sound, non-D-cracking pavements should be rehabilitated using CPR methods.
- The cost-effectiveness of pavement grinding can be further increased by grinding only the driving lane on Interstate routes.
- Delamination detection techniques should be used as an aid in locating areas in need of partial-depth patching.
- Full-depth patches should be constructed using 10 dowels with a diameter of 1.5 in. in each joint.
- When dowel bars are grouted in, the grout should be deposited at the rear of the hole and the dowel bar inserted using a back-and-forth twisting motion. The grout should be thick enough so there is no appreciable run out.
- A sawed or formed and sealed joint should be used in full-depth patching to prevent spalling.
- Void detection is necessary to determine the need for and locations of undersealing. Blanket undersealing is rarely needed.
- Spalled, rocking, and pumping patches should be replaced rather than undersealed.
- Future use of Double Vee or similar load transfer devices should be on a limited experimental basis to determine performance.
- More research is needed to determine the effectiveness of underdrains.
- Joint sealant reservoirs should be designed to provide the proper shape factors for the sealant and movement needs.

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