TABLE OF CONTENTS

CHAPTER 7

PAVEMENT REHABILITATION

7.1 Concrete Pavement Restoration Performance Review, May 22, 1997.
- Concrete Pavement Restoration Performance Review, April 1987.

7.2 Crack and Seat Performance Review Report, April 1987.

7.3 Saw and Seal Pavement Rehabilitation Technique, February 22, 1988.
- Saw and Seal Pavement Rehabilitation Technique, Technical Paper 88-01.

7.4 Reserved

7.5 FHWA Notice N5080.93, Hot and Cold Recycling of Asphalt Pavements, October 6, 1981.

7.6 Reserved.

7.7 Use of Recycled Concrete in Portland Cement Concrete Pavement, July 25, 1989.

7.8 Use of Recycled PCC as Aggregates in PCC Pavements, February 1985.

7.9 Overview of Surface Rehabilitation Techniques for Asphalt Pavements, Report Number FHWA-PD-92-008, April 6, 1992.

7.10 State of the Practice Design, Construction, and Performance of Micro-Surfacing, Report Number FHWA-SA-94-051, July 12, 1994.

7.11 Retrofit Load Transfer, Special Project 204, February 10, 1994.

7.12 Reserved.

7.13 Thin Bonded Overlay and Surface Lamination Pavements and Bridges, ISTEA 6005, July 1, 1994.

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject	Concrete Pavement Restoration and Crack and Seat Rehabilitation Performance Evaluation Reports	Date	MAY 22 1987
From	Associate Administrator for Engineering and Program Development	Reply to Attn of.	HHO-13
To	Regional Federal Highway Administrators Regions 1-10		

In a previous memorandum dated March 25, 1986, we noted the growing concerns that certain concrete pavement rehabilitation strategies and individual techniques were performing below expected levels or were not appropriate for actual project conditions. A plan was developed by Demonstration Projects Division and the Pavement Branch to conduct detailed reviews of completed Concrete Pavement Restoration (CPR) and Crack and Seat (C&S) rehabilitation projects during the remainder of the year. These reviews have been completed and the subject reports are being disseminated to provide interim technical guidance.

The University of Illinois under a FHWA research contract entitled "Determination of Rehabilitation Methods for Rigid Pavements" is undertaking a more extensive data collection and analysis effort which will provide further information on these strategies. Our CPR and C&S reviews have been coordinated with this research contract to minimize duplication of effort. Much of the initial project description data they had collected was utilized to select projects for our reviews. Conversely, detailed plan data that was gathered during our reviews is being shared with the university. The research report is expected to be available by the end of the year.

The report on CPR included an in-depth review of 26 projects in eight States. The review found that proper preliminary engineering and timing of the individual techniques are critical to project performance. When properly designed and constructed it was found that CPR will generally reduce pavement deterioration thereby, prolonging pavement life. However, continued maintenance throughout the project design life will be required. In addition, it was noted that pavements having an accelerated rate of slab cracking will continue to have a high rate of slab deterioration immediately after completion of the CPR project. The techniques which were reviewed included full depth patching, partial depth patching, diamond grinding, joint resealing, and slab stabilization (subsealing). In brief, it was found that: dowelled jointed full depth concrete patches provided satisfactory long-term performance (up to 8 years observed); partial depth patches limited to the top third of the slab and containing a compressible material in all working joints and cracks are exhibiting good performance after 6 years; diamond grinding can provide long-term improvement in ride quality, however, further evaluation on whether there is a long-term improvement in pavement friction is required; transverse joint resealing using silicone provided good performance, whereas, hot-poured sealants experienced significant adhesion failures generally within 2 years; hot-poured sealants, on the other hand, appear to be the most effective material to use in the longitudinal asphalt shoulder joint; the benefits of subsealing were not readily observed, although it did not appear to adversely affect pavement performance.

As in any other rehabilitation strategy, adequacy of design, quality and timeliness of construction, and continued maintenance determine the effectiveness of the CPR strategy. Close adherence to the technical guidance contained in the "Pavement Rehabilitation Manual", the "Techniques for Pavement Rehabilitation" course notebook, and the "1985 AASHTO-AGC-ARTBA Joint Committee's Guide Procedures for Concrete Pavement 4R Operations", generally resulted in good performance of the individual techniques within a CPR project. In cases where the pavement is suitable for rehabilitation and proper procedures are used, up to 10 years of service life can be achieved using CPR techniques.

The report on C&S included an in-depth review of 22 projects in eight States. The projects reviewed were the classic C&S type (ie., small hairline cracks, no rupturing of the reinforcement, etc.) and did not include "rubblizing" or pulverizing the pavement. Both positive and negative aspects of C&S were discovered during the review. The most positive aspect is the delay of reflective cracking. A majority of the projects reviewed showed a reduction in reflective cracking during the first few years after construction. However, most of the C&S sections exhibited the same amount of reflective cracking as the control sections after approximately 4 to 5 years. Two projects that have shown significant reduction in reflective cracking had the following similiarities: non-reinforced pavement; small changes in seasonal temperatures; and a strong base (cement treated). The primary negative aspect of C&S is the reduction in structural capacity of the pavement. To compensate for this, more overlay thickness is required, thus increasing the cost. When thick overlays (5 to 8 inches) are proposed by State highway agencies, very little structural value is given to the cracked pavement. These findings suggest that this rehabilitation strategy should be approached with caution. The costs for additional overlay thickness, the cracking and seating, and other required work such as shoulder and guardrail raising, must be evaluated when determining the most cost effective rehabilitation strategy to employ.

In developing a rehabilitation project, the process for preliminary engineering and economic analysis outlined in Administrator Barnhart's November 15, 1983 memorandum should be followed. In addition, States need to continually monitor and evaluate their previous experience with various rehabilitation strategies to determine the expected service life of these strategies in their State.

A sufficient number of copies of each report are enclosed for distribution to the Division Offices and State highway agencies in your Regions. Additional copies or technical assistance can be obtained by contacting Mr. John P. Hallin at FTS 366-1323.

Rex C. Leathers

CONCRETE PAVEMENT RESTORATION

PERFORMANCE REVIEW

Federal Highway Administration
Pavement Division
and
Demonstration Projects Division

April 1987

TABLE OF CONTENTS

I. INTRODUCTION

Federal Highway Administrator R.A. Barnhart's November 15, 1983, memorandum on pavement rehabilitation design identified the lack of good performance data as the weakest point in the rehabilitation process. Reliable performance data is a key element in evaluating alternate rehabilitation strategies and making network and project level engineering analyses.

Concerns had been expressed that the performance of certain rehabilitation strategies and individual techniques were below expected levels, or in some cases, strategies selected for specific project conditions may not have been the appropriate solution. One of the pavement rehabilitation strategies for portland cement concrete (PCC) pavements that has caused such concerns is concrete pavement restoration (CPR). Individual techniques within a CPR project include slab stabilization (subsealing), full depth patching, partial depth patching, load transfer restoration, subdrainage, shoulder restoration, diamond grinding, and joint resealing.

In order to assess the effectiveness of the CPR strategies being undertaken by State highway agencies, the Federal Highway Administration (FHWA) conducted a review of selected CPR projects. The review focused on three aspects of CPR completed on jointed plain and jointed reinforced concrete pavements:

1. Expected service life based on observed performance.
2. Variables that significantly affect the performance of individual CPR techniques.
3. Conditions under which each strategy has been used in a cost-effective manner.

Field reviews were conducted jointly by the FHWA's Pavement Division and Demonstration Projects Division between May and October 1986. Teams composed of an engineer from each division conducted in-depth reviews of 26 CPR projects in eight States. These teams were assisted by engineers from the appropriate FHWA regional and division offices and State engineers familiar with the design, construction, and maintenance of each project. The States also provided historical and inventory data for each project.

This review was closely coordinated with an ongoing research contract entitled "Determination of Rehabilitation Methods for Rigid Pavements." The research project will gather data on a large number of the variables that affect the performance of individual techniques as well as data on the success or failure of the overall strategy from approximately 150 projects in more than 20 States. Standard statistical analysis procedures will then be applied to develop conclusions. The Strategic Highway Research Program (SHRP) is expected to provide additional information regarding pavement rehabilitation strategies.

This CPR review provides interim technical guidance until the research project is completed. The findings represent the consensus of the FHWA engineers conducting the reviews based on their experience, data gathered, field observations, and discussions with field practitioners.

II. SUMMARY

Twenty-six completed CPR projects in eight States were reviewed. The review found that proper preliminary engineering and timing of the individual techniques are critical to project performance. When properly designed and constructed it was found that CPR will generally reduce pavement deterioration thereby, prolonging pavement life.

The age of the completed CPR projects ranged between 1 and 14 years. The findings presented in this report are based on project data, discussions with State personnel, field inspections of each project, and engineering judgement and experience of the FHWA team conducting the review. It is hoped that these results can be used to assist highway engineers in determining whether CPR techniques would be an effective rehabilitation strategy for particular highway projects. The report also provides information on the practices used by State highway agencies where good performance of CPR techniques was observed.

The number of individual CPR techniques undertaken on any specific project varied for project to project and State to State. Like any other pavement rehabilitation strategy the overall effectiveness of CPR techniques is highly dependent on adequacy of design, quality of construction, and maintenance practices.

The individual CPR techniques covered by this report include subsealing, full depth patching, partial depth patching, grinding, and joint resealing. Very few of the projects reviewed included pressure relief joints, subdrains retrofit load transfer devices, and shoulder restoration techniques.

Therefore, detailed comments on these techniques are not provided. It was also concluded that proper evaluation of subdrainage is not possible without performing in-depth testing. A separate project has been initiated to evaluate subsurface drainage on a variety of in-service installations.

Based on our field observations and discussions with State engineers, an effective CPR strategy will generally reduce the rate of pavement deterioration and properly designed and constructed CPR techniques can be expected to provide 6 to 10 years of service life. However, continued maintenance throughout the project design life will be required. On most projects, a followup maintenance effort was needed within 1 year of project completion.

The available preliminary engineering data developed for each CPR project was reviewed. On many projects, very little detailed information concerning the cause and extent of distress had been assembled. Some projects experienced large overruns in quantities and at least one project was terminated due to cost overruns before all CPR work could be completed. The lack of timely detailed condition data likely contributed to the major overruns.

Based on review of these 26 projects, we believe that pavements having an accelerated rate of slab cracking prior to rehabilitation will continue to have a high rate of slab deterioration immediately after completion of a CPR project. Furthermore, the percent of pavement in the right lane requiring full depth replacement of cracked slabs appears to be a good indicator of a project's suitability for CPR. The following criteria is based on our field observations of the 26 projects:

a. When 5 percent or more of the right lane required full depth replacement, the project was probably not a suitable CPR candidate.

b. When 2 percent or less of the right lane required full depth replacement, and other forms of pavement distress were within reasonable limits, the project was a suitable CPR candidate.

c. Projects requiring between 2 and 5 percent full depth replacement of the right lane were marginal CPR candidates. In these cases, we recommend that pavement deterioration be more closely monitored and evaluated. This will assist in determining whether to undertake CPR.

Of the 19 CPR projects incorporating full depth patching, 14 had a minimum patching dimension of one lane width in the transverse direction. Based on our field observations of the patches, we believe a minimum length of patch in the longitudinal direction should be 6 feet to prevent longitudinal cracking. Full depth concrete patches with dowelled joints provided satisfactory long-term performance. However, patches using the inverted tee method or those with aggregate interlock did not provide satisfactory performance. High cement factors (7 bags or more), Type III cement, and up to 2 percent calcium chloride (by weight of cement) were used to accelerate the concrete mix strength in the full depth patching projects. These projects were opened to traffic in as little as 4 hours and were performing satisfactorily after 8 years.

Partial depth patching was performed on 13 CPR projects. On eight of these projects, less than 5 percent of the total number of patches had failed. Field reviews of the patches and discussions with State engineers showed that a compressible material must be placed in all working joints and cracks within and adjacent to the patch to obtain satisfactory performance. Our field observations also confirmed that partial depth patches should be limited to the top one-third of the slab and should not extend to a depth that allows the dowel bars to bear directly on the patching material. Satisfactory long-term performance (up to 6 years observed) was achieved with standard and high-early strength PCC mixes.

Grinding was performed on 13 CPR projects to improve poor ride quality due to faulting. A ride or profile equal to or better than that for a new concrete pavement was achieved. It appears that specifications for a grinding project could reasonably include profile requirements at least as stringent as those for new PCC pavements. Grinding does not appear to have a significant positive long-term affect on pavement friction. On the four projects where friction data was available, the friction numbers returned to pregrinding levels within 2 years. Several other States reported similar trends.

Seventeen CPR projects included joint resealing. Hot-poured transverse joint sealants were used on seven projects. Those sealants experienced adhesion failure, generally occurring within 2 years after construction. Silicone sealants provided considerably better performance. However, minor adhesion failures were noted in approximately 25 percent of the joints inspected. Discussions with field personnel indicated these failures may be due to improper cleaning of the joints prior to resealing.

The benefits of subsealing could not be readily determined. Field reviews on eight projects in four States showed there was no apparent visual difference in pavement performance between States that had subsealed as part of CPR versus those that did not. Where recommended procedures were followed subsealing did not appear to have any adverse effects on pavement performance.

III. DISCUSSION

PROJECTS SELECTED

Twenty six CPR projects were reviewed in eight States. The projects reviewed and techniques evaluated are listed in Table 1. Pertinent information on each of the pavements rehabilitated is summarized in Table 2.

All of the projects evaluated were jointed concrete pavements. Thirteen of the projects were plain concrete pavements with undowelled joints. The remaining 13 projects were reinforced concrete pavements with dowelled joints. The average age of the pavements at the time of rehabilitation was 18 years, with a range of 10 to 38 years.

Very little traffic loading data was available for most of the projects. However, an attempt was made to classify the current truck loadings on the projects into four groups. The groups are based on daily volume of "5-axle or greater" trucks. This grouping was selected because these trucks generally provide 85 percent or more of the 18-kip equivalent single axle loadings on rural highways. The following groups were selected.

Loading Class	Daily 5 Axle or Greater Truck Volume
1	>1500
2	1001-1500
3	501-1000
4	<501

Table 1. Projects and rehabilitation techniques reviewed.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	SUBSEAL	EDGE DRAINS	PRESSURE RELIEF	FULL DEPTH	PARTIAL DEPTH	DIAMOND GRINDING	JOINT SEALING
					REHABILITATION TECHNIQUES						
CALIFORNIA	I-5	SHASTA CO. MP 3.8 - 14.0	1983	17	Y	Y		Y			Y
	I-80	PLACER CO. MP4 - 11.4	1984	25	Y	Y		Y	Y		
	I-5	YOLO CO. MP 23 - 27.1	1984	18	Y	Y		Y		Y	
GEORGIA	I-75	MP 226 - 232	1981	12	Y					Y	Y
	I-475	MP 0 - 15	1980	13	Y			Y		Y	Y
	I-75	MP 64 - 72	1978	17	Y			Y	Y		Y
	I-75	MP 22 - 59	1978	17	Y			Y			Y
S.CAROLINA	I-85	MP 21 - 30	1979	15	Y			Y		Y	Y
	I-20	MP 0 - 6	1984	17	Y			Y	Y	Y	Y
VIRGINIA	I-81	MP 147.2 - 161.0 NB	1984	19	Y	Y		Y	Y	Y	Y
	I-64	MP 238.4 - 254	1982	19				Y			Y
	I-64	MP 278.7 - 283.3	1983	16			Y	Y			Y
MINNESOTA	I-694	MP 37 - 46	1981	20				Y	Y		Y
	US-10	MP 204 - 211.6	1981	35						Y	
	US-71	MP 124.9 - 129.2	1983	14						Y	
	I-94	MP 81 - 103	1981	14					Y		Y
WISCONSIN	STH29	CHIPPEWA FALLS TO THORP	1983	16				Y	Y		Y
	USH61	FENNIMORE TO BOSCOBEL RD	1982	30				Y	Y	Y	
	USH151	COLUMBUS - BEAVER DAM RD	1982	20				Y		Y	
	I-90	MP 138 - 142	1981	21				Y		Y	Y
MICHIGAN	I-75	MP 64 - 80	1983					Y			
	M-47	SAGINAW CO. BT.RD TO DIV.	1983	16					Y		Y
SO. DAKOTA	I-29	MP 27 - 62	1972	10					Y		Y
	I-29	MP 0 - 15	1980	19			Y	Y	Y		Y
	I-90	MP 395.5 - 412	1985	24			Y	Y	Y		Y
	I-90	MP 265 - 292.2	1982	17					Y		Y

AVERAGE AGE	18.76
MIN	10.00
MAX	35.00

Table 2. Description of the original pavement on each project reviewed.

STATE	ROUTE	PROJECT LIMITS	YEAR OPENED	LOAD# CLASS	PAV'T TYPE	PAV'T DEPTH (IN.)	JOINT SPACING (FEET)	SKEW "/12	DOWELS	CLIMATE ZONE ##	AVG.MONTHLY TEMPERATURE MIN.	AVG.MONTHLY TEMPERATURE MAX.	FREEZE INDEX	BASE MATERIAL	SUBGRADE	YEAR REHAB	PAV'T AGE
CALIFORNIA	I-5	SHASTA CO. MP 3.0 - 14.0	1966	2	JPCP	8	VAR 12 - 19	2	N	IIC	37	97	0	CTB	CLY-GR	1983	17
	I-80	PLACER CO. MP4 - 11.6	1959	1	JPCP	8	15	0	N	IIC	39	95	0	CTB	SB-ST	1984	25
	I-5	YOLO CO. MP 23 - 27.1	1966	2	JPCP	8	VAR 12 - 19	2	N	IIC	38	96	0	CTB	ST-CLY	1984	18
GEORGIA	I-75	MP 226 - 232	1969	1	JPCP	10	30	0	N	IC	35	88	0	CTB	ST-CLY	1981	12
	I-475	MP 0 - 15	1967	2	JPCP	9	30	0	N	IC	38	93	0	ATB	SELECT	1980	13
	I-75	MP 64 - 72	1961	1	JPCP	9	30	0	N	IC	38	93	0	ATB	SELECT	1978	17
	I-75	MP 22 - 59	1961	1	JPCP	9	30	0	N	IC	38	93	0	ATB	SELECT	1978	17
S.CAROLINA	I-85	MP 21 - 34	1964	1	JPCP	9	25	0	N	IC	35	92	0	EARTH	A-7-5&6	1979	15
	I-20	MP 0 - 6	1967	2	JPCP	9	25	0	N	IC	35	91	0	EARTH	A-2-4	1984	17
VIRGINIA	I-81	MP 147.2 - 161.0 NB	1965	3	JRCP	9	61.5	0	Y	IB	29	88	0	CR. AGG.	A-6 & A-7	1984	19
	I-64	MP 238.4 - 254	1963	2	JRCP	9	61.5	0	Y	IC	32	88	0	AGG.	A-6 & A-7	1982	19
	I-64	MP 278.7 - 283.3	1967	1	JRCP	9	61.5	0	Y	IC	32	88	0	CTB	A-3 & A-2	1983	16
MINNESOTA	I-694	MP 37 - 46	1961	1	JRCP	10	40	0	Y	IA	1	81	1567	GRAVEL	A-6	1981	20
	US-10	MP 204 - 211.6	1946	4	JPCP	9-7-9	15	0	N	IA	-2	81	1987	NONE	A-1-B	1981	35
	US-71	MP 124.9 - 129.2	1969	4	JPCP	8.5	20	2	N	IIA	-1	84	1597	AGG.	A-4	1983	14
	I-94	MP 81 - 103	1967	3	JRCP	9	39.3	0	Y	IIA	-4	81	2262	GRAVEL	A-6	1981	14
WISCONSIN	STH29	CHIPPEWA FALLS TO THORP	1967	3	JRCP	9	90	0	Y	IA	3	82	1750	CR. AGG.	GRANULAR	1983	16
	USH61	FENNIMORE TO BOSCOBEL RD	1952	4	JPCP	8	20	0	N	IA	9	84	1000	CR. AGG.	GI = 14	1982	30
	USH151	COLUMBUS - BEAVER DAM RD	1954	4	JPCP	9	80	0	N	IA	8	81	1000	GRAVEL	A-4	1982	28
	I-90	MP 138 - 142	1960	1	JRCP	9	80	0	Y	IA	8	81	1000	GRAVEL	N/A	1981	21
MICHIGAN	I-75	MP 64 - 80	N/A	1	JRCP	9	99	0	Y	IA	19	83	750	N/A	N/A	1983	
	M-47	SAGINAW CO. ST.RD TO DIV.	1967	4	JRCP	9	71	0	Y	IA	15	81	750	N/A	N/A	1983	16
SO. DAKOTA	I-29	MP 27 - 62	1962	N/A	JRCP	9	61.5	0	Y	IIA	5	86	N/A	AGG.	A-7-6-12	1972	10
	I-29	MP 0 - 15	1961	N/A	JRCP	10	61.5	0	Y	IIA	5	86	N/A	AGG.	A-7-6-12	1980	19
	I-90	MP 395.5 - 412	1961	N/A	JRCP	9	61.5	0	Y	IIA	5	86	N/A	GRANULAR	A-7-6-12	1985	24
	I-90	MP 265 - 292.2	1965	N/A	JRCP	9	46.5	0	Y	IIA	5	86	N/A	LIME/AGG	A-7-6-12	1982	17

#LOAD CLASSES ARE BASED ON THE FOLLOWING ONE-WAY VOLUMES OF FIVE AXLE OR GREATER TRUCKS: >1500 = 1, 1001 - 1500 = 2, 501 - 1000 = 3, <501 = 4

AVE AGE 18.74
MIN 10.00
MAX 35.00

##CLIMATE ZONE AS DEFINED IN PART III OF AASHTO "GUIDE FOR DESIGN OF PAVEMENT STRUCTURES"

N/A = NOT AVAILABLE

A plot of pavement age at rehabilitation versus traffic load class is shown in Figure 1. As can be seen, there was no relationship for the projects covered by this review.

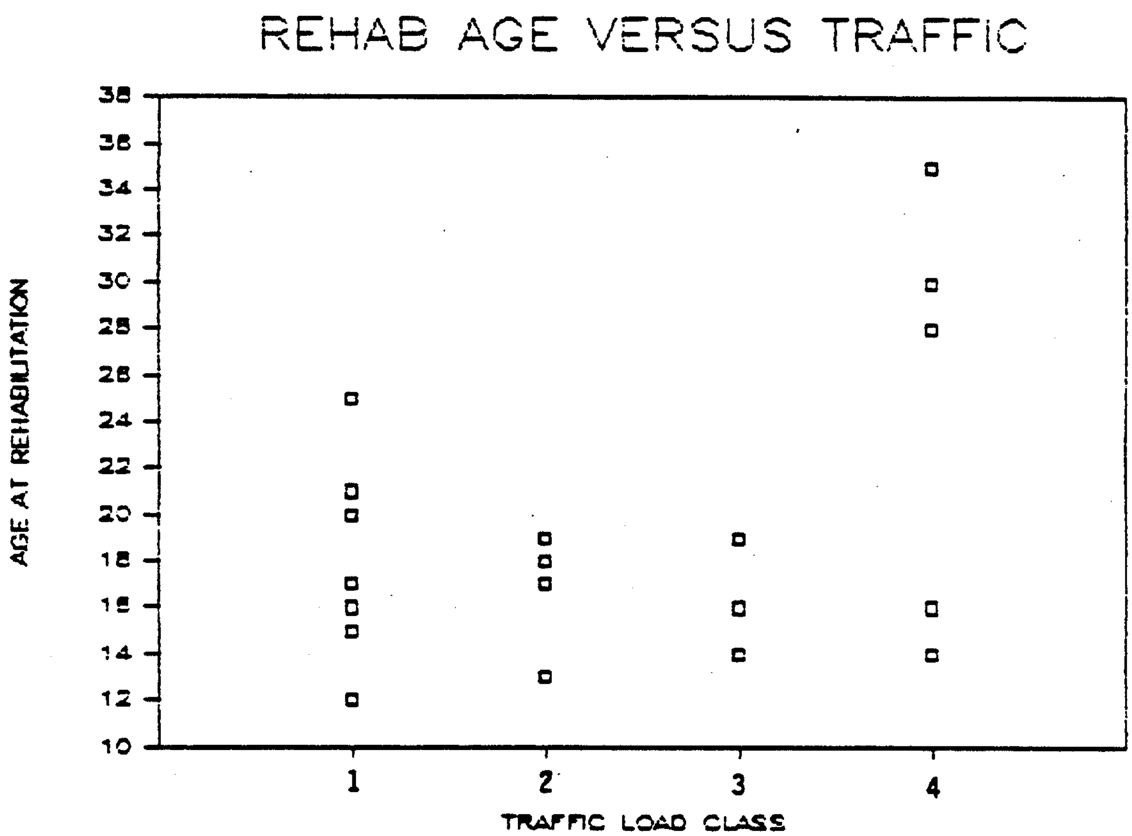


Figure 1. Age of the pavement at the time rehabilitation was required as a function of traffic.

III.B. PERFORMANCE

For each major rehabilitation project reviewed, a subjective evaluation of the performance was made. All projects were expected to provide 6 to 10 years service life. Therefore, if the project required early rehabilitation or major maintenance it was considered to have been a poor overall candidate for rehabilitation.

In addition, the performance of the individual techniques was evaluated. These techniques included full depth patching, partial depth patching, grinding, joint resealing, and subsealing. Very few projects reviewed included pressure relief joints, subdrains, retrofit load transfer devices, and shoulder restoration techniques. Therefore, detailed comments on these techniques are not provided. It was also concluded that it is not possible to evaluate subsurface drainage without performing in-depth testing. A separate project has been initiated to evaluate subsurface drainage on a variety of in-service installations. A detailed discussion of the CPR techniques follows.

OVERALL CPR STRATEGY

On many of the projects reviewed there was very little information available detailing the condition of the pavement prior to the rehabilitation project. This lack of data made it difficult to make before and after evaluations of the effectiveness of the CPR techniques. The absence of information on the rate of pavement deterioration prior to CPR made it difficult to determine how much CPR slowed the rate of deterioration.

In addition to the lack of detailed information on the extent and causes of distress, on most projects the pavement condition was not formally checked during the latter stages of project development. As a result several projects experienced overruns, for full and partial depth patching quantities, exceeding 500 percent. On one project, the quantity of partial depth patching increased from 11,511 square feet to 89,893 square feet for an overrun of $470,292. At least one project was terminated before all CPR work was completed because overruns in quantities exceeded available funds. These findings emphasize the need for detailed condition monitoring throughout the preliminary engineering phase of candidate CPR projects.

Four of the 26 projects reviewed were probably, in retrospect, poor candidates for CPR. This judgement was based on the condition of the pavements 5 years or less after rehabilitation. Three of these projects were in need of major maintenance or complete reconstruction. The other project is showing significant distress less than 2 years after rehabilitation. The principal distress on these projects was the structural failure of the slabs. At the time these projects were rehabilitated, approximately 4.7 to 16.3 percent of the pavement in the right lane was replaced by full depth patching. Generally, additional patching was required, but contract overruns were limited by fiscal constraints. In most cases, full depth patches were constructed to replace slabs that were breaking up.

Of the remaining 22 projects reviewed, 3 had a significant amount of full depth patching to correct joint distress. In general the slabs were in good structural condition other than at the joints. The remaining 19 projects were considered proper candidates for CPR and exhibited satisfactory performance for up to 8 years. The maximum quantity of full depth patching on these 19 projects amounted to 2.6 percent of the right lane surface area.

The following were findings on the use and performance of CPR:

1. An effective CPR strategy will generally reduce the rate of pavement deterioration. Properly designed and constructed CPR techniques can be expected to provide 6-10 years of service life, however, continued maintenance throughout the project's design life will be required.
2. The level of performance of each individual CPR technique was highly dependent on the adequacy of design, quality of construction, and the appropriateness of the technique selected to address the cause of distress. Even with high quality controls a few early failures of the repairs occurred, and maintenance within 1 year following completion of CPR is generally required.
3. Lack of detailed condition data during project development resulted in major overruns.
4. Pavements which have an accelerated rate of slab cracking prior to rehabilitation had a high rate of slab deterioration immediately after completion of a CPR project. It was found that the percent of the right lane requiring full depth replacement of cracked slabs appeared to be a good indicator of a project's suitability for CPR. The criteria listed below is based on field observations of the 26 projects:
   a. When 5 percent or more of the right lane required full depth replacement, the project was probably not a suitable CPR candidate.
   b. When 2 percent or less of the right lane required full depth replacement, and other forms of distress were within reasonable limits, the project was a suitable CPR candidate.

c. Projects requiring between 2 and 5 percent full depth replacement of the right lane are marginal CPR candidates and require careful monitoring to establish the current rate of pavement deterioration.

FULL DEPTH PATCHING

Full depth concrete pavement repairs were reviewed on 19 CPR projects in eight States. The age of the oldest patches observed were 8 years with an average age of 4 years. The projects reviewed are summarized in Table 3.

Three methods of providing load transfer at the transverse boundary between the patch and the existing concrete were observed. Dowels were used on eight projects. Undercutting (inverted tee) was used on four projects. Aggregate interlock only was provided for patches on six projects. In addition, on one project in Wisconsin all three methods were tried.

Full depth patches using dowels for load transfer are providing the best overall performance. A majority of the patches observed where undercutting was used exhibited settlement of the patch and/or faulting at the joint. In many cases both sides of the patch would be lower than the adjacent slabs. Patches with aggregate interlock have severe faulting, which generally appeared to be greater than the faulting on the original pavement.

Patch cracking was not a major distress, although a number of States permitted early opening (4-8 hours) after concrete placement. Typical concrete mix designs required 3000 psi compressive strength in 24 hours. Georgia indicated patches were opened to truck traffic after 6 hours at concrete compressive strengths of 1200-1500 psi with no resulting problems.

Table 3. Summary of full depth patching projects.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	PATCH AGE (YR.)	SAW DEPTH (IN.)	MINIMUM SIZE	REMOVAL METHOD	LOAD TRANSFER	CONCRETE MIX STRENGTH	OPENING TIME (HRS)
CALIFORNIA	I-5	-SHASTA CO. MP 3.8 - 14.0	1983	17	3	3	3' x 3'	BREAK	NONE	7 BAG/2% CaCl	4
	I-80	-PLACER CO. MP4 - 11.4	1984	25	2	3	N/A	BREAK	NONE	7 BAG/2% CaCl	4
	I-5	-YOLO CO. MP 23 - 27.1	1984	10	2	3	6' x 6'	BREAK	NONE	7 BAG/2% CaCl	4
GEORGIA	I-475	-MP 0 - 15	1980	13	6	FULL	LANE WIDTH x 15'	LIFT	DOWELS	7.5 BAG/TYPE III/3000 PSI/24 HRS	6
	I-75	-MP 64 - 72	1978	17	6-8	FULL	LANE WIDTH x 15'	LIFT	DOWELS	7.5 BAG/TYPE III/3000 PSI/24 HRS	6
	I-75	-MP 22 - 59	1978	17	8	FULL	LANE WIDTH x 15'	LIFT	DOWELS	7.5 BAG/TYPE III/3000 PSI/24 HRS	6
S.CAROLINA	I-85	-MP 21 - 34	1979	15	7	FULL	LANE WIDTH x 6'	BREAK	UNDERCUT 1 SIDE	8 BAG/TYPE III/2% CaCl	N/A
	I-20	-MP 0 - 6	1984	17	2	FULL	LANE WIDTH x 12'	BREAK	DOWELS	6.5 BAG/TYPE I	N/A
VIRGINIA	I-81	-MP 147.2 - 161.8 NB	1984	19	2	FULL	LANE WIDTH x 2'	LIFT	UNDERCUT BOTH SIDES	8.5 BAG/TYPE III/3000 PSI/24 HRS	48
	I-64	-MP 238.4 - 254	1982	19	4	FULL	LANE WIDTH x 2'	BREAK	UNDERCUT	8.5 BAG/TYPE III/3000 PSI/24 HRS	6
	I-64	-MP 278.7 - 283.3	1983	16	3	FULL	LANE WIDTH x 2'	LIFT	NONE	8.5 BAG/TYPE III/3000 PSI/24 HRS	N/A
MINNESOTA	I-694	-MP 37 - 46	1981	20	5	FULL	NONE SPECIFIED		DOWELS	8.5 BAG/TYPE I/5600 PSI/28 DAYS	7
WISCONSIN	STH29	-CHIPPEWA FALLS TO THORP	1983	16	3	FULL	LANE WIDTH x 6'	LIFT	DOWELS	9 BAG/TYPE I/2-4% CaCl	8
	USH61	-FENNIMORE TO BOSCOBEL RD	1982	30	4	FULL	LANE WIDTH x NO MIN.	LIFT	UNDERCUT	9 BAG/TYPE I	8
	USH151	-COLUMBUS - BEAVER DAM RD	1982	20	4	FULL	LANE WIDTH x NO MIN.	LIFT	DOWELS	9 BAG/TYPE I/2-4% CaCl	8
	I-90	-MP 130 - 142	1981	21	5	FULL	4' x 4'	LIFT	VARIED	6 BAG/TYPE IA	N/A
MICHIGAN	I-75	-MP 64 - 80	1983	N/A	3	FULL	LANE WIDTH x 6'	LIFT	DOWELS (NO GROUT)	2% CaCl/3000 PSI/8 HRS	8
S. DAKOTA	I-29	-MP 0 - 15	1980	19	6	8	LANE WIDTH x 10'	BREAK	NONE	N/A	96
	I-90	MP 395.5-412	1985	24	1	8	LANE WIDTH x NO MIN.	BREAK	NONE	N/A	96

Of the 19 CPR projects incorporating full depth patching, 14 had a minimum patching dimension of one lane width in the transverse direction. The length varied from 2 feet to a full slab length. It was noted that many of the patches less than 4 feet in length developed longitudinal cracking in the middle third of the patch. Most States reviewed specify a minimum patch of a full lane width by 6 foot length. Additional research conducted by others confirms that patches less than 6 feet in length are prone to crack and those less than a full lane width create a weakened plane in the slab.

The review confirms that full depth concrete repairs can be constructed to provide satisfactory long-term performance. The additional expense of the dowel bars appears very cost effective in reducing the recurrence of faulting.

Patching is a major cost item on most CPR projects. Therefore, the rate of continued deterioration of the pavement joints and slabs not repaired must be given careful consideration in the economic (life-cycle cost) analysis of rehabilitation alternatives.

The following were findings on full depth patching:

1. Dowelled jointed full depth concrete patches provide satisfactory long-term performance (up to 8 years observed).
2. Patches should have a minimum dimension of one lane width in the transverse direction and 6 feet in the longitudinal direction.
3. Satisfactory performance was achieved from patches opened to traffic in as little as 4 hours: higher cement factors, Type III cement, and up to

2 percent calcium chloride (by weight of cement) were used to accelerate the strength gain.

PARTIAL DEPTH PATCHING

Partial depth patching was performed on 13 CPR projects. Nine of the projects were jointed reinforced pavements with dowelled joints. The remaining four projects were plain pavements with undowelled joints. On all projects reviewed, the partial depth patches were generally used to correct spalling at transverse contraction joints.

No problems were indicated with partial depth saw cuts and concrete removal. Georgia noted excellent results with their procedure. They specify a series of parallel partial depth saw cuts within patch boundaries to facilitate concrete removal with lightweight chipping hammers (30 lb. maximum size).

The patching materials included "Set 45" on one project in Michigan and high aluminum cement on one project in California. The remaining projects were patched with PCC using either Type I or Type III cement. Some of the projects used up to 2 percent calcium chloride (by weight of cement) as an accelerator.

The cement content for the PCC patches ranged from 6.5 to 8.5 bags. Air contents were between 5.5 and 7 percent. A bonding agent was used on all of the PCC patches. The most commonly used material was a sand-cement slurry applied to either a dry or damp surface. On several projects, epoxy resin was applied to a dry surface.

The projects reviewed, specifications, and comments on performance are outlined in Table 4. The performance of the partial depth patching projects is summarized as follows:

Failure Rate (Percent of Patches)	Number of Projects
<5 percent	8
15 percent	1
75 percent	2
>90 percent	2

The only consistent difference between the projects having a low failure rate versus those with greater than 5 percent was the use of compressible material in the joints. One of the projects with a 75 percent failure rate required a compressible material in the joints, however, the depth of many of the patches was reported to be below the depth of the dowel bars. The specifications on the project required the compressible material to be placed to the top of the dowels. A bond breaker was to be used below the dowels.

Problems were observed on several projects where partial depth patches extended into the dowel bars. It is very difficult to place the compressible material in the joint adjacent to the dowels. Also, the dowel to patch contact may result in excessive stresses in the patch and at the interface of the patch and existing concrete. These stresses develop during joint movement or during curling and warping of the slab.

Table 4. Summary of partial depth patching projects.

PARTIAL DEPTH PATCHING

Project	Depth Saw Saw	PCC Removal	Prep.	Patch Mat.	Bond Agent	Surface (Dry/Wet)	Air Temp	Joint Form	Remarks
<u>California</u>									
I80-MP-4-11-.4	1.5"	Air Hammer	Sand blast	High Aluminum Cement	Epoxy	Dry	>45	Styrofoam or Cardboard	<than 5 percent failure
<u>Georgia</u>									
I475, MPO-15	Depth of Failure, 2" beyond spall	Air Hammer	Sand blast	8 bags Type III, 3000 psi/ 24 hours	Unknown	Unknown		None Saw within 24 hours	Most of the original partial depth patches have been replaced (>90% failure)
<u>South Carolina</u>									
I20, MPO-6.0	2" depth, 6" beyond spalls	Air Hammer Min. length 12" Min. width 12" Max. depth 5"	Sand blast Air	6.5 bags Type I, 2000 psi/min to open to traffic	Epoxy resin	Dry		Insert 5" deep, 6" beyond each side patch	Patches looked good, 1 failure in a 1/2 mile sample. (<5% failure)
<u>Virginia</u>									
I81, MP147.22 to 161.77	3"	Air Hammer 15 lbs. Max. Min. length 12" Min. width 12" Min. depth 3" Max. depth 4"	Water & Air blast	8.5 bags Type III Cement, 3000 psi/ 24 hours	Cement Slurry	Damp		Bituminous expansion board	Partial depth patches were performing good. (<5% failure)

PARTIAL DEPTH PATCHING (Con't)

Project	Depth Saw Saw	PCC Removal	Prep.	Patch Mat.	Bond Agent	Surface (Dry/Wet)	Air Temp	Joint Form	Remarks
<u>Minnesota</u>									
I-694, MP37 to MP46	1"	Air Hammer 30 lbs. max. Min. depth 1" Max. depth 5"	Sand blast & Air	8.5 bags Type I Cement, 5600 psi/ 24 hours, 5.5% Air	Equal parts Sand & Cement, Water to produce a stiff slurry	Dry		20 mil. plastic	Approximately 15% have failed, common distress was noted as spalling at edges.
I-94, MP81 to MP 103	1"	Air Hammer 30 lbs. max. Min. depth 1" Max. depth 5"	Sand blast & Air	8.5 bags Type I, 5600 psi/ 24 hours, 5.5% Air	Equal parts Cement, water to produce stiff slurry	Dry		Compressible materials	<5% showed any distress and none observed had failed completely.
<u>Wisconsin</u>									
S.T.H. 29 Chippewa Falls to Thorp	2"	Air Hammer Min depth 2" Max not spec.	Brooming & Water pressure	Portland Cement Concrete Type 1.	Acryl 60	Dry		Premolded filler strip or sawing	Approximately 75% of patches have failed.
U.S.H. 61 Feminore to Boscobel Rd.	2 1/2"	Air Hammer Min depth 2 1/2" Max depth- Bottom of slab	Water blasting	7 bags Type IA	Acryl 60	Dry		None- Tooled Surface	Most were loose. (>90% have failed)

PARTIAL DEPTH PATCHING (Con't)

Project	Depth Saw Saw	PCC Removal	Prep.	Patch Mat.	Bond Agent	Surface (Dry/Wet)	Air Temp	Joint Form	Remarks
Michigan									
M 47 Saginaw Co. N58 to US10	1 3/4"	Air Hammer 15# Spec. 30# Actual for some	Compres. Air	Set 45	None	Damp		Polyurethene Foam	Approximately 2% of the patches have failed.
South Dakota									
I-90 MP265-M292.22	1"	Light Jack Hammer	Sand blast	7 bags Type III		Dry		Joints formed to top of dowels only.	0% failure
I-29 MP MP 27-66				7 bags Type III Air 7 ± 2	Cement 1 part sand 1 part cement 1/2 part water	Dry		See above	Bottom of patches were at the dowels or below. (75% failure)
I-29 MP 0-15	1"	Air Hammer 30# Max		7 bags Type III	Cement Grout	Dry		See above	<5% failure.
I-90 MP 395.5 412	1"	Air Hammer 30# Max	Sand blast	7 bags Type III		Dry		See above	<5% failure.

The following were findings on partial depth patching:

1. A compressible material must be placed in all working joints and cracks within or adjacent to the patch. The compressible material should extend 1 inch below and 3 inches laterally beyond patch boundaries.
2. Partial depth patches should be limited to the top one-third of the slab and should not extend to a depth that allows the dowel bars to bear directly on the patching material.
3. Standard and high-early strength PCC mixes provided satisfactory long-term performance (up to 6 years observed).

GRINDING

Thirteen projects reviewed included diamond grinding. Eleven of these projects were on plain undowelled pavements. The purpose of grinding was to improve poor ride quality due to faulting.

Before and after ride or joint faulting data were limited. Based on the data available and comments by personnel familiar with the projects, grinding appears to provide a significant long-term improvement in ride quality. Continuous grinding of the entire lane achieved uniform ride, friction, and appearance. It was noted that grinding in the direction of or against traffic flow met ride specifications. Several States prefer to grind in the direction of traffic as a safety precaution.

Pavement texture was another area that was discussed. Three of the six States that performed grinding require the contractor to adjust the blade spacing to achieve a specified texture. Wisconsin specifies that 95 percent of any 3 foot by 100 foot section being ground must meet the texture (depth and spacing) requirements.

A ride or profile equal to or better than new concrete pavement can be obtained through grinding. It appears that specifications for a grinding project could reasonably include profile requirements at least as stringent as those for new PCC pavements. Data on the projects with grinding are summarized in Table 5.

Performance to date indicates 5 to 10 years of service life can be expected before faulting returns to the condition existing prior to grinding. However, grinding is only appropriate if the project meets criteria for CPR as previously outlined. Figures 2 and 3 are plots of the faulting index recorded on two Georgia grinding projects. Georgia's faulting index is the sum of 1/32 inches of faulting over five consecutive joints. Both these projects were on pavements in the high loading category. The data indicates that faulting is still well below the prerestoration condition 5 years after the grinding was completed.

On the four projects where friction data was available, grinding did not provide a long-term increase in pavement friction. On one project in California, the pavement had an average SN 40 of 38, 3 years prior to grinding as compared to an average of 35, 1 year after grinding. Friction data for a project in Georgia is shown in Figure 4. The figure indicates that there is a significant increase in friction number immediately after grinding, however, the friction numbers generally returned to their pregrinding levels within about 2 years. Several other States reported similar trends.

Table 5. Summary of grinding projects.

GRINDING

Project	Extend	Friction		Ride		Specifications		
		Before	After	Before	After	Profile	Equipment	
California								
I5, Yolo MP 23-27.1	All lanes entire project	3 years before 38	1 year after 35	3 years before PCA-29 PSI=2.6	1 year after PCA-12 PSI=3.8	California Profileograph 15"/mile	None	
Georgia								
I75, MP227 to 232	Outer lane (by Maint.)	see figures		see faulting index		PCA Roughness 300 max., if >300 must meet Rainhart of <7" mile		
1475, MPO-15	All lanes for entire project by State forces	see figures		see faulting		work performed by maintenance		
175, MP64-72	Outside lane					PCA <300 Areas not meeting PCA to meet Rainhart 7"/mi. Diff. at Jts. shall not exceed 1/6" Cross slope 1/4" in 12'	Self propelled machine Peaks 1/32 higher than bottom 60 grooves/ft. 60 grooves/ft	
175, MP22 to 59	Outside lane					"	"	Ride good

Project	Extend	Friction Before	Friction After	Ride Before	Ride After	Specifications Profile	Specifications Equipment	
South Carolina								
I-85, MP21-34	All lanes		N/A		N/A	1/8" in 10'	Machines using Diamond	Average faulting 1/16"-1/8" with areas of 1/4"
120 MP 0-6.0	All lanes		N/A		N/A	Mayes Meter <55 inches per mile	"	Diamond grinding saw blade spacing appeared to have been wide resulting in high fines.
Virginia								
I81, MP147-16	All lanes	35	N/A			1/8" in 10 ft.	Power Driver Self propelled Min wt. 15,000 lbs. Min. HP 200	
Minnesota								
US10, MP204-211	All lanes					3/16" in 10' parallel to centerline		Grooves between 0.097 and 0.128" wide, spaced 0.062" - 0.115" apart. Depth = 0.31 - 0.115'
						1/8 in 3' at right angle to centerline 1/16" at joints		
US71,MP124-129	All lanes					"		" "

N/A = Not Availab'

GRINDING

Project	Extend	Friction		Ride		Specifications		
		Before	After	Before	After	Profile	Equipment	
<u>Wisconsin</u>								
USH 61 Fennimore to Boscobel	All lanes							95% of any ' 3' x 100' section 1/16" from top to bottom 50 grooves/ft. min.
U.S.H. 151 Columbus-Beaver Dam	All lanes					1/8" or 10' or 0.3" in 25'		" "
I-90,MP138-142	All lanes					Transverse 1/8" in 3.		" "

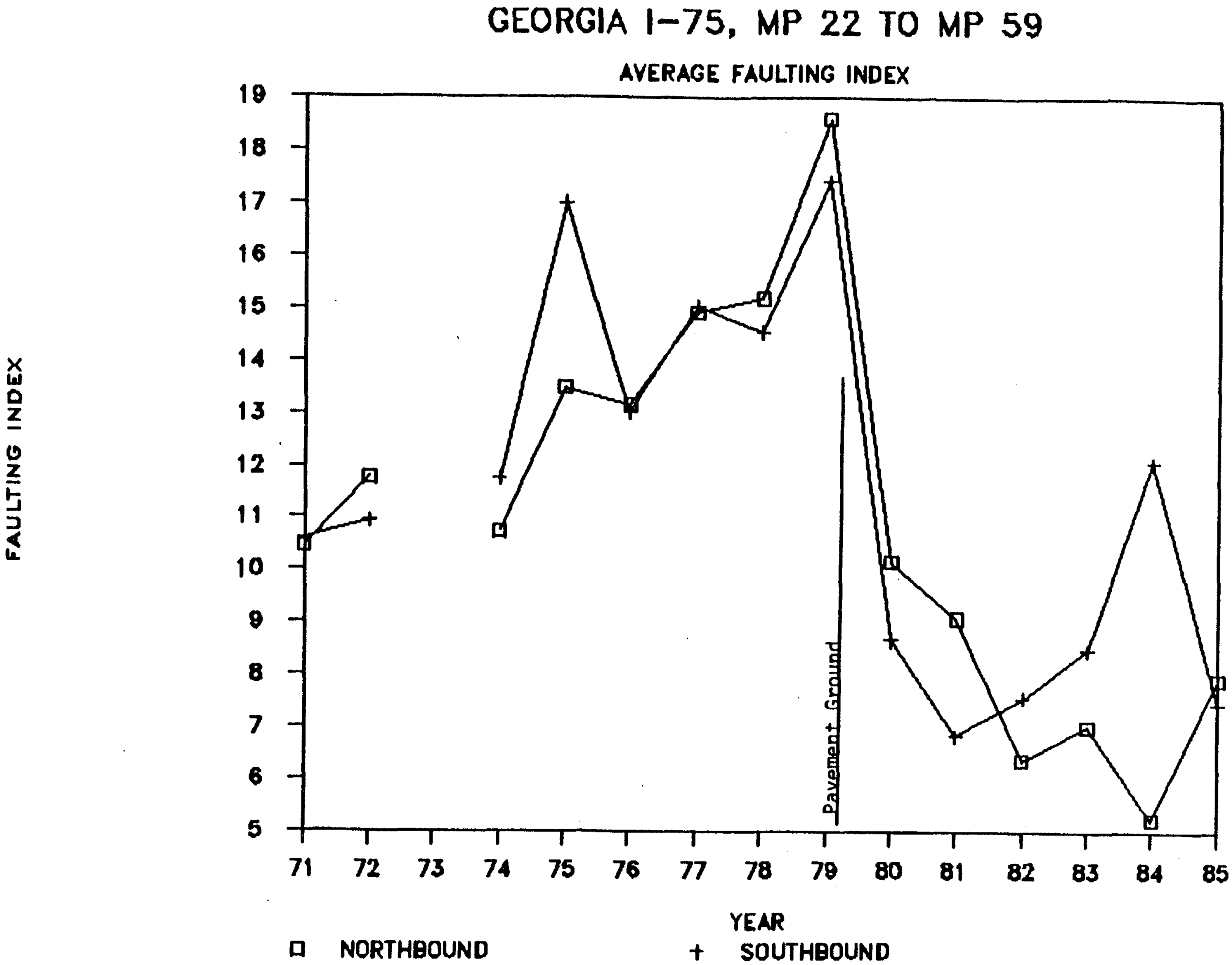


Figure 2. Faulting index before after grinding.

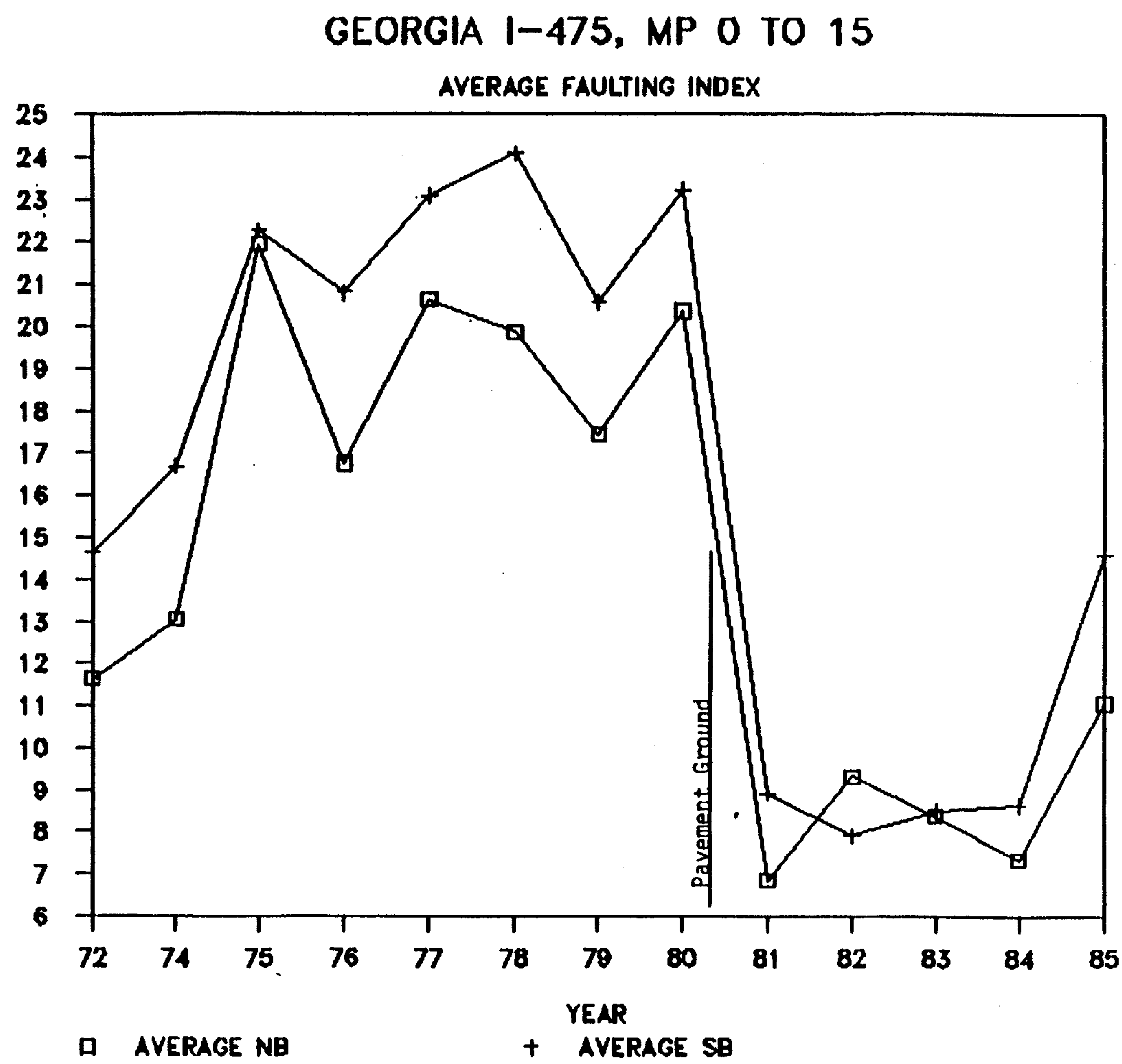


Figure 3. Faulting index before and after grinding.

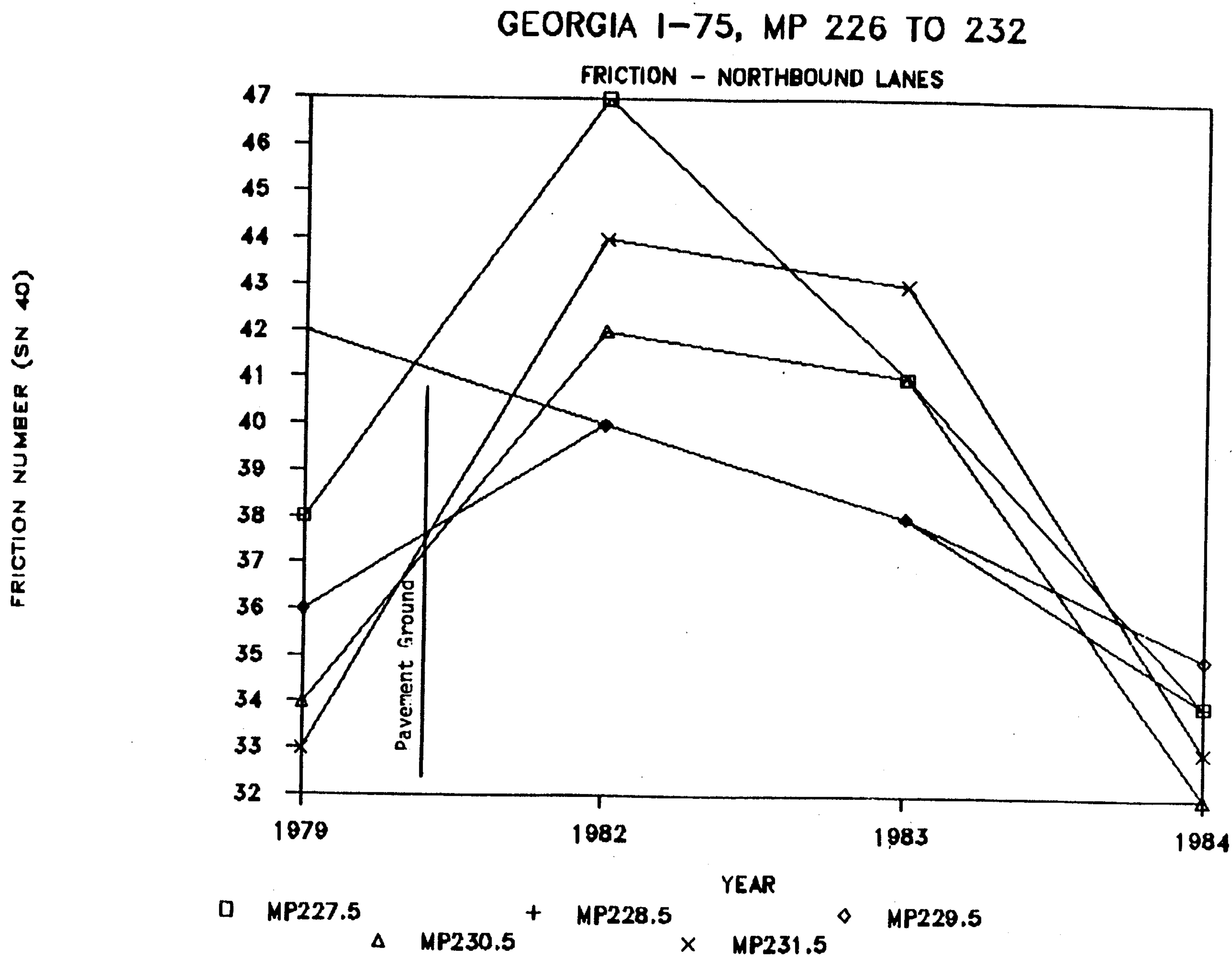


Figure 4. Friction before after grinding.

The following were findings on grinding:

1. A significant long-term improvement in ride quality can be obtained through grinding on CPR projects. Performance to date indicates a service life of 5 to 10 years can be expected before faulting returns to the pregrind condition.
2. Grinding does not appear to provide a significant long-term increase in pavement friction. Available data indicated that skid number values returned to their pregrind levels within 2 years.
3. Ride specifications were met when grinding was performed in either direction with respect to the flow of traffic.

JOINT RESEALING

Fourteen of the CPR projects included joint resealing. It should be noted that the review concentrated primarily on resealing of the transverse joints. As such, the following observations and subsequent findings are applicable to transverse joints unless indicated otherwise. The projects reviewed are summarized in Table 6. Eleven projects used one type of sealant material exclusively — six used silicone, four used hot-poured sealant, and one used neoprene. The remaining three projects used two sealant materials.

On the projects inspected, the hot-poured joint seals experienced adhesion failures, generally within 2 years after construction. Preformed neoprene seals appear better suited to new construction. Because minor joint spalling is generally present in rehabilitation projects, the full bearing required for preformed seals to remained compressed and in place may not be achieved.

Table 6. Summary of Joint Resealing Projects.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	TYPE OF JOINT	SEALANT MATERIAL	SEALANT RESERVOIR SHAPE WIDTH (IN.)	SEALANT RESERVOIR SHAPE DEPTH (IN.)	SEALANT RESERVOIR SHAPE TOP BELOW SURFACE	BACKER ROD	COMMENTS Page 1
CALIFORNIA	I-5	SHASTA CO. MP 3.8 - 14.0	1983	17	Transverse	Rubber-asphalt	1/2	1 1/4	N/A	Not required	Adhesion failures were noted.
					Longitudinal	Rubber-asphalt	1/2	1 1/4	N/A	Not required	
					Shoulder(Long.)	Not Sealed					
GEORGIA	I-75	MP 226 - 232	1981	12	Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Minor failures at 25-50% of joints
					Longitudinal	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	
	I-475	MP 0 - 15	1980	13	Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Joints regularly maintained. Most in good condition.
					Longitudinal	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	
	I-75	MP 64 - 72	N/A		Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Installed by Maintenance. Condition still good.
	I-75	MP 22 - 59	N/A		Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Installed by Maintenance. Condition still good.
S.CAROLINA	I-85	MP 21 - 34	1979	15	Transverse	Hot-poured ASTM D3405	3/8-1/2	1 1/4-1 1/2	N/A	Upholstery chord	Sealant failed in adhesion.
					Long.(ctr.line)	Not sealed					
					Shoulder(Long.)	Hot-poured ASTM D3405	3/4	3/4	N/A		
	I-20	MP 0 - 6	1984	17	Transverse	Low Mod. Silicone (Dow)	Manufacturer's Recommendation			Polyethylene	Minor adhesion failures in 25% WB, 2% EB.
					Longitudinal	Low Mod. Silicone (Dow)	Manufacturer's Recommendation			Polyethylene	
					Shoulder(Long.)	Low Mod. Silicone (Dow)	1/4	1/4	1/2		
					Shoulder(Trans)	Low Mod. Silicone (Dow)	3/8-1/2	2 1/2	Flush		
VIRGINIA	I-81	MP 147.2 - 161.8 NB	1984	19	Trans. (1-1/8"	Low Mod. Silicone (Dow)	Varied	Varied	1/4	Polyethylene	Numerous adhesion failures,possibly due to aggregate incompatibilty.
					Trans.)1-1/8"	Pref. Compression Seal					Seals looked to be in good condition.
					Long. (1-1/8"	Low Mod. Silicone (Dow)	Varied	Varied	1/4	Polyethylene	
					Long.)1-1/8"	Pref. Compression Seal					
	I-64	MP 238.4 - 254	1982	19	Transverse	Hot-poured Elastomeric	1/2	1/2	Flush	N/A	All failed in adhesion.
					Longitudinal	Hot-poured Elastomeric	1/2	1/2	Flush	N/A	
	I-64	MP 278.7 - 283.3	1983	16	Transverse	Rubber-Asphalt	3/4	3/4	Flush	N/A	Adhesion failures were noted.
					Longitudinal	Rubber-Asphalt	3/4	3/4	Flush	N/A	

N/A = NOT AVAILABLE

Table 6 con't.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	TYPE OF JOINT	SEALANT MATERIAL	SEALANT RESERVOIR SHAPE			BACKER ROD	COMMENTS
							WIDTH (IN.)	DEPTH (IN.)	TOP BELOW SURFACE		
MINNESOTA	I-694	MP 37 - 46	1981	20	Transverse	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	Sealants failed in adhesion.
					Longitudinal	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	
	I-94	MP 81 - 103	1981	14	Transverse	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	Sealants failed in adhesion.
					Longitudinal	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	
WISCONSIN	STH29	CHIPPEWA FALLS TO THORP	1983	16	Transverse	Hot-poured Elastomeric	1/2-3/4	1/2-1	1/8-1/4	N/A	All failed in adhesion.
					Longitudinal	Hot-poured Elastomeric	1/2-3/16	1/2	1/8-1/4	N/A	
	I-90	MP 130 - 142	1981	21	Transverse	Low Mod. Silicone (Dow)	3/8	1/2	1/8	Polyethylene	75% had failed in adhesion.
					Longitudinal	Hot-poured ASTM D3405	(Exist.)	3/4	1/4	N/A	
MICHIGAN	M-47	SAGINAW CO. ST.RD TO DIV.	1983	16	Transverse	Neoprene	(Exist.)	(Exist.)	1/4	None	
					Longitudinal	Sof Seal	1/4-3/8	1-1 1/4	1/8	Polyurethane	50% had failed in adhesion.
SO. DAKOTA	I-29	MP 27 - 62	1972	10	Transverse	Neoprene	N/A	N/A	N/A	N/A	Construction data not available.
	I-29	MP 0 - 15	1980	19	Transverse	Low Mod. Silicone (Dow)	7/8	1/2	1/4	Polyethylene	
					Longitudinal	Not sealed					
	I-90	MP 395.5 - 412	1985	24	Transverse	Low Mod. Silicone (Dow)	7/8	1/2	1/8	Polyethylene	
					Longitudinal	Hot rubber-asphalt	7/8	1 7/8	1/8		
					Shoulder (Long.)	Not Sealed					
	I-90	MP 265 - 292.2	1982	17	Transverse	Low Mod. Silicone (Dow)	N/A	N/A	N/A	Polyethylene	

N/A = NOT AVAILABLE

A majority of projects with joints resealed using silicone material are providing good performance. However, periodic maintenance is required to cut out and repair isolated failures. Where field personnel indicated that manufacturer's guidance for installation was not followed explicitly, significant failures (up to 75 percent) were observed. Items such as refacing the joint to allow for the proper shape of the joint sealant material, providing a clean and dry bonding surface, and close inspection of the application and tooling of this sealant are critical to performance.

Several projects included sealing of the longitudinal asphalt shoulder joint with hot-poured sealants. This material appears to be effective for about 2 years before maintenance is required.

The following were findings on joint resealing:

1. Preformed neoprene joint seals are generally not suitable for rehabilitation projects because even a small amount of spalling in the existing joint can result in failure of the seal.
2. Hot-poured sealants observed experienced significant adhesion failure, generally within 2 years.
3. Silicone, when properly installed, provided good performance. However, in all cases, maintenance was required after 1 to 2 years to correct construction deficiencies.
4. Hot-poured sealants used in the longitudinal asphalt shoulder joint requires maintenance on about a 2 year cycle.

SLAB STABILIZATION (SUBSEALING)

Eight projects were reviewed in four States where subsealing had been used as a rehabilitation technique. Criteria for subsealing ranged from blanket subsealing of all faulted slabs to subsealing only those slabs which exceeded a minimum specified deflection under loading. The projects reviewed are summarized in Table 7.

It was difficult to evaluate the effectiveness visually. However, an attempt was made to determine if the subsealing had any apparent detrimental affects. Slabs were closely observed for cracks which passed through or radiated from the subsealing holes. No distress of this type was observed. One project had a high level of slab breakup following a CPR project which included subsealing. However, a review of the project records and discussion with project personnel indicated that there was a high rate of deterioration occurring prior to the CPR project.

A research project is currently underway to document and evaluate undersealing techniques. A product of that project will be a users manual for undersealing.

The following was the finding on subsealing:

1. Although the benefits of subsealing could not be readily observed, there appeared to be no adverse affect to pavement performance when procedures outlined in FHWA's "Pavement Rehabilitation Manual", "Techniques for Pavement Rehabilitation" course notebook, or the "1985 AASTHO-AGC-ARTBA Joint Committee's Guide Procedures for Concrete Pavement 4R Operations" were followed.

Table 7. Summary of Subsealing projects.

Project	Delfection	Grout	Hole Pattern	Pressure	SUBSEALING Depth of Hole	Min. Air Temp.	Holes Plugged	Lift	Remarks
California									
I5, Yolo MP23-27.1	None available	2.5 parts Pozzolan 1 part cement time efflux 10-16	3-5 in leave slab	none specified	15 inches	45 F	Wooden pegs	1st movement	No deterioration which could be related to subsealing noted.
180 Placer MP4.0-11.4	see remarks	" "	3-4 in leave slab	"	15 inches	45 F	Wooden pegs	1st movement	Deflections were taken before and after subsealing. When taking deflections the difference between the deflection of loaded and unloaded side were measured. before 0.007 inches after 0.008 inches
I5, Shasta MP 3.8 to 14	load transfer only	3 parts pozzolan 1 part cement time efflux 11-16 spec., 10-11 actual, comp. strength spec. 750 psi/ 7 days, Actual 1430 psi/ 7 days	3-4 in leave slab	"	15 inches Nozzle not to extend below bottom of slab			1st movement but not monitored in all slabs	

SUBSEALING (Cont.)

Project	Delfection	Grout	Hole Pattern	Pressure	Depth of Hole	Min. Air Temp.	Holes Plugged	Lift	Remarks
<u>Georgia</u>									
I75, MP 227-232	Min 0.025"								No deflection >0.025 in. so subsealing was deleted.
I475, MPO-15	Min 0.005"	3 parts lime-stone 1 part cement time efflux 16-22 sec	2 holes 18" from leave, 4 additional along outer edge		8" below bottom of slab		No	1/8"	
<u>South Carolina</u>									
I85, MP 21 to 34	Tested under 18 kip Axle if movement visually observed it was subsealed	3 parts ag. lime, 1 part type III cement, water content to give slurry appear-ance of thick cream	1 hole on approach side, 4 on leave side	None speci-fied	Bottom of slab	35 and rising 40 and falling	Wooden pegs	When dial indicates movements Max. 1/8"	Slabs retested and regrout as necessary.

Project	Delfection	Grout	Hole Pattern	Pressure	Depth of Hole	Min. Air Temp.	Holes Plugged	Lift	Remarks
<u>South Carolina</u>									
I20, MPO-6	Min 0.020"	3 parts ag. lime, 1 part cement time efflux 14-22 sec	2 hole on approach 4 on leave	None speci-fied	8" to 10" below slabs	35 and rising	Wooden pegs	<1/8" any slab raised more than 1/8" replaced.	Slabs retested and any with movement >0.020 in. was regrouted.
<u>Virginia</u>									
I81, MP 147 to 161 NB	Not performed. All slabs in RT Lane were subsealed.	3 parts Pozzolan 1 part cement time of efflux 10-16 sec Mean compres-sive Str. 1 day 300 psi 3 day 620 psi 7 day 1000 psi	7 holes per slab	200 psi max.	Bottom of slab	35 F	Wooden pegs	0.125 in. per hole. Many holes pumped to max.	

VI. COST DATA

Available cost data for the rehabilitation techniques used on each project are listed in Table 8. Also shown are the planned and final quantities. The data is presented by project to give the reader a feel for the total scope of each project.

Table 8. Summary of quantities and bid prices.

STATE	ROUTE LIMITS	YEAR REHAB.	TECHNIQUE	UNITS	BID PRICE	QUANTITIES PLAN	QUANTITIES FINAL	COMMENTS
CALIFORNIA	I-5 SHASTA CO. MP 3.0 - 14.0	1983	Slab Replacement	sq. yd.	$36.27	8970	11307	No load transfer.
			Corner Patch	sq. yd.	$214.33	45	45	
			Subseal Hole	ea.	$4.50	21600	32521	
			Grout	cwt.	$15.00	11000	19562	
	I-80 PLACER CO. MP4 - 11.4	1984	Full Depth Patches	sq. yd.	$63.95	2340	2580	No load transfer.
			Partial Depth Patches	sq. yd.	$250.00	210	191	
			Subseal Hole	ea.	$4.50	14200	13181	
			Grout	ton	$320.00	230	222	
	I-5 YOLO CO. MP 23 - 27.1	1984	Full Depth Patches	sq. yd.	$73.33	2073	2706	No load transfer.
			Subseal Hole	ea.	$5.50	6310	5813	
			Grout	ton	$1,600.00	150	52	
			Grinding	sq. yd.	$2.99	95000	N/A	
GEORGIA	I-75 MP 227 - 232	1981	Subseal Hole	ea.	$3.00	2500	0	
			Grout	bags cem.	$27.50	600	0	
			Subseal Pre. Test	mile	$1,200.00	12	12	
			Subseal Stab. Test	joint	$600.00	800	0	
			Grinding	sq. yd.	$4.04	80000	80761	Includes traffic control.
			Joint Seal	lin. ft.	$0.64	167000	133026	Low mod silicone.
			Sawing Joints	lin. ft.	$0.50	167000	133026	
	I-475 MP 0 - 15	1980	Slab replacement	cu. yd.	$90.00	240	457	Dowelled load transfer.
			Slab removal	sq. yd.	$70.00	600	1259	
			Partial Depth Patches	sq. ft.	$19.50	4000	7069	
			Subseal Hole	ea.	$4.42	5300	4670	
			Grout	bags cem.	$19.50	1100	494	
			Subseal Pre. Test	mile	$1,050.00	20	20	
			Subseal Stab. Test	joint	$556.00	890	820	
			Grinding	sq.yd.				Done by maintenance forces
			Saw and Reseal	lin. ft.	$1.61	92300	80742	Existing joints were Uni-tube. Low mod silicone.
			Saw and Reseal	lin. ft.	$1.40	154200	154309	Other joints and cracks. Low mod silicone.
	I-75 MP 64 - 72	1978	Grinding	sq. yd.	$3.49	N/A	126900	
	I-75 MP 22 - 59	1978	Grinding	sq. yd.	$3.07	N/A	232600	
S. CAROLINA	I-85 MP 21 - 34	1979	Full Depth Patches	sq.yd.	N/A	3800	N/A	No load transfer.
			Subseal Hole	ea.	N/A	2375	N/A	
			Grout	bags cem.	N/A	1068	N/A	
			Grinding	sq. yd.	N/A	361891	N/A	Continuous grinding.
	I-20 MP 0 - 6	1984	Full Depth Patches	sq.yd.	$110.80	750	2155	Dowelled load transfer.
			Partial Depth Patches	sq. ft.	$24.00	1000	9555	
			Subseal Hole	ea.	$4.00	12240	12459	
			Grout	bags cem.	$22.00	4080	1472	
			Grinding	sq.yd.	$2.48	217621	183551	Continuous grinding.
			Joint Sealing	lin. ft.	$1.53	170311	171136	Existing jts. were Uni-tube. Low mod silicone.
			PCC Shoulder 4 ft.	sq. yd.	$14.75	36448	34431	Full depth retrofit shoulders.
			PCC Shoulder 6 ft.	sq. yd.	$14.05	8548	9142	Full depth retrofit shoulders.
			PCC Shoulder 10 ft.	sq. yd.	$13.55	70757	61984	Full depth retrofit shoulders.

Table 8 con't.

STATE	ROUTE LIMITS	YEAR REHAB.	TECHNIQUE	UNITS	BID PRICE	QUANTITIES PLAN	QUANTITIES FINAL	COMMENTS
VIRGINIA	I-81 MP 147.2 - 161.8 NB	1984	Full Depth Slab Rep.	sq. yd.	$100.00	1884	3502	Final quantities represent only one-half of entire project. Other half was readvertised. Inverted tee if patch 2' to 42'. Dowelled if longer.
			Partial Depth Patches	sq. yd.	$80.00	0	87	
			Subseal Hole	ea.	$2.00	9885	10956	
			Grout (Cement)	cu. ft.	$25.00	2275	10009	
			Grinding	sq. yd.	$2.00	202611	100851	
			Joint Sealing	lin. ft.	$1.30	104985	61900	Silicone
			Joint Sealing (2" pref.)	lin. ft.	$12.00	7001	888	Neoprene compression seal.
			Joint Sealing (3" pref.)	lin. ft.	$20.00	1046	871	Neoprene compression seal.
			Joint Sealing (3.5" pref.)	lin. ft.	$40.00	38	234	Neoprene compression seal.
			Edge Drains	lin. ft.	$10.00	16685	8835	
	I-64 MP 230.4 - 254	1982	Full Depth Patches	sq. yd.	$109.00	2300	2721	Final quantities represent only one-half of entire project. Project was terminated. Inverted tee.
			Joint Sealing	lin. ft.	$0.58	174010	79644	Hot poured elastomeric.
	I-64 MP 278.7 - 283.3	1983	Full Depth Patches	sq. yd.	$89.00	9323	8221	No load transfer.
			Joint Sealing	lin. ft.	$0.65	96222	150516	Rubber asphalt joint sealant.
			Pressure Relief Joint	lin. ft.	$25.00	2300	2044	
MINNESOTA	I-694 MP 37 - 46	1981	Full Depth Patches	N/A				Full and Partial depth patch quantities were intermixed. Partial depth patch boundaries are tapered. Dowelled load transfer.
			Partial Depth Patches	N/A				
	US-10 MP 204 - 211.6	1981	Grinding	sq. yd.	$2.00	59269	N/A	
	US-71 MP 124.9 - 129.2	1983	Grinding	N/A				Quantities and costs not available.
	I-94 MP 81 - 103	1981	Partial Depth Patches	cu. ft.	N/A	500	4421.	Boundaries are tapered.
			Joint Sealing	lin. ft.	N/A	271787	278500	Hot poured rubber.
			Crack Sealing	lin. ft.	N/A	6275	2864	
WISCONSIN	STH29 CHIPPEWA FALLS TO THORP	1983	Full Depth Patch 6' by 12'	sq. yd.	$47.00	14136	15440	Dowelled load transfer.
			Full Depth Patch 8' by 12'	sq. yd.	$42.84	960	1397	Dowelled load transfer.
			Full Depth Patch 10' by 12'	sq. yd.	$39.37	400	733	Dowelled load transfer.
			Full Depth Patch 12' by 12'	sq. yd.	$37.01	176	416	Dowelled load transfer.
			Full Depth Patch 16' by 12'	sq. yd.	$35.63	427	1024	Dowelled load transfer.
			Full Depth Patch 20' by 12'	sq. yd.	$34.50	720	347	Dowelled load transfer.
			Partial Depth Patches	sq. ft.	$9.00	2000	4154	
			Longitudinal Joint Sealing	lin. ft.	$0.85	167000	152098	Hot poured elastic.
			Transverse Joint Sealing	lin. ft.	$0.90	40000	37807	Hot poured elastic.
	USH41 FENNIMORE TO BOSCOBEL RD	1982	Full Depth Patches	sq. yd.	$30.60	7981	7197	Inverted Tee (6" by 6") load transfer.
			Partial Depth Patch	sq. ft.	$16.00	1000	1392	
			Grinding	sq. yd.	$2.50	77500	77460	
	USH151 COLUMBUS - BEAVER DAM RD	1982	Full Depth Patches	sq. yd.	$68.00	700	1172	Inverted Tee (6" by 6") load transfer.
			Grinding	sq. yd.	$2.55	97350	89759	
	I-90 MP 130 - 142	1981	Full Depth Full Lane Patch	sq. yd.	$40.00	4074	7866	Dowelled and no load transfer methods used.
			Full Depth Corner Patch	sq. yd.	$42.00	475	800	No load transfer.
			Full Depth Inverted Tee Patch	sq. yd.	$50.00	210	416	
			Grinding	sq. yd.	$2.92	41710	41674	
			Longitudinal Joint Sealing	lin. ft.	$0.70	20785	30156	Hot poured elastic meeting ASTM D3405.
			Transverse Joint Sealing	lin. ft.	$1.25	40722	64941	Low mod. silicone (Dow 888).

Table 8 con't.

STATE	ROUTE	LIMITS	YEAR RCNAD.	TECHNIQUE	UNITS	BID PRICE	QUANTITIES PLAN	FINAL	COMMENTS
									PAGE 3
MICHIGAN	I-75	MP 64 - 80	1983	Full Depth Patches	sq.yd.	N/A	23532	26481	Dowelled load transfer.
				Pressure Relief Joints	ea.	N/A	0	844	
	M-47	SAGINAW CO. ST.RD TO DIV.	1983	Joint Spall Repair	lin. ft.	$16.00	7750	10918	
				Patching Material	cu. ft.	$42.00	930	1032	
				Sealing Transverse Cracks	lin. ft.	$1.75	10960	16514	Sealtight Sof-Seal (Not poured).
				Replace Neoprene Seals	lin.ft.	$3.10	N/A	20940	
				Exp. Jt. Removal and Reseal	lin. ft.	$4.25	N/A	7512	Sealtight Sof-Seal (Not poured).
SO. DAKOTA	I-29	MP 27 - 62	1972	Partial Depth Patch (Type B)	sq. ft.	$17.50	364	568	
				Partial Depth Spall Repair	sq. ft.	$5.79	24337	105336	
				Joint Sealing	lin. ft.	$1.01	117481	110223	Neoprene
	I-29	MP 0 - 15	1980	Full Depth Repair	sq. yd.	$60.00	N/A	800	No load transfer.
				Partial Depth Patch (Type B)	sq. ft.	$40.00	255	255	
				Partial Depth Patches	sq. ft.	$6.40	127653	109193	
				Joint Sealing	lin. ft.	$2.00	65424	67765	Silicone (Dow 888)
				Pressure Relief Joints	ea.	$800.00	107	106	
	I-90	MP 395.5 - 412	1985	Full Depth Patches	sq. yd.	$60.00	636	1581	Dowelled load transfer.
				Partial Depth Patch (Type A)	sq. ft.	$6.00	11511	89893	Type A patch is over 0.4-foot wide.
				Joint Sealing (Silicone)	lin. ft.	$1.68	77712	74981	No pave't/shoulder jt. sealing set-up in contract.
				Pressure Relief Joints	ea.	$400.00	32	32	
	I-90	MP 265 - 292.2	1987	Partial Depth Patch (Type A)	sq. ft.	$6.87	36806	32547	Type A patch is over 0.4-foot wide.
				Partial Depth Patch (Type B)	sq. ft.	N/A	72	181	Type B patch is 0.4-foot or less wide.
				Joint Sealing	lin. ft.	$1.59	6094	6094	Silicone sealant.
				Pressure Relief Joints	ea.	$695.00	45	45	

CRACK AND SEAT PERFORMANCE

Review Report

Federal Highway Administration

Demonstration Projects Division

and

Pavement Division

April 1987

TABLE OF CONTENTS

A. Executive Summary

Based on the findings of this review, the use of cracking, seating, and overlaying as a pavement rehabilitation alternate should be approached with caution. Since both positive and negative aspects of cracking and seating (C&S) were identified during the review, State agencies contemplating the use of C&S should do a thorough project by project analysis to determine if it is the most cost effective rehabilitation technique to employ.

Of the 22 projects reviewed, only four showed appreciably less reflective cracking in the C&S sections than in the control sections. Observations by the review team, coupled with previous State reports, indicate that there generally is a reduction in the amount of reflective cracks through the overlay during the first few years following construction of a C&S project. However, after 4 to 5 years the C&S sections exhibited approximately the same amount of reflective cracks as the control sections. A significant reduction in reflective cracks occurred on two of the projects reviewed. These projects are located on I-4 in Florida and on SR-99 in California. Both had the following similarities:

1. Constructed on a strong base (cement treated),
2. Small changes in seasonal temperatures, and
3. Non-reinforced pavement.

The main concern with C&S is the reduction of the structural capacity of the pavement. To compensate for the reduction in structural capacity caused by cracking the pavement, more overlay thickness is required, thus increasing the cost. In addition, study is needed to determine if the delay in reflective cracking actually extends the life of the pavement as opposed to conventional overlays and if so, is it cost effective.

B. Background/Introduction

When portland cement concrete pavement (PCCP) approaches the end of its design life, a decision must be made on what action to take. The most common rehabilitation technique currently used for PCCP is to construct an overlay of asphalt concrete (AC). In time, cracks in the underlying PCCP reflect into the overlay. These cracks are primarily caused by stresses that develop at the bottom of the new overlay directly over the in-place cracks and joints of underlying pavement. These stresses are a result of vertical and horizontal movements of the underlying pavement. Vertical movements are differential movements at the joint/crack in the underlaying pavement and are caused by moving loads. Horizontal movements are due to expansion and contraction caused by temperature and/or moisture changes.

In addition to these changes in the underlying slab, total movement at a crack or joint is affected by slab length and the stiffness of the underlying material. The horizontal movement of cracked slabs under a bonded bituminous surface causes high tensile stresses in the immediate area over the crack. Likewise, vertical movement causes high stresses in the overlay. Because an AC surface is stiffer at lower temperatures, it loses some of its flexible characteristics and can withstand only small temperature-induced stresses.

One method that several States have tried for control of reflective cracking in an overlay is to crack the concrete pavement slab into small segments before overlaying with AC. The intent of pavement cracking and seating is to create pavement sections that are small enough to reduce movement to a point where thermal stresses will be greatly reduced, yet still be large enough to maintain some aggregate interlock between pieces and retain a significant percentage of the original structural strength of the PCC pavement. Seating of the broken slabs after cracking is intended to reestablish support between the subbase and the slab where voids may have existed.

C. Objectives

The objectives of this review were to obtain a better understanding of the expected performance of C&S and overlaying, and to identify the conditions under which this technique has been used in a cost-effective manner. It is hoped that the information obtained from the review will aid States in determining when and how to use C&S as an effective rehabilitation strategy.

D. Selection Criteria

A total of 22 projects in 8 States were reviewed. All of the projects reviewed were of the classic crack and seat method (small hairline cracks, no rupturing of the reinforcing, and no rubblizing of the pavement). The following factors were considered in selecting the projects to be reviewed:

- preferably 3 or more years of service;
- located on a high volume facility;
- historical data accessible;
- overlay thickness of 6 inches or less; and
- a control section.

Using these factors, C&S projects were selected for review in:

- California
- Michigan
- Minnesota
- South Dakota
- Wisconsin

After analyzing the data obtained on projects built in the originally selected States, it was decided to extend the review to include projects in Florida, Tennessee, and Indiana, as well as additional projects in California.

E. Field Survey Results

The general condition of each C&S project reviewed is described in this section.

1. California

   a. I-80 Alameda and Contra Costa Counties

I-80 is an 8-inch undoweled jointed plain concrete pavement (JPCP) on a 4-inch cement treated base (CTB) on 8 inches of select material. The original 6-lane pavement was opened to traffic in the mid-1950's.

In 1982, a rehabilitation project which included C&S with an AC overlay and with edgedrains retrofitted on both the C&S and the control sections was constructed. The pavement was broken into 3- by 4-foot segments with an air operated pile driver and then rolled with a vibratory sheepsfoot roller weighing not less than 12 tons to seat the slabs. The control sections were overlaid with 3 1/4 and 5 inches of AC, but not cracked and seated. The C&S section was overlaid with 5 inches of AC. This was the first C&S project in California, therefore, the bid price of $12.50 per square yard was very high. The current average daily traffic (ADT) is 177,000 with 7.3 percent trucks.

The original pavement was badly cracked and faulted (greater than 1/4 inch). Rocking slabs were reported.

With the exception of two reflective cracks from known rocking slabs, which were intentionally left unseated for evaluation purposes, no other reflective cracks were observed on the project. After nearly 4 years, both the 3 1/4-inch and 5-inch control sections and the C&S sections are performing about the same.

b. I-80 Yolo County

I-80 is a 9-inch undoweled JPCP with a 15-foot joint spacing over a 6-inch dense graded aggregate base (DGAB) over an additional 9-inch aggregate subbase. The original dual-lane facility was constructed in 1942 and two additional lanes were added in 1964.

In 1982, the pavement was C&S and overlaid with 4.8 inches of AC. A CMI hydraulic stamper was used to crack the pavement. The specified crack pattern was a minimum 2- by 2-foot and a maximum of 4- by 4-foot. A vibratory pneumatic tired roller weighing not less than 12 tons was used to seat the pavement. The project also included an uncracked control section with a 4.8 inch AC overlay. The C&S cost was $0.75 per square yard. The current ADT is 20,400 with 22.8 percent trucks.

After 4 years, no reflective cracks were observed. The C&S section and the control section are performing the same.

c. SR-99 Kern County south of Bakersfield

SR-99 is a 9-inch plain jointed, undoweled, PCCP. The pavement is 36 feet wide (three lanes) with AC shoulders. The "inside" two lanes were constructed in 1956 on an asphalt treated base (ATB). The "outside" lane (lane used for comparison purposes) was constructed in 1968 on a CTB. The C&S project, completed in June 1983, was an experimental project with seven 600-foot test sections:

Section	Description
A.	Control - 3.6 inch overlay no fabric
B.	Crack and seat, seated with vibratory sheepsfoot roller, 3.6 inch overlay
C.	Control - 3.6 inch overlay with fabric
D.	Crack and seat, seated with rubber tired roller, 3.6 inch overlay
E.	Crack and seat, seated with a vibratory sheepsfoot roller, 3.6 inch overlay

F. Crack, not seated, 3.6 inch overlay

G. Crack and seat, seated with a vibratory sheepsfoot roller, 3.6 inch overlay

The C&S cost was $1.60 per square yard.

(1) In the control section (Section A; no C&S, no fabric), 100 percent of the transverse joints had reflected through the overlay with low severity cracks.

(2) In the other control section (Section C; no C&S, with fabric), approximately 50 percent of the transverse joints had reflected through with low severity cracks.

(3) Sections B, D, E, F, and G all involved C&S and exhibited no reflective cracking.

(4) All of the cracking exhibited (Sections A & C) was in the right lane only. All cracks extended no further than the lane joint with an intersecting short longitudinal reflective crack at the joint, forming a "T." This was probably due to the different pavement age and base type.

(5) Deflection testing indicated generally higher deflections after the seating operation than just after cracking. A 13-ton roller was used with 10 passes.

In summary, after 3 years the C&S sections were exhibiting no reflective cracks and were outperforming both of the control sections.

d. Others

A number of other C&S projects were reviewed. Because there was not a true control section for comparison purposes and there was no distress evidenced on either the C&S or the normal overlay portions, these projects are summarized in one discussion.

Route	County	ADT (% Trucks)	Built	Total Overlay Thickness	C&S Cost Per Sq. Yard
I-5	Shasta	25,600 (23)	6/83	5.4 inches	0.75
I-580	Alemeda	56,000 (16)	3/84	4.2 inches	0.80
I-680	Contra Costa	152,000 (4.9)	11/83	4.8 inches	0.55
I-680	Contra Costa	157,000 (4.6)	10/83	3.4 inches	0.85
I-680	Contra Costa	69,000 (6.7)	11/83	4.2 inches	0.60

The projects consisted of 8-inch JPCP on 4-inches of CTB. All of these projects used a fabric interlayer between AC overlay courses and used the same specifications for C&S calling for 4- by 6-foot cracking pattern. These projects only called for C&S in the outer lane(s).

2. Michigan

a. US-10 in Clare County

The original pavement opened to traffic in the mid-1930's was a widened edge (9"-7"-9") jointed reinforced concrete pavement (JRCP). Joints were undoweled with a 60-foot spacing. The original PCCP was overlaid with approximately 4 inches of AC in 1960.

The 8-mile rehabilitation project, completed in October 1983, consisted of milling off the existing bituminous overlay, C&S the pavement, and overlaying with approximately 2 1/4 inches of AC. The pavement was cracked into 18- by 18-inch pieces and seated with a 50-ton vibratory steel wheel roller. The type of breaker was not specified. The C&S cost was $0.20 per square yard. Longitudinal edgedrains were added in select locations. A control section was not built. The current ADT is 1410 with an average of about 120 ESAL's/day since the rehabilitation.

(1) Nearly all transverse joints had reflected through the 2 1/4-inch overlay. The reflective cracks are primarily medium in severity. In addition, intermediate transverse cracks have also reflected.

(2) Less than 5 percent of the longitudinal lane joint has reflected through.

(3) Some minor rutting (1/4 inch) of the asphalt surface is evident.

(4) The ride quality on this project was very good.

b. US-23 in Monroe County

This was an experimental C&S project of approximately 1 1/4 miles within an overall 8-mile long overlay project. US-23 is a 4-lane freeway section with an original 9-inch JRCP with 99-foot doweled joint spacing.

The C&S experimental project, completed in 1983, consisted of 24-, 36-, and 48-inch cracking patterns plus control sections (no cracking), and two overlay thicknesses of 440 and 660 pounds per square yard (approximately 4 and 6 inches). A whip hammer was used to crack the pavement and a 50-ton rubber- tired roller was used to seat the pavement. The C&S cost was $0.19 per square yard. The current ADT is 11,350 with a daily loading of about 3,800 ESAL's per day.

(1) In all four of the comparisons (three different crack patterns and control section) the 660 pounds per square yard overlay (6 inches) had less reflective cracking than the 440 pounds per square yard (4 inches) overlay.

(2) Generally, the least amount of reflective cracking within the C&S sections occurred in the section with the 48-inch crack pattern.

(3) The test section with the least cracking (best condition) was the 660 pounds per square yard control section (no C&S) followed closely by the section with 660 pounds per square yard and the 48-inch crack pattern.

(4) The project showed no signs of distress, other than low severity reflective cracks.

3. Minnesota

a. T.H. 169, Scott County

This project is on T.H. 169 from 0.55 miles south of Belle Plaine's city limits to County Road 66. The original project was constructed in 1956 and consisted of a widened edge (9"-7"-9") non-reinforced PCCP. The joints were undowled with 20-foot spacing.

The rehabilitation project, completed in 1982, consisted of three 1,000-foot sections. One section had a 3-foot crack spacing with no crack closer than 5 feet from a joint or existing transverse crack, one section was cracked at 1 1/2-foot intervals, and the other section was not cracked. A spade type breaker was used to crack the pavement. A 30-ton rubber-tired roller was used to seat the pavement.

The three sections were overlaid with 5 3/4 inches of AC. The C&S cost was $50 per road station ($0.18 per square yard). The current ADT is 10,627.

The section with 3-foot crack spacing was exhibiting random reflective cracks at the joint and minor raveling. The section with the 1 1/2-foot crack pattern and the control section had low severity reflective cracks.

b. T.H. 60 and T.H. 169, Blue Earth County

This project is on T.H. 60 and T.H. 169 near the city of Mankato. The original project was constructed in 1961 and consisted of an 8-inch reinforced PCC pavement over 5 to 9 inches of aggregate base. The pavement was 25 feet wide and the joints were doweled with a 40-foot spacing.

The rehabilitation project, completed in 1982, consisted of eight 1,000-foot test sections. Test sections 1, 2, 5, and 6 were cracked with a spade type breaker. Test section 8 was cracked with a roller breaker. All the sections were seated with a 30-ton pneumatic-type roller. Each section was overlaid with a 6-1/4 inches of AC. The C&S cost was $55 per road station ($0.21 per sq. yd.) The current ADT is 8,454. A summary of the test sections follow:

Test Section	Rehabilitation
Section 1	3-foot crack spacing and edgedrains
Section 2	3-foot crack spacing, no edgedrains
Section 3	No cracking, no edgedrains
Section 4	No cracking, edgedrains
Section 5	1.5-foot crack spacing, edgedrains
Section 6	3-foot crack spacing, no edgedrains,
Section 7	Edgedrains, saw cut construction
Section 8	Edgedrains, cracked with pavement roller breaker

To date there has been very little difference in the performance of the test sections. Each section exhibited reflective cracks approximately every 40 feet (at each joint).

c. T.H. 71, Kandiyohi County

This project was the first C&S project in Minnesota and was completed in 1976. The original roadway structure was a widened edge (9"-7"-9") non-reinforced concrete pavement 22 feet wide with a continuous longitudinal centerline joint and undoweled transverse joints constructed every 15 feet. The surface had spalled at some of the joint locations and maintenance crews had patched these areas with bituminous mixture.

The rehabilitation called for a 6-inch thick AC overlay with the thickness being increased to 7 1/2 inches at some locations. The in-place PCC panels were cracked with a vehicle-mounted spade type breaker at the mid and quarter points thereby reducing the size of the PCCP to pieces about 3 3/4 by 11 feet. A control section of uncracked in-place PCCP with a 7 1/2-inch overlay was constructed to use as a comparison to the broken section. The overlay consisted of 3/4-inch plant-mixed AC wearing course, 1 1/2-inch plant-mixed AC binder course, and either 3 3/4 or 5 1/4 inches of plant-mixed AC base course depending on the location of the overlay. The C&S cost was $70 per road station ($0.26 per square yard). The current ADT is 3,974.

The 1981 final report by the Minnesota Department of Transportation(1) states, "the cracking of the in-place PCCP did reduce the amount of reflective cracking in comparison to similar sections where the PCCP was not cracked."

However, during our review, there were reflective cracks throughout the project. Thus, it appears that C&S did delay reflective cracks for the first 5 years, but after 10 years there was little or no difference in the performance of the C&S section and the control section.

4. Wisconsin

a. I-94, Eau Claire County

The original pavement, constructed in 1967, consisted of 9 inches of reinforced concrete with a 6-inch aggregate base and a 12-inch granular subbase. The joints were doweled with 80-foot spacing.

The rehabilitation project was completed in 1983. A pile drive hammer was used to crack the pavement with a maximun pattern of 18 inches. A 50-ton vibratory roller was used to seat the cracked pavement. The C&S cost was $0.30 per square yard. The current ADT is 16,000. The project consisted of the following:

Section	Overlay Thickness	Performance
Control	4 inches	Reflective cracks every 80 feet, some edgeline cracks
C&S #1	5½ inches	Random centerline reflective cracks
C&S #2	7 inches	Very few small reflective cracks
C&S #3	4 inches	Random edgeline and centerline reflective cracks

The C&S sections with the 5 1/2-inch and the 7-inch overlays were performing slightly better than the C&S section with the 4-inch overlay and the control section.

b. USH 14, Dane and Rock Counties

This was the first C&S project in Wisconsin and was completed in 1980. The original 9-inch non-reinforced PCCP pavement on a 9-inch aggregate base was constructed in 1952. The joints were undowled with 20-foot spacing.

The rehabilitation project, completed in 1980, was 6 miles in length. The pavement was cracked with hydro-hammer type breaker into pieces not exceeding 1 square yard in area. The cracked pavement was then rolled with a 50-ton pneumatic roller and overlaid with 4 1/2 inches of AC. The control section was not cracked and had a 4 1/2 inch AC overlay. The C&S cost was $0.45 per square yard. The current ADT is 4,000.

There were reflective cracks throughout the project and there was no difference in the performance of the C&S section and the control section.

c. STH 140, Rock County

The original project, a 9-inch non-reinforced PCCP with a 9-inch aggregate base, was constructed in 1931. The joints were undoweled with 20-foot spacing.

The C&S project, completed in 1982, required the pavement to be broken into pieces having a maximum dimension of 12 inches with a pile drive hammer and seated with a 50-ton vibratory roller. The control section and the C&S section were each overlaid with 4 inches of AC. The C&S cost was $0.35 per square yard. The current ADT is 2,000.

There were reflective cracks throughout each section with no difference noted in the performance.

5. South Dakota

a. US Route 18, Lincoln County

The original project consisted of mesh reinforced PCCP that was a widened edge (9"-6"-9") section, 20 feet wide, with a 6-inch aggregate base. The joints were undoweled with 20-foot spacing. The original construction was completed in 1930.

This rehabilititation project was completed in 1982. A total of 3.89 miles east and west of Canton was C&S and the 2-mile section through the town of Canton was just overlaid. A spade type breaker was used to crack the pavement at 5 foot intervals and a vibratory steel wheeled roller was used to seat the cracked pavement. The C&S section was overlaid with 3 1/2 inches while the non-C&S section had a 2-inch AC overlay. The C&S cost for this project was $4,000 per mile ($0.20 per square yard). The current ADT is 3,466 with 8.8 percent trucks.

There were reflective cracks about every 40 feet throughout the project. However, there were a few more cracks in the non-C&S section which is expected since it received 1 1/2 inches less AC.

b. US Route 50, Clay and Union Counties

The original project consisted of a mesh reinforced PCCP with a widened edge (9"-6"-9") section on a 6-inch aggregate base that was 20 feet in width. The original construction was completed in 1938. The joints were not doweled.

The C&S project was completed in 1980. It consisted of breaking the 40-foot panels at the quarter points with a spade type breaker, seating the pavement with a vibratory steel wheeled roller, and overlaying with a total of 4 1/2 inches of AC. There was no control section on this project. The C&S cost was $4,000 per mile ($0.20 per square yard). The current ADT is 1,492 with 8.8 percent trucks.

Approximately 90 percent of the project had centerline cracks. There were also random transverse and longitudinal cracks throughout the project.

c. US Route 14, Beadle County

The original construction consisted of a 22-foot wide, 8-inch thick mesh reinforced PCCP on a 6-inch aggregate base that was constructed in 1947. The panels were 15 feet long and the joints were not doweled.

The C&S project was completed in 1979. The 15 foot panels were cracked at 5-foot intervals with a hydro-hammer. A loaded scraper was used to seat the cracked pavement. A 500-foot section of the pavement was left uncracked to serve as the control section. The C&S and the control sections were overlaid with 4 1/2 inches of AC. The cost of C&S on this project was $3,258.90 per mile ($0.23 per square yard). The current ADT is 2,122 with 13.4 percent trucks.

There were random cracks observed at the joints throughout the project with little or no difference noted between the control and the C&S sections.

6. Florida

a. I-4, Hillsbourough County

The original pavement was a 9-inch plain jointed undoweled (except near expansion joints) PCCP with a 20-foot joint spacing on 12-inch cement stabilized base.

The rehabilitation project was completed in 1979. Four test sections were set up to evaluate C&S and two types of fabric to reduce reflective cracking. A drop hammer was used to crack the pavement into 36-inch maximum size pieces. Vibratory compacting equipment or traffic rollers weighing at least 15 tons were specified as equipment to seat the cracked pavement. All sections were overlaid with a 100 pound per square yard (approximately 1 inch) AC leveling course, 2 inches of AC binder, and a 5/8-inch open graded friction course. All sections also received underdrains.

The following is a breakdown of the performance of each section made by the Florida Department of Transportation in March 1986.

Section	Description	Percent Reflected Joints: Rt. Edge	Lt. Edge	Center Longitudinal	Transverse
A	Control with underseal No Fabric	100	50	0	94
B	Crack and Seat No Fabric	87	10	0	22
C	Control with underseal and fabric	100	80	35	72
D	Control with underseal and fabric	80	36	35	35

7. Indiana

I-74 Montgomery/Boone County, a length of 12.4 miles.

The original pavement was a 10-foot reinforced (welded wire) and doweled PCCP on about 6 inches of open graded aggregate subbase. Contraction joints were spaced at 40-foot intervals. Longitudinal edgedrains were provided in the original construction. The pavement was very deteriorated prior to the rehabilitation with 100 percent of the slabs having intermittent cracking at a rate of about 45 feet of cracking per 100 square feet of pavement and about 22 breakups per 100 square feet. Every joint was "D" cracked.

This rehabilitation project was completed in 1984 and consisted of the following sections:

Sections	Description
A.	Asphalt underseal with 4 1/4 inch asphalt overlay
A.1	Same as A with fiber reinforced asphalt base layer
A.2	Same as A with fiber reinforced asphalt base and binder layers
B.	Cracked and seated with 5 1/2 inch AC overlay
B.1	Same as B with fiber reinforced asphalt base layer
B.2	Same as B with fiber reinforced base and binder layers
C.	Cracked and seated with 6 1/2 inch AC overlay
D.	Cracked and seated with 8 1/2 inch AC overlay

The C&S sections used two types of pavement breakers, a whip hammer and a drop hammer. The cracks were required to be mainly transverse, spaced 18 to 24 inches apart. A 50-ton rubber-tired roller was used to seat the pavement. The C&S cost was $0.64 per square yard.

Since the overlay thickness of the "control" does not match the C&S, a direct comparison is not possible. The performance results of the 5-inch overlay in the C&S section are compared below with the 4 1/4 inch "control" overlay.

a. There were no reflective cracks in the 6 1/2- and 8 1/2-inch overlaid C&S pavements. (sections C&D)

b. Only a couple of reflective cracks were observed in the 5-inch overlaid pavements (sections B, B1, B2) which amounted to about 1 percent of the joints.

c. About 40 percent of the transverse joints in the 4 1/4-inch "control" pavements (sections A, A1, A2) had reflected through.

d. All cracks observed were medium in severity and followed a "jagged line pattern" across the pavement at the joint.

e. There were isolated "blow-up" areas in both the control and C&S sections.

f. There was one area about 1/2-mile long of the 4 1/4-inch overlay control sections that showed no reflective cracking. The lack of reflective cracking in this one area could not be readily explained and is not indicative of the "control" sections in the project.

The 1986 Initial Construction and Interim Performance Report from the Indiana Department of Highways(2) concludes in part... "the drop hammer was the most effective machine for producing regular transverse cracks in the pavement. Cracking reduced the strength of the concrete slab without decreasing the subbase support. Rolling with the 50-ton roller decreased both the slab strength and subbase support. Therefore, a heavy roller should not be used as it does not seat the pavement, but rather unseats it."

8. Tennessee

SR-5 Bypass, Madison County

The existing pavement was a 9-inch PCCP on a 6-inch CTB, with no dowels and a 25-foot joint spacing.

The C&S with overlay was completed in November 1983. It consisted of cracking the slab from 18- to 24-inch pieces, seating with a 50-ton pneumatic-tired roller, and overlaying with 5 3/4 inches of AC. The control section had undersealing with fly ash/cement grout, full-depth joint repair, joint resealing, and a 5 3/4-inch overlay. The existing pavement was in fair condition with less than 5 percent of the slabs with cracks. The C&S cost was $0.40 per square yard.

a. About 20 percent of the transverse joints had reflected through the control section overlay with primarily low severity cracking.

b. About 3 percent had reflected through in the C&S section.

c. There were a few isolated locations where longitudinal cracking appeared in the wheel paths of the C&S section.

F. DISCUSSION

1. Of the 22 projects reviewed, only four projects showed appreciably less reflective cracking in the C&S sections than in the control sections. To quantify the benefits of C&S, a measure of the difference in the percent of transverse joints which had reflected through the overlay was employed. Observations made during this review coupled with previous State condition surveys, where available, indicated a reduction in the percent transverse joints reflecting through the overlay during the first few years when C&S is applied. However, after 4 to 5 years the C&S sections generally have approximately the same cracking as the control sections. Therefore, it can be concluded that overall, C&S appears to

provide benefits under some conditions by delaying, not eliminating, reflective cracking.

2. The two projects where the C&S sections performed best were:
   a. SR-99 near Bakersfield, California
   b. I-4 near Tampa, Florida

   Because of the notable difference in the percent of transverse joints reflecting through between the C&S and the control sections on these projects, similarities were investigated. It is believed that the following combination of conditions had the greatest impact on the success of these two projects.

   a. Strong base (cement-treated)
   b. Small changes in seasonal temperatures
   c. Non-reinforced pavement

   These similarities tend to indicate that C&S works best under the same limited conditions as other methods used to reduce reflective cracking (pavements that tend to have little vertical and horizontal movement). Small changes in seasonal temperatures understandably result in less thermal movement in the slab, thereby reducing tensile stress in the AC overlay and the possibility of reflective cracking. A strong base should help in reducing the vertical shear stresses in the overlay. With non-reinforced pavements, the aggregate interlock of the crack interface is the controlling factor in resisting differential deflection or vertical movement. A strong base helps maintain uniform support and should minimize differential deflections of the individual pavement pieces.

In addition, non-reinforced pavements should provide better performance since the presence of reinforcing steel in a slab will tend to inhibit the development of cracks which penetrate all of the way through the slab. Even when the pavement is cracked full depth the steel will tie the sections together concentrating the thermal movement at the original joints which should result in reflective cracking. Non-reinforced pavements generally have shorter slab lengths than reinforced pavements. This reduces the thermal movement at the joints and, therefore, should reduce reflective cracking.

3. The reduction of the structural capacity of the existing pavement appears to be an undesirable feature of C&S. The size of the cracked sections have a direct relationship to structural capacity.

The 1986 AASHTO Guide for Design of Pavement Structures includes a methodology for overlay of C&S pavements. Using this methodology, the suggested structural layer coefficients (indication of carrying capacity per inch of pavement) of the C&S pavement are as follows:

Crack Spacing	Structure Layer Coefficient
1 Foot	0.25
2 Feet	0.35
3 Feet	0.45

A research report(5), completed for the National Asphalt Pavement Association, concluded through back calculation of deflection testing performed on Minnesota's C&S projects that the structural layer coefficients for the C&S project test sections ranged from 0.21 to 0.53. The crack spacing and degree of cracking appeared to be related to the structural layer coefficients. This tends to support and verify the values used in the AASHTO Guide.

Since the structural capacity of the existing pavement is reduced by cracking, more overlay thickness is required to maintain the same structural number as the non-cracked pavement. Using an overlay analysis such as AASHTO would typically result in the need for up to 3 inches to maintain equivalent structural capacity.

The additional cost of: 1) the additional overlay thickness; 2) the cracking and seating; and 3) other required work such as shoulder and guardrail raising, must be evaluated to determine if these costs are justified.

Based on this review and the limited field performance data available to date, it appears these extra costs may not be justified since the condition of the C&S and control sections seemed to be the same after some period of time on most of projects reviewed.

One project where this type of comparison is possible is on US 23 in Michigan. This project had two overlay thicknesses, 440 pounds per square yard and 660 pounds per square yard (approximately 4 and 6 inches) on both the C&S and the control. The extra 2 inches alone has given added performance life because the amount of reflective cracking is much less for the thicker overlay. The C&S with the thicker overlay is performing no better than its control section which indicates no benefit can be seen at this point.

Other C&S projects where various overlay thicknesses were constructed are:

Wisconsin I-94:

	Section	Overlay Thickness
a.	Control	4 inches
b.	C&S #1	5 1/2 inches
c.	C&S #2	7 inches
d.	C&S #3	4 inches

During the review, 3 years after construction, it was observed that the sections b. (C&S-5½") and c. (C&S-7") were performing slightly better than sections d. (C&S-4") and a.(Control-4").

Indiana I-74:

	Sections	Overlay Thickness
a.	Control	4 inches
b.	C&S	5 inches
c.	C&S	6 1/2 inches
d.	C&S	8 inches

At the time of the review, 2 years after construction, there were no reflective cracks in Sections c and d indicating more time is bought by the additional AC thickness.

4. Very little deflection testing has been performed on C&S projects. Only two of the projects reviewed had completed research in this area. The following is a general description of the results of that research.

Indiana, I-74: A Dynaflect was used to measure deflections. Deflection measurements were made before cracking, immediately after cracking, and after the seating operation. The effectiveness of the seating operation was tested after three passes of a 50-ton rubber tired roller as required in the specifications. Test data was also obtained on seven subsections after a variable number of passes of the roller. The average increase in deflection per pass of the seating roller was:

2.3×10^{-5} inch/pass for No. 1 sensor

0.8×10^{-5} inch/pass for No. 5 sensor

Since the deflection increased with each pass of the roller for both sensors, the concrete slab and the subbase lost strength with each pass. The research report states "... the heavy roller caused the slab pieces to unseat rather than to seat as was originally intended. This means that the heavy roller should not be used to attempt to seat the cracked slab pieces."

California, SR-99: Deflection testing was done with the Benkleman Beam and an 18 kip axle load. Deflection measurements were taken before C&S, after cracking, and after seating. The results of the testing are summarized below.(4) Rolling was performed with a 13-ton roller.

After Breaking/Before Seating

Change in Deflection	Number of Joints	Amounts
Reduced	36 of 39 (92%)	Average = 0.006 inches
Increased	1 of 39 (3%)	Average = 0.001 inches
Unchanged	2 of 39 (5%)	-----

After Seating

Change in Deflection	Number of Joints	Amounts
Reduced	9 of 35 (26%)	Average = 0.001 inches
Increased	14 of 35 (40%)	Average = 0.001 inches
Unchanged	12 of 35 (34%)	-----

The results of these two projects cast doubt on the need for seating after cracking. More research is needed in this area.

5. A review of the two projects where a direct comparison of the cracking pattern is possible, Michigan U.S. 23 and Minnesota 60/169, reveals that the larger crack spacing generally performed better than the smaller crack spacing. This would

be expected since for the same overlay thickness, the larger crack spacing is structurally superior to the smaller crack spacing.

Figure 1 shows the results of specific research by Michigan and Minnesota which compared performance of different cracking patterns. In both cases, the larger crack patterns performed better than the smaller crack patterns. Line "0" on Figure 1 is the performance of the control section. Any value above "0" indicates better performance and values below "0" means worse performance.

CRACK PATTERN COMPARISON

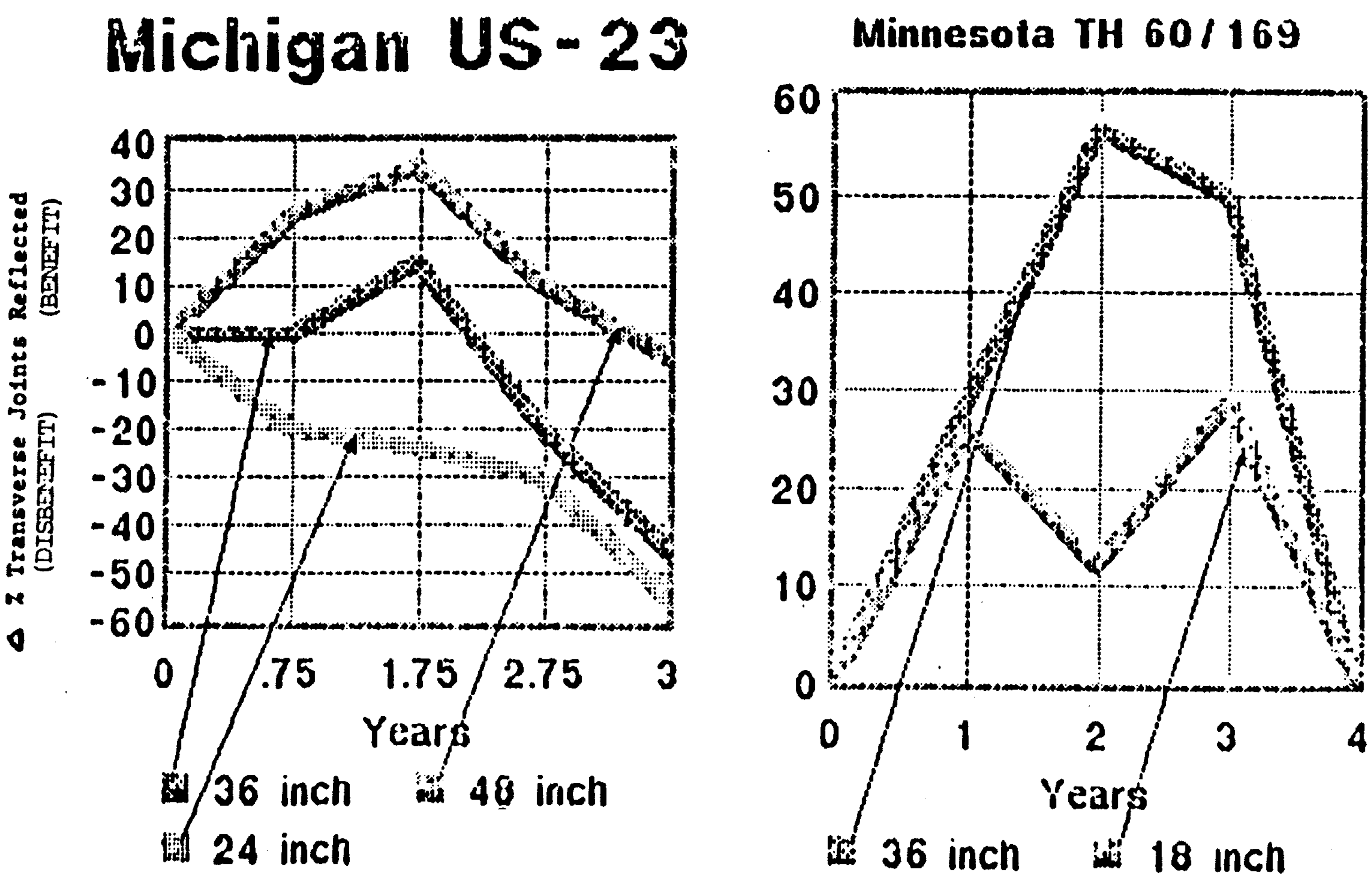


Figure 1

STATE	PROJECT	REINFORCE	DOWELS	JOINT SPACING	PAVEMENT THICKNESS	BASE TYPE	BREAKER TYPE	BREAK PATTERN	ROLLER TYPE	ROLLER WEIGHT	OVERLAY (INCHES)	YEARS	COST	REMARKS
FLORIDA	I-4	NO	NO	20	9"	CTB	DROP HAMMER	36" MAX	OPTIONAL	15T	3 5/8	7	-	C&S PERFORMING BEST
CALIFORNIA	SR-99	NO	NO	-	9"	CTB	DROP HAMMER	4' X 6'	VIB & RUBBER	13T	5	3½	1.60	NO CRACKS IN C & S
CALIFORNIA (ALEMEDIA & CONSTRA COSTA)	I-80	NO	NO	-	8	CTB	PILEDRIVE HAMMER	3' X 4'	VIBRATORY	12T	3½, 5	4	12.50	NO DIFFERENCE IN PERFORMANCE
CALIFORNIA (YOLO COUNTY)	I-80	NO	NO	15	9	AC	HYDRAULIC STAMPER	2' X 2'	VIBRATORY	12T	4.8	4	0.75	NO DIFFERENCE
TENNESSE	SR-5	NO	NO	25	9"	CTB	PILE DRIVER	18"-24"	RUBBER	50T	5 3/4	3	0.40	C&S SLIGHTLY BETTER
INDIANA	I-74	YES	YES	40	10	AG	WHIP & DROP HAMMER	18"-24"	RUBBER	50T	5	2	0.64	C&S MARGINALLY BETTER
SOUTH DAKOTA	U.S. 18	YES	NO	20	9-6-9	AG	SPADE	5 FEET	VIBRATORY	-	3½	4	0.20	3½" C&S SLIGHTLY BETTER THAN 2" CONTROL
SOUTH DAKOTA	U.S. 50	YES	NO	40	9-6-9	AG	SPADE	10 FEET	VIBRATORY	-	4½	6	0.20	NO CONTROL SECTION
SOUTH DAKOTA	U.S. 14	YES	NO	15	8	AG	HYDRO-HAMMER	5 FEET	LOADED SCRAPER	-	4½	7	0.23	NO DIFFERENCE BETWEEN C&S AND CONTROL
MINNESOTA	TH 169	NO	NO	20	9-7-9	AG	SPADE	18", 36"	RUBBER	30T	5 3/4	4	0.18	LOW SEVERITY CRACKS IN EACH SECTION
MINNESOTA	TH 60/169	YES	YES	40	8"	AG	SPADE	18", 36"	PNEUMATIC	30T	6½	4	0.21	NO DIFFERENCE IN PERFORMANCE
MINNESOTA	TH 71	NO	NO	15	9-7-9	-	DROP HAMMER	3 3/4'	RUBBER	-	6, 7½	10	0.26	AFTER 10 YEARS PERFORMANCE THE SAME
WISCONSIN	I-94	YES	YES	80	9	AG	PILEDRIVE HAMMER	18"	VIBRATORY	50T	4,5½,7	3	0.30	THICKER OVERLAYS PERFORMING SLIGHTLY BETTER
WISCONSIN	USH14	NO	NO	20	9	AG	HYDRO-HAMMER	"36 X 36"	PNEUMATIC	30T	4½	6	0.45	NO DIFFERENCE IN PERFORMANCE
WISCONSIN	STH140	NO	NO	20	9	AG	PILEDRIVE HAMMER	12"	VIBRATORY	50T	4	4	0.35	PERFORMANCE IS THE SAME
MICHIGAN	U.S.10	YES	NO	60	9-7-9	-	NOT SPECIFIED	"18 X 18"	VIBRATORY	50T	2½	3	0.20	NO CONTROL SECTION REFLECTIVE CRACKS IN C&S
MICHIGAN	U.S.23	YES	YES	99	9	SELECT MATERIAL	WHIPHAMMER	24, "36", "48"	RUBBER	50T	4,6	3	0.19	6" C&S & CONTROL PERFORMING THE SAME

G. References

1. Minnesota Department of Transportation, Research and Development, Crack Reflectançe on Bituminous Overlaid PCC Pavement (August 1981).

2. Indiana Department of Highways, Division of Research and Training, Initial Construction and Interim Performance Report (September 1986).

3. State of Florida Department of Transportation, Memorandum Inspection of Asphalt Over Concrete, Test Section Located on I-4 in Hillsboro County, (March 1986).

4. California Department of Transportation, Memorandum, Report of Construction, "Effects of Slab Breaking and Seating on Differential Vertical Movement at PCC Slab Joints and Cracks."

5. Midwest Pavement Management, Inc., Structural Evaluation of Crack and Seat Overlay Pavements, (in Minnesota) (September 1986).

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject: Technical Paper - Saw and Seal Pavement Rehabilitation Technique

Date: FEB - 2 1988

From: Chief, Pavement Division

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Division Administrators
(Pavement Specialists)

The purpose of the attached technical paper is to provide practicing pavement technologists with a brief summary of experience on the technique of saw and seal.

The sawing and sealing technique involves the marking of the existing transverse joints of a PCC pavement on the surface of the AC overlay. Next, a saw is used to cut a joint into the asphalt surface, directly over the existing transverse joint. This produces a straight, neat joint in the overlay, which establishes a stress relief plane. The joint is then sealed and maintained as a normal pavement joint.

Currently underway is a FHWA Research and Development Administrative contract, "Performance/Rehabilitation of Rigid Pavements." The contractor has recently completed the field survey to evaluate performance of saw and seal projects. These projects range in age from 3 to 10 years old and are located in 6 States. The preliminary review of the data gathered during the field survey indicates that projects incorporating the saw and seal technique out perform those not using the technique. The research contractor is now entering the field survey data into a data base for further analysis. A more comprehensive report, including a chapter for the Pavement Rehabilitation Manual is expected at the end of the year.

Because of the good performance observed from the saw and seal technique, we are providing the attached technical paper as interim state of the practice information. We can anticipate minor modifications to the current procedures upon completion of detailed analysis of the research data and further experimental usage.

We recommend that you and your staff promote the further usage of this technique during the forthcoming construction season. The Pavement and Demonstration Projects Divisions would welcome any data as it becomes available. If needed, some limited funding is available, on a first come first serve basis, for the evaluation and reporting on the performance of recently completed or planned saw and seal projects under Experimental Project No. 9, Pavement Rehabilitation Techniques.

Norman J. Van Ness

Norman J. Van Ness

TECHNICAL PAPER 88-01 — Saw and Seal Pavement Rehabilitation Technique

I. BACKGROUND

For the purposes of this technical paper, reflection cracking can be defined as fractures in an asphalt concrete overlay that are the result of, and reflect, the joint pattern in the underlying Portland Cement Concrete pavement, and may be either environmental or traffic induced.

The basic mechanisms generally assumed to lead to reflection cracking are the vertical and horizontal movements of the pavement being resurfaced. Vertical movements are differential movements at the joint in the underlying pavement and are caused by moving loads; horizontal movements are due to expansion and contraction caused by temperature change and/or moisture change. The horizontal movement of slabs under an asphalt overlay causes high tensile stresses in the immediate area over the joint. Particularly during lower temperature, the AC surface stiffens and can withstand only small temperature-induced stresses. In addition to temperature changes in the underlying slab, total movement at the joint is affected by slab length, moisture changes, friction or bonding to the base, and the stiffness properties of the overlying material.

The problem of reflective cracking is one of the most perplexing facing the pavement engineer. There do not appear to be any treatments which can prevent the eventual reflection of existing cracks. One treatment, the sawing and sealing of joints in the overlay above existing joints and cracks, has been demonstrated to effectively control severity and extend the service life of the overlay.

The sawing and sealing technique involves the marking of existing transverse joints on the surface of the overlay. Next, a saw is used to cut a joint into the asphalt surface, directly over the existing transverse joint. The joint should be continued through the shoulders, from outside edge to outside edge. This produces a straight, neat joint in the overlay, which establishes a stress relief plane. The joint is then sealed and maintained as a normal pavement joint.

II. DETAILS OF THE TECHNIQUE

Accurately locating joint -

The most critical step in sawing and sealing the overlay is the process of locating the transverse joints in the existing pavement. Experience has shown that as little as one inch deviation from the existing joint location can cause the joint to reflect through the overlay at its location rather than at the sawed joint. Therefore, extreme care must be taken in locating the existing transverse joints.

sealing operation should follow the sawing. They found that at moderate temperatures the joints did not close in four days, but at higher temperatures, the shoving by traffic did close up the joints.

A practical recommendation would be that the overlay should be sawed before any occurrences of sub-freezing temperatures and that the sealing take place as soon as possible or at least before traffic is allowed on the overlay.

Pre overlay treatments -

The effectiveness of sawing and sealing depends greatly on the condition of the underlying pavement. To obtain the full benefit, only concrete pavements with relatively good joints and no surface deterioration should be selected. Joints wider than 3 inches make it difficult to control reflective cracks. Concrete pavements with numerous full-depth and surface patches, misaligned slabs, and midslab cracking are not candidates for this technique. Consequently, there should be a minimum of pre-overlay treatments.

New York specifications include a requirement that if a full depth patch is wider than 0.5 feet, then an additional saw cut shall be made at the patch interface.

There have been recent applications of saw and seal technology on projects requiring significant joint repair. In one instance, the joints were D-cracked. Consequently, the D-cracked material was milled out 2 inches deep over the joints and backfilled with AC prior to the overlay.

In another instance, the joints were spalled. Again the joints were milled 3 inches deep and backfilled with AC. Both of these installations are relatively new and no significant performance data is yet available.

III. APPLICATIONS AND LIMITATIONS

Jointed reinforced PCC pavements -

All of the saw and seal projects have been on jointed reinforced PCC pavements, with relatively long joint spacing. This raises the question as to the cost effectiveness of the saw and seal technique on plain jointed PCC pavement with a lesser joint spacing.

Northeastern States -

Most of the States using saw and seal are located in the northern tier of the country where there is a potential for sizable temperature related slab movement. Connecticut, Massachusetts, New Jersey, New York, and Pennsylvania have the most experience with the saw and seal technique.

New Jersey - This test section was constructed on US 22 in 1977. The original pavement was a 9 inch jointed PCC. It was overlaid with 2 inches of AC. The joints were sawed 3/8 inch wide and 1/2 inch deep and sealed with rubberized joint sealer.

New Jersey - This test section was constructed on I-80 in 1977. The original pavement was a 9 inch jointed PCC with 78 foot joint spacing. It was overlaid with a 2 inch AC overlay. The joints were sawed 3/8 inch wide by 5/8 inch deep and sealed with hot poured elastic (ASTM D 1190).

New York - This test section was constructed on State Route 5 in 1980. The original pavement was a 9 inch jointed PCC with 90 foot joint spacing. It was overlaid with 2 1/2 inches AC. The joints were sawed 1/2 inch wide by 5/8 inch deep and sealed with hot poured sealant (ASTM D 3405).

New York - This test section was constructed on I-81 in 1984. The original pavement was a 9 inch jointed PCC with 63 foot joint spacing. It was overlaid with 3 1/2 inches of AC and sawed at the joints 1/2 inch wide and 5/8 inch deep and sealed with hot poured sealant (ASTM D 3405).

New York - This test section was constructed on I-87 in 1984. The original pavement was a 9 inch jointed PCC with 60 foot joint spacing. It was overlaid with 4 1/2 inches of AC and sawed at the joints 5/8 inches wide and 5/8 inches deep and sealed with hot poured sealant (ASTM D 3405).

Connecticut - This test section was constructed on I-91 in 1978. It was overlaid with 2 3/4 inches of AC and sawed at the joints 3/8 inch wide and 1/2 inch deep and sealed with hot poured elastic sealant (AASHTO T 187).

Connecticut - This test section was constructed on I-84 in 1982. It was overlaid with 3 inches of AC and the joints were sawed 3/8 inch wide and 1/2 inch deep and sealed with hot poured elastic sealant (AASHTO T 187).

Indiana - This test section was constructed on I-80 in 1986. It was overlaid with 5 1/2 inches of AC. The joints were sawed 1/8 inch wide and 2 inches deep. This was followed by routing 1/2 inch wide by 1 inch deep and sealed with a single component hot poured elastomeric polymer.

V. SAMPLE SPECIFICATIONS

Sample specifications from New York and Indiana are provided as examples of comprehensive specifications on saw and seal projects.

VI. COST DATA

There is limited cost data available, however, reported costs range between $1 and $4 a linear foot for the sawing and sealing technique.

TO:

ENGINEERING INSTRUCTION

NEW YORK STATE DEPARTMENT OF TRANSPORTATION

SUBJECT: SAWING AND SEALING JOINTS IN BITUMINOUS CONCRETE OVERLAYS

Subject Code:

Distribution: 31 Main Office 33 Regions 34 Special

Code: 85-43

Date: 9/12/85

Supersedes:

APPROVED:

M. Tegza

M. TEGZA, Final Plan Review Bureau

On April 15, 1985 EI 85-25 was issued implementing item 18403.2501 for all asphalt overlays effective with the lettings of August 8, 1985. Since then the Materials Bureau has discovered that sometimes the concrete joints are milled to a depth of 3" or more. This results in a total overlay thickness greater than 4½" over the joint.

The current note 1 on page 4 would require a 1/8" wide sawcut over these milled joints but the wording may allow a Contractor to avoid constructing the 1/8" wide sawcut because of the reference to the T&L course.

The new note 1 requires the 1/8" wide sawcut be included whenever the total thick of asphalt concrete over the existing joint exceeds 4½ inches. This change sho eliminate problems interpreting when to sawcut. Also, the minimum depth has been increased to 2½" minimum.

The new item number will be 18403.2502. This will be effective with the letting of January 30, 1986.

ITEM 18403.2502 - SAWING AND SEALING JOINTS IN BITUMINOUS CONCRETE OVERLAYS

If the top course is to be placed the following Spring, due to seasonal paving limitations, all underlying courses shall receive a 1" deep by 1/8" wide sawcut to facilitate and control reflective cracking as well as to provide a means of properly referencing the sawcut to eventually be made in the top course. These sawcuts shall be made in all underlying courses within seven (7) days after the underlying courses are placed and before any evidence of reflective cracking has developed. Sealing of these sawcuts will not be required. Payment for sawcutting all underlying courses shall be included in the unit bid price for sawing and sealing.

Sawcutting of Transverse Joints. The contractor shall sawcut transverse joints to the appropriate dimensions shown in Figure I, based on the existing pavement slab length and new overlay depth. Full depth patches adjacent to joints in the underlying concrete shall have separate sawcuts in the overlay over the patch/slab interface. See Figure II. Sawcuts over patch interface shall conform to Figure I. The sawcut joints shall be directly over the existing portland cement concrete pavement joints and shall be accurately located by a method employing pins and stringline. The pins shall be accurately located prior to paving. Details of the method for locating the sawcuts shall be subject to the approval of the Engineer.

The blade or blades shall be of such size and configuration that the desired dimensions of the sawcut can be made with one pass. Either dry or wet cutting will be allowed. No spacers between blades will be allowed.

The transverse sawcut joints shall normally extend the full width of the pavement and shall extend into the asphalt shoulder to a distance three (3) feet beyond the edge of the underlying portland cement concrete pavement edge, unless otherwise detailed on the plans or in the proposal. Existing transverse joints that are offset at the longitudinal joint by more than 1 inch, measured between the centers of the joint cavities, shall require separate sawcuts terminating at the longitudinal joints.

Cleaning. Dry sawed joints shall be thoroughly cleaned with a stream of air sufficient to remove any dirt, dust or deleterious matter adhering to the joint walls or remaining in the joint cavity. Wet sawed joints shall be thoroughly cleaned with a water blast (50 psi minimum) immediately after sawing to remove any sawing slurry, dirt, or deleterious matter adhering to the joint walls or remaining in the joint cavity. Wet sawed joints shall be blown with air to provide dry joint surfaces prior to sealing.

All sawing slurry from the wet sawing process shall be immediately flushed from the pavement surface. Dry dust and material from the dry sawing process shall be blown or brushed off the pavement surface.

The contractor shall be required to provide protective screening, subject to the approval of the Engineer, if his cleaning operations are capable of causing damage to or interference with traffic in adjacent lanes.

DETAILS FOR TRANSVERSE JOINTS IN ASPHALT CONCRETE OVERLAYS

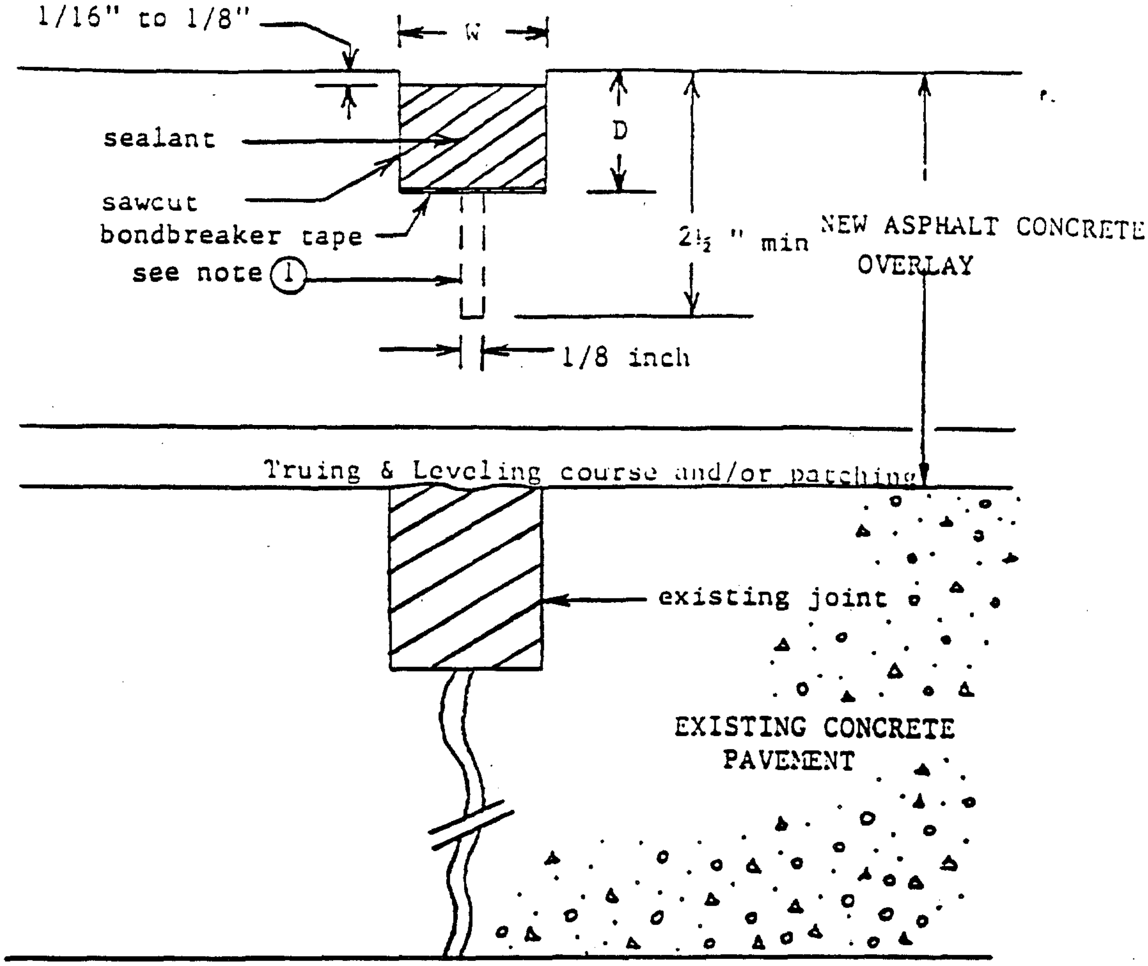
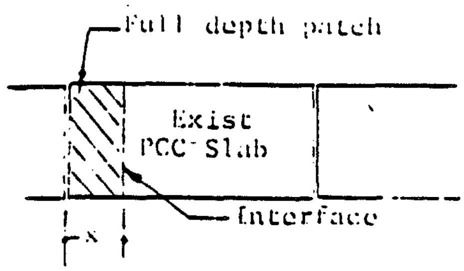


FIGURE I

SAWCUT DIMENSIONS

SLAB LENGTH	W	D
50 Ft or less	1/2 in	5/8 in
51 to 62 Ft	5/8 in	5/8 in
63 to 75 Ft	3/4 in	5/8 in
76 to 87 Ft	7/8 in	3/4 in
88 to 100 Ft	1 in	7/8 in



If "x" is greater than 0.5 ft. a sawcut is req'd at patch/slab interface in the overlay.

FIGURE II PLAN VIEW

NOTE (1)

When the total thickness of asphalt concrete over the existing joint exceeds 4½ inches, an 1/8 inch wide sawcut shall be included in the joint geometrics to a minimum depth of 2½ inches.

3. CONSTRUCTION -

(a) General. Locate and reference the location of each existing transverse joint prior to placement of any bituminous courses. Make all saw cuts directly above the existing transverse joints.

Do not perform saw cutting until the bituminous course has thoroughly cooled. Perform saw cutting within 7 days after placement of the wearing course. Perform this work on all finished overlay areas prior to discontinuing of work due to seasonal paving limitations.

Extend the saw cuts the full width of the pavement including any widening. Provide separate saw cuts in each lane when existing transverse joints are offset more than 1 inch.

If the wearing course is to be placed the following construction season due to seasonal paving limitations, provide a 1 inch deep, 1/8 inch wide saw cut in the last placed bituminous concrete course.

(b) Sawing. When the total depth of overlay exceeds 4½ inches, not including scratch or leveling courses, make a 1/8 inch wide saw cut to a minimum depth of 2 inches or 1/3 of the total overlay thickness.

Saw a reservoir, in the wearing course having a width of ½ inch and a depth of 1 inch. If wet sawing is used, immediately flush the reservoir with water.

(c) Sealing. Do not place sealing material unless the reservoir faces are thoroughly clean and dry. Do not place on the same day as wet sawing. Clean the reservoir by using compressed air immediately before placing sealing material. Use compressed air free of oil, moisture, or any other substance that would prevent bonding of sealing material to the reservoir faces.

Do not place sealing material when the air temperature is less than 40 F. Use heating equipment of an indirect heating type, constructed as a double boiler. Provide positive temperature control and mechanical agitation.

Obtain the safe heating temperature and recommended pouring temperature from the manufacturer's shipping container. Place the material within this temperature range, but as close as possible to the recommended pouring temperature. Maintain a safe heating temperature. Maintain a single material batch at the pouring temperature for no more than 4 hours. Heat material only once.

Fill the reservoir with sealing material to a level 1/8 inch plus or minus 1/16 inch below the pavement surface. Do not allow sealing material to spread over the pavement surface.

4. MEASUREMENT - Linear Foot.

SUBJECT	FHWA NOTICE
HOT AND COLD RECYCLYING OF ASPHALT PAVEMENTS	N 5080.93 October 6, 1981

1. PURPOSE

To present the Federal Highway Administration's (FHWA) position on recycling of asphalt pavements.

2. CANCELLATION

The FHWA Technical Advisory T5040.9 dated February 16, 1979, Hot Recycling of Asphalt Pavement Materials, is cancelled.

3. BACKGROUND

The pressing need to conserve energy and minimize costs in highway construction requires that special effort be made to identify and make the maximum use of procedures that will result in reduced energy usage and minimum cost. Because recycling of asphalt pavements has the potential to be an effective method of conserving energy and materials and reducing costs, it is FHWA's policy that recycled asphalt concrete, defined as asphalt concrete containing salvaged paving materials including the use of suitable reclaimed material from other projects, be allowed for use on all projects. States with insufficient experience to properly evaluate the reuse of these materials should take immediate steps to initiate experimental projects.

4. DEFINITIONS

a. Recycled hot asphalt concrete is an asphalt concrete mix, processed hot in a central plant, which consists of sized salvaged asphalt material, new asphalt, and/or recycling agents and new and/or salvaged aggregates, and meets all standard material and mix specifications for the type of mix being produced.

b. Recycled cold mix is an asphalt concrete mix, processed in a central plant or on the grade which consists of sized salvaged asphalt material, some type of stabilizing agent and new and/or salvaged aggregates. This material meets specifications of an asphalt aggregate base and generally requires that an asphalt surface course or surface seal be used.

5. PAVEMENT DESIGN

a. Recycling should be one of the options considered at the design stage of all rehabilitation projects. Material testing of the old pavement may be necessary to determine that recycling is a practical option. The decision to recycle or to overlay should be based on cost and performance on a life cycle basis rather than initial cost and should be specified by the contracting agency. It is emphasized that alternate bids between recycling and overlay are not recommended.

b. Cracks and material deficiencies in the overlaid pavement will cause reflective cracks and points of weakness to occur in an overlay. Cracks can be eliminated and material deficiencies can be corrected by recycling.

c. Recycled mixes placed experimentally as base layers, top structural layers, and wearing surfaces are still being evaluated and it would be premature to offer definite conclusions on life cycle performance. However, the earliest of those pavements are 5 years old or older and are performing as well as pavements constructed with new materials. While there is limited experience with recycled mixes, it appears that reasonable performance can be obtained.

d. It is reasonable to assume that a recycled layer is structurally equivalent to an equal thickness of new hot mix pavement provided the mix meets all of the laboratory design criteria for a new mix intended to perform the same functions.

e. Only proven methods and materials with which there has been adequate experience to assure success should be used on large projects with high traffic or heavy loading.

6. MIX DESIGN

a. Recommendations for detailed mix design procedures are contained in NCHRP Report 224. Gradation and other material requirements should be the same for a recycled mix as those developed for mixes using all new materials for the same type of pavement.

b. Distress observed on a few projects is directly attributable to improper or poor mix designs represented by low stabilities, uncorrected aggregate stripping problems, and low job achieved densities. These problems emphasize the need for proper mix design and construction control. Research results indicate that testing for water susceptibility is especially important for recycled mixes.

c. Variation in material properties of the pavements to be salvaged should be identified by sampling and a sufficient proportion of new material provided to reduce the variation to an acceptable level. Major changes in mix characteristics for various sections along the same route usually demand separate mix designs.

d. Removal and sizing of salvaged pavement materials have at times created additional minus 200 sieve material. The amount depends on the type and operation of the sizing process and aggregate properties. Final mix design should always be corrected to final properties of the material processed by the actual equipment used on the project. Large amounts of minus 200 sieve material or other gradation deficiencies can be compensated for by limiting the amount of salvaged material used in the recycled mix and varying the gradation of the added new material. Experience has indicated that in most cases crushing the recycled material to a maximum particle size of 2 inches is adequate for hot mix. Additional crushing may result in excess fines.

e. A soft asphalt alone has been used successfully to restore the penetration and viscosity of the reclaimed asphalt binder. A number of commercial recycling agents have also been used when salvaged asphalt binder in the salvaged material was severely hardened. Any proposed softening agent should be tested with the salvaged asphalt for the specific project, in the ratio to be used, to assure the desired properties of the combination are realized.

7. REMOVAL AND SIZING

The type and degree of deterioration in a pavement to be constructed and/or the type of material underlying the pavement will usually determine whether a full or partial depth removal technique is utilized. Full-depth pavement removal and sizing can be accomplished using standard construction equipment such as dozers and loaders and portable or stationary crushers or by milling machines. The latter process, although generally more expensive, allows removal of one lane without disturbing traffic movement on adjacent lanes. Excessive dropoffs can be minimized by milling successive levels to a specific depth. While milling machines usually are specified for partial depth removal, the choice of the method used for full-depth removal will be influenced by economics and maintenance of traffic through construction.

8. EQUIPMENT

Virtually all equipment manufactured today for the production of asphalt concrete can be built to produce acceptable recycled mixes and meet all air quality standards. Existing equipment can be modified at reasonable cost. In hot mix recycling, batch plants are generally limited to the reuse of a maximum amount of 50 percent salvaged asphalt material in a recycled mix, while an upper limit of approximately 70 percent is attainable in some drum plants.

9. SAVINGS

Materials savings are realized from the reduction in new asphalt and aggregate. Energy savings result primarily from reduced aggregate haul and drying, and asphalt transportation. Cost savings are greatly influenced by length of aggregate haul and distance from the plant to the job site. Other factors which have a major influence on bid prices are the degree to which contractors in the area are familiar with and equipped for recycling, the size of the State's present and projected recycling program, and State contract procedures.

10. RECOMMENDATIONS

a. Allow the contractor the use of salvaged asphalt materials and aggregates in the production of asphalt concrete.

b. Allow the contractor to determine the source and amount of salvaged material to be used as long as the mix produced meets all standard material and mix specifications called for in the contract.

c. Require that a revised mix design be submitted and approved prior to changing either the source or amount of salvaged material originally approved.

d. Serious consideration should be given to transferring ownership of all material to be removed to the contractor. This allows the owner agency to receive instant credit, in the form of lower bids, for the value of the salvaged material removed.

e. Do not specify how to remove and size a pavement scheduled for full-depth reconstruction; what type of hot mix plant (batch, continuous or drum) to use; the use of recycling agent--but allow it to be used; and what percentage of salvaged material to be used. All of these will be determined by economics resulting from the competitive bidding process.

f. Recycled hot asphalt concrete should be paid for on the basis of a bid price per ton regardless of the percentage of salvaged material used. This price per ton is also to include the costs of all new additional asphalt, recycling agent, and aggregate.

11. DISCUSSION OF RECOMMENDATIONS

These recommended practices will allow the production of recycled asphalt concrete, if economically feasible, at any time in any location. Because no restrictions are placed on percentages of used salvaged material, a batch plant owner can economically compete with owners of drum plants. If across the board use of salvaged materials is allowed in the production of asphalt concrete, the contracting industry can better justify gearing up for such production and write off the additional plant modification cost over a much larger tonnage basis over a longer period of time than on only one or two projects.

Transferring ownership of all removed salvaged material to the contractor encourages recycling because surplus material can be used in private work at additional savings to the contractor.

12. EVALUATION

Most highway agencies have successfully constructed one or more hot recycling projects and are continuing to develop new projects. These projects have been constructed under NEEP Project 22, Pavement Recycling, distributed by Notice N 5080.64 dated June 3, 1977. Many projects have also been constructed with technical and financial assistance from the Demonstration Projects program. It is recommended that the evaluation of these projects be continued to validate long-term performance projections. Broad participation is needed to provide the data base necessary to require additional projects to be programmed experimental. The projects under evaluation should be representative of recycling procedures adopted by a State which have become routine. When a significantly new or innovative feature is contemplated, or when a project is in a significantly different environment, the highway agency should be urged to designate the project as experimental.

A recycling data bank is being developed under a contract through the FHWA Office of Research that will provide a means of long term evaluation of pavement recycling. The contract is scheduled to be completed in 1982.

R. D. Morgan
Associate Administrator for
Engineering and Traffic Operations

Attachments

REPORT FROM WISCONSIN DIVISION, April 13, 1981

WISCONSIN

1981 RECYCLING PROGRAM

During the first 6 months of 1981, Wisconsin Department of Transportation (WISDOT) let 42 contracts involving recycling of the existing bituminous pavement. The dollar amount of these contracts totaled $40.4 million and included 52 Federal-aid projects. Contractor competition for these contracts has been good with only 1 out of the 26 successful contractors having more than three contracts.

The contracts let to date have provided 696,700 tons of recycled bituminous pavement for paving 418 lane miles. The average bid price for this recycled bituminous pavement has been $8.84 per ton. This is significantly less than the $14.24 per ton average for virgin bituminous concrete pavement. When the savings in asphalt and shoulder aggregate are considered, the savings are almost $8.00 per ton.

In addition to the above tonnage, eight contracts totaling 34,800 tons of single aggregate bituminous surface have been let with the contractor having the option to use recycled or virgin aggregate. Most (6) of these projects were relatively small and provided less than 3,500 tons of bituminous pavement per project. The two larger projects provided 8,800 and 13,350 tons of single aggregate bituminous surface. Five of these contracts with optional recycling were in Milwaukee County.

The contracts let in FY 1981 have also provided for salvaging 496,000 tons of existing bituminous pavement. The average cost of salvaging bituminous pavement has been $4.41 per ton.

In addition to the "normal" recycling type of project, Wisconsin's 1981 recycling program has included three contracts that provide for recycling as part of a sulfur extended asphalt pavement. The cost of the sulfur for these projects has averaged $149 per ton.

One of the major accomplishments in WISDOT's recycling program is the savings in energy, natural resources, and cost. It is estimated that the energy savings this fiscal year is equivalent to 915,000 gallons of gasoline; the aggregate savings is 574,700 tons of aggregate, and the cost savings is $4.8 million.

Reports Dealing with Recycling

NCHRP Synthesis of Highway Practice 54, "Recycling Materials For Highways," 1978.

NCHRP Report 224, "Guidelines for Recycling Pavement Materials," 1980.

American Society of Testing Materials, STP 662, 1976.

Association of Asphalt Paving Technologists, Volume 46, 1977; Volume 48, 1977; Volume 49, 1980.

Proceedings of the National Seminar on Asphalt Pavement Recycling, Dallas-Ft. Worth, Texas, 1980 - Transportation Research Record 780.

The above reports are available at a charge from:

The National Technical Information Service (NTIS)
Springfield, Virginia 22161

Evaluation of Selected Softening Agents used in Flexible Pavement Recycling, FHWA-TS-79-204, 1978.

Hot Recycling - Minnesota - Modified Dryer Drum, FHWA-TS-80-233, 1980

Hot Recycling - Wyoming Dryer Drum, FHWA TS-80-234, 1980.

The above reports are available free of charge from:

Federal Highway Administration
Region 15
Demonstration Projects Division (HDF-15)
1000 North Glebe Road
Arlington, Virginia 22201

REPORTS PREPARED FOR DEMONSTRATION PROJECT NO. 39 RECYCLING ASPHALT PAVEMENTS

FHWA-DP-39-1 - IN-PLACE RECYCLING OF ASPHALT PAVEMENT - REPUBLIC COUNTY, KANSAS - CONSTRUCTION REPORT - Clarence W. Smith - August 1978 - 30 pages

FHWA-DP-39-2 - SURFACE RECYCLING ASPHALTIC CONCRETE PAVEMENT - MC ALLEN, TEXAS - CONSTRUCTION REPORT - Wade D. Barnes and Jack T. Trammell - September 1877 - 58 pages

FHWA-DP-39-3 - WASHINGTON STATE DEPARTMENT OF TRANSPORTATION'S FIRST ASPHALT CONCRETE RECYCLING PROJECT - ELLENSBURG, WASHINGTON - CONSTRUCTION REPORT - R. V. LeClerc, R. L. Schermerhorn, and J. P. Walter - July 1978 - 52 pages

FHWA-DP-39-4 - RECYCLING OF ASPHALT CONCRETE-OREGON'S FIRST HOT MIX PROJECT - WOODBURN, OREGON - INTERIM REPORT - James Dumler and Gordon Beecroft - November 1978 - 56 pages

FHWA-DP-39-5 - PAVEMENT SURFACE RECYCLING ON PARKS HIGHWAY BETWEEN LITTLE SUSITNA RIVER AND WILLOW CREEK - ANCHORAGE, ALASKA - INTERIM REPORT - John W. Henry - February 1978 - 31 pages

FHWA-DP-39-6 - BLEWETT PASS RECYCLING PROJECT - BLEWETT PASS, WASHINGTON - PRELIMINARY REPORT - September 1979 - 57 pages

FHWA-DP-39-7 - MILLING BITUMINOUS SURFACE - ELLENDALE, NORTH DAKOTA - CONSTRUCTION REPORT - September 1978 - 32 pages

FHWA-DP-39-8 - EVALUATION OF RECYCLED BITUMINOUS PAVEMENTS - ELKHART COUNTY, INDIANA - FINAL REPORT - Barry L. Elkin - August 1978 - 60 pages

FHWA-DP-39-9 - RECYCLING OF ASPHALTIC CONCRETE PAVEMENTS - LARAMIE, WYOMING - INITIAL REPORT - Wyoming State Highway Department, Materials Division - February 1979 - 89 pages

FHWA-DP-39-10 - EVALUATION OF RECYCLED ASPHALT CONCRETE PAVEMENTS - KOSSUTH COUNTY, IOWA - CONSTRUCTION REPORT - Richard P. Henely - February 1979 - 52 pages

FHWA-DP-39-11 - RECYCLING ASPHALTIC CONCRETE PAVEMENT - ROSCOE, TEXAS CONSTRUCTION REPORT - Bobby R. Lindley - March 1979 - 142 pages

FHWA-DP-39-12 - EXPERIMENTAL PROJECT SURFACE RECYCLING OF ASPHALT CONCRETE PAVEMENT - NATCHEZ, MISSISSIPPI - PROGRESS REPORT - James D. Webb, Gayle E. Albritton, and Thomas L. Chance

FHWA-DP-39-13 - COLD RECYCLING - MENOMINEE INDIAN RESERVATION WISCONSIN - CONSTRUCTION REPORT - Steve Beckett and Roy J. Calbo - February 1979 - 45 pages

FHWA-DP-39-14 - EVALUATION OF RECYCLED ASPHALTIC CONCRETE - CHESTER, VIRGINIA - CONSTRUCTION REPORT - C. S. Hughes - August 1977 - 26 pages

FHWA-DP-39-15 - INTERIM REPORT ON HOT RECYCLING - Douglas J. Brown - April 1979 - 99 pages (English or Spanish)

FHWA-DP-39-16 - PAVEMENT RECYCLING PROJECT - GILA BEND, ARIZONA - CONSTRUCTION REPORT - Arizona Department of Transportation Research Division - October 1978 - 59 pages

FHWA-DP-39-17 - RECYCLING ASPHALT CONCRETE ON INTERSTATE 80 - GOLD RUN, CALIFORNIA - INTERIM REPORT - R. N. Doty and T. Scrimsher - April 1979 - 134 pages

FHWA-DP-39-18 - RECYCLING OF BITUMINOUS SHOULDERS - FERGUS FALLS, MINNESOTA - INTERIM REPORT - Ronald H. Cassellius and Roger C. Olson - March 1979 - 31 pages

FHWA-DP-39-19 - RECYCLING OF ASPHALT CONCRETE PAVEMENTS - PALM BEACH COUNTY, FLORIDA - INITIAL REPORT - Charles F. Potts and Kenneth H. Murphy - January 1980 - 35 pages

FHWA-DP-39-20 - COLD RECYCLING ASPHALT PAVEMENT - SHERVURNE, VERMONT - INITIAL REPORT - R. I. Frascoia and D. N. Onusseit - January 1979 - 42 pages

FHWA-DP-39-21 - SURFACE RECYCLING OF ASPHALT CONCRETE PAVEMENT - OHIO - PROGRESS REPORT - Willis B. Gibboney - November 1979 - 23 pages

FHWA-DP-39-22 - SURFACE RECYCLING OF ASPHALT CONCRETE PAVEMENTS - FORT MYERS, FLORIDA - INITIAL REPORT - Charles F. Potts and Kenneth H. Murphy - September 1979 - 62 pages

FHWA-DP-39-23 - RECYCLING OF ASPHALT CONCRETE PAVEMENTS - PANAMA CITY, FLORIDA - INITIAL REPORT - Charles F. Potts and Kenneth H. Murphy - December 1979 - 53 pages

FHWA-DP-39-24 - COLD RECYCLING OF PAVEMENT USING THE HAMMERMILL PROCESS - MAINE - FINAL REPORT - David W. Rand - December 1978 - 41 pages

FHWA-DP-39-25 - COWHERD ROAD COLD ASPHALT RECYCLING PROJECT - JACKSON COUNTY, MISSOURI - CONSTRUCTION REPORT - Kirk Phillips - November 1979 - 99 pages

FHWA-DP-39-26 - COLD BITUMINOUS PAVEMENTS RECYCLING - WIBAUX, MONTANA CONSTRUCTION REPORT - John J. Wright - May 1979 - 75 pages

FHWA-DP-39-27 - COLD RECYCLING OF A SOIL-ASPHALT ROADWAY - BEAVER COUNTY, OKLAHOMA - INTERIM REPORT - Jack C. Stewart - April 1980 - 52 pages

FHWA-DP-39-28 - HOT MIX RECYCLING - DURANGO, COLORADO - INTERIM REPORT - Robert F. LaForce - May 1980 - 61 pages

FHWA-DP-39-29 - BITUMINOUS CONCRETE PAVEMENT RECYCLING - INTERIM REPORT - Edgar J. Hellriegel - NORTH BRUNSWICK, NEW JERSEY - July 1980 - 61 pages

FHWA-DP-39-30 - HOULTON - LITTLETON HOT RECYCLING PAVING PROJECT - HOULTON, MAINE - PRELIMINARY & CONSTRUCTION REPORT - D. W. Rand - March 1980 - 61 pages

FHWA-DP-39-31 - HOT RECYCLING OF ASPHALTIC CONCRETE PAVEMENT - BEAVER, UTAH - INTERIM REPORT - Wade B. Beteson - October 1980 - 170 pages

FHWA-DP-39-32 - 1980 PAVEMENT RECYCLING PROGRAM - SPRINGFIELD, MISSOURI - INTERIM REPORT - prepared by Anderson Engineering, Inc. - January 1981 - 75 pages

OTHER RELATED RECYCLING REPORTS

DEMONSTRATION PROJECT NO. 39 - RECYCLING ASPHALT PAVEMENTS - PROJECT STATUS REPORT - February 1979 - 66 pages

RECYCLING OF ASPHALTIC CONCRETE - ARIZONA'S FIRST PROJECT - James A. McGee and A. James Judd - 28 pages

MINNESOTA HEAT TRANSFER METHOD FOR RECYCLING BITUMINOUS PAVEMENT - REPORT ON MAPLEWOOD, MINNESOTA, RECYCLING PROJECT - Richard C. Ingberg, Richard M. Morchinek, and Ronald H. Cassellius - 1977 - 43 pages

EVALUATION OF AIR POLLUTION CONTROL DEVICES FOR ASPHALT PAVEMENT RECYCLING OPERATIONS - PROGRESS REPORT - Richard P. Henely - December 1977 - 47 pages

RECYCLING ASPHALT CONCRETE PAVEMENT - DEPARTMENTAL RESEARCH REPORT NO. 524-1-F - DHT 1-9-76-524-1F - Charles H. Hughes - August 1977 - 145 pages

COLD RECYCLING OF ASPHALT CONCRETE PAVEMENT - EXPERIMENTAL PROJECTS - REPORT NO. 613-1 - B. R. Lindley - October 1975 - 27 pages

RECYCLED ASPHALTIC CONCRETE PAVEMENT - SR-26, SR-100 TO HOLDEN RS-0303(3) - Wade B. Betenson - February 1979 - 94 pages

COLD RECYCLING OF PAVEMENT BY HAMMERMILL PROCESS - INTERIM REPORT - David W. Rand - August 1977 - 82 pages

RECYCLING OF SUBSTANDARD OR DETERIORATED ASPHALT PAVEMENTS - A GUIDELINE FOR DESIGN PROCEDURES - Donald D. Davidson, William Canessa, and Steven J. Escobar - February 1977 - 51 pages

FHWA-DP-PC-1000-1 - PRODUCTION EFFICIENCY STUDY ON PAVEMENT PLANING EQUIPMENT - INTERIM REPORT - David R. Lewis - March 1979 - 58 pages

HOT RECYCLING IN HOT-MIX BATCH PLANTS - National Asphalt Pavement Association - 5 pages

PRODUCING A BITUMINOUS WEARING COURSE BY DRUM MIX RECYCLING (MICHIGAN) - R. B. Moore and R. A. Welke - January 1979 51 pages

BATCH PLANT RECYCLING (MICHIGAN) - John E. Norton - April 1979 - 30 pages

USE OF RECYCLED ASPHALT SURFACE MATERIAL IN THE CONSTRUCTION OF A BITUMINOUS STABILIZED BASE (MICHIGAN) - J. H. DeFoe and G. F. Sweeney - April 1978 - 21 pages

MIXED-IN-PLACE STABILIZATION OF HIGHWAY BASE AGGREGATES AND PULVERIZED BITUMINOUS SURFACING USING ASPHALT STABILIZERS (MICHIGAN) - J. H. DeFoe - March 1977 - 39 pages

RECYCLING OF BITUMINOUS MAINLINE AND SHOULDERS (MINNESOTA) - Roger C. Olson - February 1979 - 26 pages

RECYCLING OF ASPHALTIC CONCRETE PAVEMENTS NO.2 (WYOMING) - Materials Division of Wyoming State Highway Department - 86 pages

RECYCLED COLD-MIX ASPHALT BASE CATOCTIN MOUNTAIN PARK (REGION 15, FHWA) - William F. Bensing - December 1978 - 34 pages

HOT MIX RECYCLING GEORGE WASHINGTON MEMORIAL PARKWAY (REGION 15, FHWA) - Reynaldo Cortez - 31 pages

EXPERIMENTAL TEST SECTION NEAR COVE FORT (UTAH) - Uath Department of Transportation - 59 pages

RECYCLING ASPHALTIC CONCRETE PAVEMENT (TEXAS) - FINAL REPORT (I-20 PROJECT) - Bobby R. Lindley - January 1980 - 4 pages

EVALUATION OF RECYCLED ASPHALT CONCRETE PAVEMENTS (KOSSUTH COUNTY, IOWA) - FINAL REPORT - Richard P. Henely - 30 pages

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Use of Recycled Concrete in Portland Cement Concrete Pavements

Date: JUL 25 [redacted]

From: Chief, Pavement Division
Washington, D.C. 20590

Reply to Attn. of: HHO-12

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

A Pavement Design and Rehabilitation Team Review was recently made of pavements, located in one State, reconstructed with recycled portland cement concrete (PCC) pavement. The purpose of the review was to analyze causes for the transverse working cracks which were developing in many of the slabs.

The pavements were reconstructed in 1984 and 1985 using the recycled existing PCC pavement. The design called for a 10-inch reinforced PCC pavement with plain PCC shoulders and an open-graded granular (unstabilized) subbase. The mainline pavement joints were spaced at 41 feet.

Numerous intermediate cracks with spalling and faulting were observed. Typically there were two cracks per slab, occurring at the third points. These cracks were significantly more severe in the driving lane. A few slabs had also developed one or two additional cracks, some of which showed signs of staining.

Observation of the concrete which was removed from the pavement indicated that some of the recycled material, used as large aggregate in the reconstructed pavement, was mainly mortar with very little if any aggregate.

The team made the following recommendations to this specific State:

1. Based on recent findings, it is our recommendation that recycled PCC pavement not be used as aggregate in reinforced PCC pavements. The wire mesh reinforcement in reinforced pavements is intended to hold cracks close together so that load transfer can be obtained through aggregate interlock. However, the recycled concrete aggregate does not appear to provide adequate aggregate interlock for two reasons. The first is due to its fairly small size; on these projects the top size was 1 inch.

The second is due to portions of this larger aggregate being merely lumps of mortar, which easily grind smooth with pavement deflections caused by traffic loading. Since the recycled pavement does not provide sufficient aggregate interlock, the wire mesh reinforcement is subjected to excessive shearing forces. As a result, the wire mesh ruptures and the crack begins to function as a working joint. Plain PCC pavements are designed not to crack, so aggregate interlock is not a factor, providing dowel bars are properly installed at the joints.

2. If the decision is made to use recycled PCC pavements as aggregate in reinforced pavements, it is recommended that a 3-sized aggregate mix be used, with the recycled PCC pavement serving as the middle-sized aggregate. A larger-sized coarse material should be added to provide the necessary aggregate interlock.

3. The high absorptive level of the recycled aggregate (approximately 6 percent) may have resulted in high drying shrinkage of the concrete. This in turn could cause the cracks to open wider than normal, further reducing load transfer through aggregate interlock. When recycled concrete pavement is used as aggregate, consideration should be given to moistening the aggregate prior to adding it to the mix.

Based on the performance of the recycled concrete in reinforced pavement which was observed in this State, we believe a review of reinforced concrete pavements constructed with recycled concrete is warranted. We would appreciate your assistance in identifying reinforced pavements, both jointed and continuously reinforced, containing recycled concrete as aggregate. These pavements should have carried traffic at least 2 years and be located on the Interstate or a route carrying moderate to high volume truck traffic.

We would also like to receive information on any investigations the State may have undertaken to evaluate the load transfer at cracks in reinforced pavements with recycled aggregates.

We are planning to complete this review during September and October so the results will be available to the States for use in developing their 1990 projects. Mr. John Hallin will be performing the review. Please contact him at 366-1323, if you have any questions. To expedite the review, please advise him, by phone, of the projects which are available for review.

Louis M. Papet

The Use of Recycled PCC as Aggregates in PCC Pavements

Stephen W. Forster
Construction, Maintenance and Environmental Design Division
Office of Engineering and Highway Operations Research and Development

February 1985

The Use of Recycled PCC as Aggregates in PCC Pavements

Table of Contents

The Use of Recycled PCC as Concrete Aggregate

1. Introduction

1.1 The Incentive to Recycle. Economic considerations are the primary reasons for recycling, although often there are also environmental benefits to be derived. In some areas of the country there are no available supplies of virgin aggregates and recycling is the only viable economical solution. In other areas available sources of new rock are inaccessible, either because the value of the land is too high, or because zoning-type constraints prevent the opening of pits or quarries to obtain the material. In some instances, such as highly developed urban areas, economic incentive comes from the inability to properly dispose of the wasted material, and hence, it is less expensive and more environmentally acceptable to re-use it. Therefore, when a PCC pavement will be removed prior to replacement with a new pavement, the project is a prime candidate for recycling, thereby serving as a source of aggregate in the new concrete and eliminating the need and expense of disposing of the material removed. Further, if the project is large enough to set up an aggregate plant on site, additional savings can be realized by the elimination of much of the materials' transportation costs.

1.2 Recycling History. Results of a 1971 survey conducted by the Texas State Highway Department and the Texas Transportation Institute (ref.1) indicate that at that time little consideration was being given by most States to recycling existing pavement material other than as unstabilized base courses. PCC removed from a roadway was normally disposed of in landfills, or at best as erosion control in drainage ditches. This attitude has changed, as the use of natural resources and energy has had increasing economic impact.

Proposals to use recycled PCC as concrete aggregate material generated a number of questions. First, what would the quality of the new concrete containing the recycled material be, compared to the old concrete and also to new concrete made with natural aggregate?

Would the crushed concrete make good aggregate? How could the reinforcing be easily removed? Would recycling for this purpose (aggregate) be an economically viable alternative? These questions, and many others, concerning the recycling of PCC have now been substantially answered by subsequent work. This report will deal particularly with the use and properties of the recycled material as aggregates in PCC.

1.3 FHWA Involvement. The Federal Highway Administration (FHWA) initiated Demonstration Project No. 47 (DP47), Recycling Portland Cement Concrete Pavements, in May, 1978, and it is still active. The initial report under this project was the reprinting of an Iowa Department of Transportation report on an early recycling project, which is summarized later in this report. A number of other States have since conducted recycling projects under DP47 and States continue to show interest in participating in DP47.

FHWA also established project 22 on pavement recycling under its National Experimental and Evaluation Program (NEEP) in June, 1977. Both asphalt and portland cement concrete recycling were included in NEEP 22. Throughout its duration 42 States participated in the project which has now been integrated into either DP 47, mentioned above, or DP39, Asphalt Pavement Recycling.

FHWA sponsored a national seminar on PCC recycling and rehabilitation in September, 1981, which was conducted by the Transportation Research Board (TRB). Many of the details given is this report are from the proceedings volume (ref. 2) and the summary volume (ref. 3) for this meeting.

1.4 Definitions. Recycling as applied to PCC pavements many be grouped into 3 categories. First is surface recycling, which includes milling or grinding the surface (approximately the top inch (25 mm)) of the pavement to remove surface deterioration, restore rideability, and improve surface friction. The material removed is usually quite fine and in relatively small quantities, so it is normally not used as concrete aggregate. A second type of recycling is in-place recycling in which the old pavement is crushed and combined with the existing base or subbase material to form a base for support of a new pavement. The third type of recycling may be called plant recycling, in which the existing PCC pavement is broken up, removed from the roadway to a crushing operation, crushed and sized. The aggregate material thus produced is incorporated in a new PCC mixture for placement on the job. It is this use of the old concrete as aggregate in new PCC which will be the major topic of this report.

2. Properties of Recycled PCC Aggregate

2.1 Aggregate Tests. A number of laboratory studies have compared the properties of aggregate material made from crushed PCC with the properties of natural aggregates. Early work in this area was done by Alan Buck of the U.S. Army Engineers Waterways Experiment Station (WES), (ref. 4). Buck examined the properties of aggregate made from crushed concrete containing chert gravel (coarse) and natural sand (fine) and a second aggregate made from crushed concrete containing limestone (coarse) and natural sand (fine). These manufactured aggregates were tested and compared with natural aggregate and then incorporated into new concrete mixes for further comparisons. Results of absorption and specific gravity tests are shown in Table 1.

Visual inspection of the crushed concrete indicated a good particle shape. The fine aggregate as produced did not meet the normal gradation requirements, but was used as produced in the concrete mixes.

Results of studies conducted by WES, the Iowa Department of Transportation, Massachusetts Institute of Technology, The Minnesota Department of Transportation, the Michigan Department of Transportation and FHWA are summarized by Yrjanson (ref. 1). He found the following points of agreement:

> The aggregate particles produced by crushing concrete have good shape, high absorptions and low specific gravity compared to natural mineral aggregates.

The Michigan Department of Transportation (ref. 5) conducted a laboratory investigation of a series of crushed concrete materials for comparison with natural aggregate. Table 2 shows their test results. Michigan also tested a concrete material which had been recycled twice. Its specific gravity was still lower (2.11) and the absorption event higher (8.36 percent). These results are predictable since with each successive recycling the amount of natural aggregate decreases when expressed as a percent of the aggregate material and the amount of lighter, more absorptive cement paste increases. Interestingly, the soundness loss of the recycled material was less (0.9-2.0) then that of the natural aggregate (3.9).

2.2 Concrete Tests. Buck (ref. 4) made all his recycled concrete mixes with a water cement ratio of 0.49, a target air content of 6 ± 1/2 percent, and a slump of 2 1/2 ± 1/2 inches (63 ± 13 mm). He found that concrete made with recycled concrete as both coarse and fine aggregate had lower slumps and higher cement content than comparable mixes made with either all natural aggregate or recycled coarse aggregate and natural sand fine aggregate. He also noted that the concrete with recycled aggregate had compressive strengths 300-1300 psi (2068-8962 kpa) less than the control concrete throughout the period of testing (up to 180 days of age). Freeze-thaw test results differed depending on the original aggregate type. Recycled concrete containing freeze-thaw susceptible coarse aggregate performed better as aggregate in a new concrete than concrete containing that stone as coarse aggregate (although whether the improvement is sufficient to bring performance to an acceptable level would have to be judged on a case by case basis). Conversely, new concrete made with recycled concrete containing an originally freeze-thaw resistant aggregate performed somewhat worse than the control mix with the natural coarse aggregate, although both mixes performed acceptably well. Finally, Buck found that volume change in response to temperature changes or increased moisture was similar for the recycled concrete mixes and the controls.

Yrjanson (ref. 1) presented the following conclusions about the recycled concrete in his report:

1. The use of crushed concrete as coarse aggregate had no significant effect on mixture proportions or workability of the mixtures compared to the control mixes.

2. When crushed concrete was used as fine aggregate the mixture was less workable and needed more water and therefore more cement. Substitution of natural sand for up to 30% of the recycled fine aggregate improves workability to the approximate levels of a conventional mix.

3. The frost resistance of the concrete made from recycled aggregates was usually much higher than that made with natural aggregates.

4. The use of recycled aggregate did not have any significant effect on the volume response of concrete specimens to temperature and moisture changes.

5. The use of low strength recycled concrete as aggregate need not be detremental to the concrete's compressive strength.

6. The use of water reducing admixtures to lower the water content is effective in increasing strengths of concrete mixtures that contain recycled concrete as aggregate.

Fergus (ref. 5) reported that the Michigan Department of Transportation used various percentages of recycled PCC in the fine aggregate to determine its effect on the mixture. They also used various percentages of recycled bituminous concrete in the mixture to simulate contamination which would occur in practice. They made their mixtures with a water cement ratio of 0.43, a cement factor of 6 sacks/cuyd (7.8 sacks/m^3) and an entrained air percentage of 5.5 $\pm$ 1.5. The results of this research agrees with the findings of

Buck and Yrjanson. The slump of the recycled concrete mixtures was less than that of the control mixture due to differences in workability. Compressive and flexural strengths of the recycled concrete were slightly less than those of the control mixture made with a gravel aggregate, but still exceeded the Michigan Department of Transportation minimum specifications for pavement concrete. Those recycled materials with crushed bituminous concrete (patches, unremoved overlay spots, etc.) included as a small percentage of the aggregate were not detrimentally affected unless there was an inclusion of crushed bituminous fines. These fines are almost totally bitumen coated and therefore act essentially as voids in any strength test of the new concrete. The recycled concretes exhibited durability factors superior to that of the control mix.

3. Special Concerns for Recycled PCC

3.1 Recycled "D" Cracked Pavement. The possible use of crushed "D" cracked pavement as an aggregate material presents an additional concern. The question posed is whether the recycled material will continue to promote "D" cracking, or will the problem be alleviated (at least to the level of economically available natural aggregate material) by the crushing which takes place during the recycling process.

Prior to carrying out a recycling project using a "D" cracked PCC pavement, the Minnesota Department of Transportation conducted a laboratory study (ref. 6) to determine the behavior of recycled "D" cracked material when used as aggregate in new concrete. For the laboratory work a three foot (0.98 m) section, full width of the candidate pavement, was removed and crushed for testing in the laboratory. Four initial mixture designs contained: 1) 100 percent recycled aggregate; 2) recycled coarse aggregate and natural sand fine aggregate; 3) the same as 2, except fly ash was substituted for 10 percent of the cement; 4) the same as 2, except 20 percent fly ash was substituted for 15 percent of the cement. They also made a control with all natural aggregate and 20 percent fly ash substituted for 15 percent of the cement. Like other investigators, they found that the recycled material passing the No. 4(4.75mm) sieve was very angular and that this increased the water demand substantially (to provide acceptable workability). Mix 1 (recycled fine aggregate) required 333 lbs/yd^3 (197.5 kg/m^3) of water versus 250-260 lbs/yd^3 (48.3 to 15.42 kg/m^3) for the control. This higher water demand also increased the cement requirement. Compressive strengths were at or above conventional mixtures and they had no problem entraining the necessary air. Based on these results, three more trial mixes were made. The recycled aggregate all passed the 3/4 inch (19 mm) sieve and 0-5 percent passed the No. 4(4.75 mm) sieve. One of the mixes had no fly ash, one had 10 percent of the cement replaced by fly ash, and the third had 15 percent of the cement replaced by 20 percent fly ash. To evaluate the "D"-cracking susceptability, these mixes were subjected to freeze-thaw testing. In comparison with concrete containing the "D" cracking natural aggregate, the concrete with the recycled concrete aggregate was somewhat more resistant to freeze- thaw action, and the mixtures with 10-20 percent substituted fly ash had a greatly reduced "D" cracking potential. The fly ash also acted as a plasticizer, thereby lowering the amount of water necessary to make the mix workable.

Based on these laboratory results, the State went ahead and reconstructed U.S. 59 using the recycled concrete as coarse aggregate. The specific gravity of the recycled coarse aggregate was 2.41 and its absorption was 4.4 percent. Natural sand was used as the fine aggregate and 20 percent fly ash was substituted for 15 percent of the cement. Average core strength on the concrete was 4590 psi (31.6 MPa) after 60 days. The minus number 4 (4.75 mm) recycled material was used in the base course as a stabilizing material.

3.2 Salt Content of Recycled Pavement. As part of the Michigan study summarized above (ref. 5), they examined the NaCl content of the recycled PCC aggregate material, since large amounts of rock salt are used as a deicer on their highways. They found that the recycled material contained less than 2 lbs/yd^3 (1.2 Kg/m^3) compared to their critical NaCl level of 4 lbs/yd^3 (2.4 Kg/m^3) used for bridge decks. They concluded that no restrictions were necessary on the use of the material based on its salt content. Further, since the recycled material is used as only the aggregate portion, the overall level of chloride in the new concrete would be even less (the amount in the recycled PCC times the fraction of the new concrete which is recycled material).

In preparation for a recycling project, Connecticut (ref. 7) examined the total chloride content of recycled PCC material. They found 12 lbs/yd^3 (7.1 kg/m^3) at the 1.5 in (38 mm) level, 0.96 lbs/yd^3 (0.57 kg/m^3) at the 4in (102 mm) level, and 0.27 lb/yd^3 (0.16 kg/m^3) at the 6.5in (166 mm) level. The new mixture with the recycled concrete aggregate contained 1.93 lb/yd^3 (1.14kg/m^3) total chloride.

To summarize, it would be advisable to check the NaCl content of any recycled material which may have excessive salt, and based on the findings calculate what the salt content would be for the new mix. Based on the results, a decision could be made as to whether any additional steps (reinforcement coating, etc.) would be necessary to avoid problems.

3.3 Alkali - Aggregate Reactivity. Three things are necessary to cause damaging alkali - aggregate reactivity: 1) an aggregate with sufficient amounts of reactive constituents that are soluble in highly alkaline aqueous solutions; 2) enough water soluble alkali from some source (usually the cement) to drive the pH of the liquid in the concrete up to 14-15 and hold it there so that swelling alkali - silica gel is produced; 3) sufficient water to maintain the solutions and provide moisture for the swelling of the gel.

The consequences of using recycled PCC material which has suffered from alkali - aggregate reaction as an aggregate in a new concrete have not been throughly studied. In this special case of PCC recycling, several questions must be answered. How severe is the extent of the reaction and the resultant distress at the time of recycling? Has the reaction gone to completion - that is, has the reactive mineral matter been used up? If petrographic or other examination seems to indicate this, it may be safe to go ahead and use the material. On the other hand, merely the use of a low alkali-cement in the new concrete may not prevent further alkali-aggregate reaction with the recycled material because the reaction may continue within the recycled material between the old mortar and aggregates. Probably the only safe way to screen materials with this potential problem is to do long term mortar bar expansion tests (ASTM C-227) with the recycled material in cements with various alkali contents to determine what level of alkali is acceptable. If reaction is taking place between the recycled materials, it may be that no level of alkali - in the cement will be low enough to prevent the reaction. It has been speculated that the addition of limestone aggregate in the mix may reduce the probability of alkali-aggregate reactivity (ref. 8) but this is not yet proven. Reduction in recycled aggregate size may also be helpful in controlling the reaction problem. The question of recycling alkali-aggregate reactive materials needs additional investigation, and work is currently underway in a cooperative study in Colorado.

4. Field Projects with Recycled PCC

As a result of field projects incorporating recycled PCC as aggregate in the mixture, several facts were learned which should aid in the planning and conduct of future recycling projects. Iowa (ref. 9) had one of the early recycling projects on U.S. Rte 75 in 1976. They stockpiled the entire crushed recycled PCC from the secondary crusher (1 1/2 inch (38 mm) minus) in a single stockpile and found that segregation problems resulted as well as inconsistent feed through the automatic bin gates of the batching plant. They therefore went to splitting the material on the 3/8 in. (9.5 mm) sieve on subsequent projects, which alleviated the problem. Using recycled material for both coarse and fine aggregate produced a harsh mix which was nearly unworkable, so 15 percent concrete sand was added which made the mixture much easier to work. It was found that less air entraining agent was needed to reach the desired air content than would have been true with a conventional mix. The amount of contaminants in the recycled material must be controlled because they often have an effect on the air content of the new concrete. They found that approximately 75-80 percent of the old pavement is recovered as crusher product. Using the experience gained in the initial project, Iowa conducted two additional projects in 1977. As was found in the first project, the crusher product was low in fine material (22-24 percent passing the number 4 (4.75mm) sieve). A three aggregate blend (coarse and fine recycled, plus concrete sand) controlled segregation of the recycled material and made for a workable mixture. Washing the recycled material was found to be unnecessary if proper removal and processing practices were followed.

Minnesota (ref. 6) conducted a recycling project on U.S. 59 in the southeastern part of the State in 1980. This was a "D" cracked pavement and the results pertaining to that particular problem are discussed in section 3.1. However, several conclusions reached as a result of this project are applicable to recycling projects in general. As in Iowa, Minnesota found that the crushed material passing the number 4 (4.75mm) sieve is very angular and results in increased water demand and cement content when used in the mix. To avoid this situation, Minnesota removed the minus number 4 (4.75mm) material from the crushed concrete and used it as a stabilizer in the base material. They found that even in this use it needed constant watering to achieve target densities. They calculated that they would have enough recycled material for coarse aggregate in the mix if they had an aggregate blend of 60 percent coarse aggregate and 40 percent natural sand. The actual yield proved to be very close to this estimate.

5. Specifications

Several States (Iowa, for example, ref. 10) have developed specifications for removal, crushing, storing, and incorporating recycled materials in new PCC. These specifications cover all phases of the construction, and the reader is referred to them for this information. The discussion of specifications here will be limited to items directly effecting the recycled aggregate material.

5.1 Removal and Contamination. Some limit should be set on the amount of allowable contamination in the material recycled, either from any asphalt overlay, patch, joint sealant or subbase material. It has been found that some amount of adhering asphaltic concrete is allowable and not detrimental to the mixture.

5.2 Crushing and Stockpiling. Maximum size of material should be specified and may vary depending on the use of the concrete, however, typically top size is specified as 100% less than 1 1/2 inches (38 mm). The maximum size specified may have to be reduced (100% less than 3/4in (19 mm)) if the material being recycled is a "D" cracked pavement. Standard good stockpiling techniques should be followed, and the plus 3/8 in. (9.5 mm) and minus 3/8 in. (9.5 mm) should be stored separately to avoid segregation. Washing is not normally necessary, however this would be dictated by individual job conditions. Provision should be made to limit the amount of minus 200 (.075 mm) material to some maximum percentage.

5.3 Mix Proportions. Crushed recycled material may be used for both the coarse and fine aggregate, however use of 15-30% natural sand in the fines may be specified to improve workability and finishability of the mix. Mix proportions should be determined based on trial mixes made in the laboratory. An effort should be made to proportion use of the coarse and fine recycled material in the same ratio as it is produced by the crusher.

Cement factor will be determined according to the strength desired, as with a conventional mix. Water shall be used in a ratio which will provide acceptable workability and finishability without being so high that excessive cement is required to maintain strength. To this end, addition of natural fine aggregate (as noted above) may be specified to improve these characteristics while holding the water content at a reasonable level. Water reducing admixtures may also be considered for the specification to maintain the water cement ratio at an acceptable level. Air entrainment will also increase workability.

5.4 Durability. The durability of the concrete produced should be required to be checked in the laboratory according to ASTM C-666 or some equivalent method. If alkali-aggregate reactive material is being recycled, the expansive characteristics of the new concrete may also be checked by ASTM C-227 or equivalent to determine if it will perform adequately.

5.5 Air Entrainment. Air content may be specified and obtained using the addition of an approved air entraining agent as with a conventional mix. If the recycled material is air entrained, the specified air for the new concrete may have to be set higher than normal since the measured air will include the newly entrained air plus the air content of the recycled material. When the air content of the recycled material is subtracted from the measurement obtained on the new plastic concrete, the residual will then provide a measure of the amount of air in the new mortar. The presence of organic contaminants may cause high air contents and therefore de-air entraining agents may be needed.

6. Summary & Conclusions

This report is an assemblage of the current knowledge on the use of recycled PCC as aggregate in new concrete construction. The following points highlight its contents.

1. Recycling PCC is a viable alternative to using natural aggregate in concrete construction in many instances, particularly those in which the natural aggregate would have to be transported some distance and there is a problem disposing of the old concrete removed.

2. FHWA continues to encourage States to try recycling projects through its Demonstration Project 47, Recycling Portland Cement Concrete Pavements.

3. The recycled material may be tested using many of the same tests used for natural aggregate material. Recycled PCC tends to have a higher absorption and lower specific gravity than natural aggregates. The crushed material has a good particle shape.

4. The use of recycled concrete as the aggregate in a new mixture has several effects. If the recycled material is used for the fine aggregate, its harshness decreases the workability of the mixture. This may be compensated for by substituting some natural aggregate fines for the recycled material, increasing the water (and therefore the cement) content, adding a water reducing admixture, or some combination of the three.

5. The freeze-thaw resistance of the new concrete is generally better than that of a comparable concrete made with natural aggregates.

6. The durability of recycled "D" cracked concrete is greatly improved over that of the original concrete, and may be improved still more if necessary by specifying a reduced maximum size for the recycled material. Fly ash appears to decrease the tendency for "D" cracking in the recycled concrete mix.

7. Compressive and flexural strengths of recycled concretes tend to be slightly less than those of comparable mixes with natural aggregates, however strengths above the minimum normally required are still easily obtained with proper mix design.

8. In the few studies examining the possible problems of recycling salt contaminated concrete, the NaCl levels were not high enough to promote distress. More work needs to be done to determine the level at which salt content in the recycled material becomes determinantal to the new mix, particularly since the recycled material is usually

used in pavements rather than bridges which present different corrosion conditions.

9. The use of recycled PCC suffering from alkali-aggregate reactivity in a new concrete has not been adequately addressed. The surest approach at this time is to subject any suspected material to the mortar bar expansion test (ASTM C-227) to evaluate of its behavior. Further research is needed in this area.

10. Specifications for recycled PCC aggregate material should have the performance requirements which are generally applied to natural aggregates. Attention must be paid to the recycled material's effect on the workability of the new mix and the various ways to improve it. Depending on the condition and distress of the recycled pavement, statements may have to be included in the specification to require testing for durability, expansion, permeability and strength.

11. Recommendations and Extensions

The recycling of PCC as aggregate in a new concrete mix is a viable alternative to the use of natural aggregates in many instances. Experience has shown that with proper planning, testing, and construction techniques, quality concrete can be made using recycled PCC as aggregate.

There are a number of recycling situations where additional study still needs to be done to determine long term effects. The recycling of concrete which has suffered from alkali-aggregate reaction still involves some unknownsas to the long term behavior of the recycled aggregate material. A cooperative study is currently being done in Colorado to determine the effects of fly ash on new mixtures using this type of recycled aggregate.

The presense of chlorides in the recycled concrete is another area of concern. We know pretty well what levels of chloride content are critical in causing corrosion of bridge deck reinforcement. However, in pavement concrete there is usually much less steel which has a greater cover of concrete. The effect of having the chloride concentrated in the aggregate initially is also unknown.

A synthesis study on recycling of PCC pavement is included in the National Cooperative Highway Research Program for fiscal 1985. This will summarize current knowledge and practices in this subject area. Recycling of PCC will also be one of the subjects addressed by the Strategic Highway Research Program, now in the planning stage.

References

1. Yrjanson, W.A., Recycling Portland Cement Concrete, in Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, pp 128-133.

2. Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, 196 p.

3. Pavement Recycling: Summary of two Conferences, FHWA-TS-82-208, April 1982, 66 p.

4. Buck, A.D., Recycled Concrete, in Utilization of Waste Materials and Upgrading of Low-Quality Aggregates, HRR 430, 1973 pp 1-8.

5. Fergus, J.S., Laboratory Investigation and Mix Proportions for Utilizing Recycled Portland Cement Concrete as Aggregate, in Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, pp 144-160.

6. Halverson, A. D., Recycling Portland Cement Concrete Pavements, FHWA-DP-47-3, May 1981, 66 p.

7. Lane, K. R., Construction of a Recycled Portland Cement Pavement, Connecticut Department of Transportation Report No. 646-1-80-12, September, 1980, 47p.

8. Heck, W.J., Study of Alkali-Silica Reactivity Tests to Improve Correlation and Predictability for Aggregates, in Cement, Concrete, and Aggregates, Vol 5, no 1, Summer 1983, pp 47-53.

9. Bergren, J.V. and R.A. Britson, Portland Cement Concrete Utilizing Recycled Pavement, FHWA-DP-47-1, January 1977, 35 p.

10. Huisman, C.L. and R.A. Britson, Recycled Portland Cement Concrete "Specifications and Quality Control", in Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, pp 140-143.

	Recycled Material Chert Concrete	Recycled Material Limestone Concrete	Natural Material Chert Gravel	Natural Material Crushed Limestone
Coarse Friction				
Absorption	4.0 - 4.3	3.9	2.6	0.8
SSD Sp. Gravel	2.43-2.44	2.52	2.52	2.67
			Sand	
Fine Fraction				
Absorption	7.6 - 9.0	-	0.4	
SSD Sp. Gravel	2.36	-	2.63	

Table 1. Properties of Crushed Concrete and Natural Aggregates (After Buck, 1973).

	Recycled Material Once Recycled	Recycled Material Twice Recycled	Natural Material Gravel
Coarse Fraction			
Absorption	3.43 - 5.0	8.36	1.02
Bulk Sp. Gravel	2.31 - 2.40	2.11	2.67
Fine Fraction			
Absorption	7.17 - 8.31	-	1.38
Bulk Sp. Gravel	2.15 - 2.23	-	2.60

Table 2. Properties of Crushed Concrete and Natural Aggregates (After Fergus 1981).

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: Technical Paper - An Overview of Surface Rehabilitation Techniques for Asphalt Pavements

Date: APR 6 1992

From: Chief, Pavement Division

Reply to Attn of: HNG-42

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

During the past year, the Pavement Division, in conjunction with the Office of Technology Applications, has been involved in a comprehensive effort to develop an information base on existing and emerging surface rehabilitation techniques for asphalt pavements. Examples of techniques we are evaluating include: (1) cold mixtures such as slurry seals and micro-surfacing; (2) single and multiple chip seals; and (3) open and dense graded thin hot-mix overlays. The use of modified binders and fibers in these applications will also be examined. This project will provide information on the usage, design, construction, cost, and anticipated performance of these techniques when applied as a functional improvement to a structurally sound higher volume roadway pavement. Further, this project will complement and expand on the information gained from the Strategic Highway Research Program's specific pavement studies (SPS-3) experiment.

Attached are copies of the technical paper entitled, "An Overview of Surface Rehabilitation Techniques for Asphalt Pavements," (FHWA-PD-92-008). You may wish to provide copies of this paper to your division offices. This paper summarizes known preventative maintenance and surface rehabilitation techniques based on our literature search and some limited field work. During the coming months, we will be visiting several existing and new projects to gather additional related information on various applications. Your staff assistance in this regard will be appreciated.

If you have any questions on our effort or like to arrange for a presentation on this subject, please call Messrs. Hassan Raza at FTS 366-1338 or James Sorenson at FTS 366-1333.

Louis M. Papet

Attachments

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: ACTION: Distribution of Publication

Date: July 12, 1994

From: Director, Office of Engineering
Director, Office of Technology Applications

Reply to Attn of: HNG-42

To: Regional Administrators
Federal Lands Highway Program Administrator

The attached publication, State of the Practice Design, Construction, and Performance of Micro-surfacing (FHWA-SA-94-051) provides a comprehensive discussion on an emerging surface rehabilitation technology. Sufficient copies of this publication are attached for your use and further distribution to the division offices and States within your region. Copies have also been distributed to each of the LTAP Technology Transfer Centers. Additional copies are available in limited supply from the Research and Technology Report Center, HRD-11, 6300 Georgetown Pike, McLean, Virginia 22101-2296 (telephone 703-285-2144).

Micro-surfacing consists of polymer-modified asphalt emulsion, crushed aggregate, mineral filler, water, and field-controlled additives as needed. Micro-surfacing is primarily used to seal existing surfaces, improve surface friction, and fill wheel ruts on both moderate and high volume roads. When properly designed and constructed, micro-surfacing has shown promising results with several years of service life. This surface rehabilitation technique has also been used effectively on portland cement concrete pavements to improve surface friction or address mechanical wear in the wheel paths.

This state-of-the-practice paper is a result of a joint effort by the offices of Engineering and Technology Applications, and the industry to develop information on existing and emerging surface rehabilitation techniques for asphalt pavements. The first product of this effort, An Overview of Surface Rehabilitation Techniques for Asphalt Pavements (FHWA-PD-92-008) was developed and distributed in April 1992. Presentation slides for both of the above papers will be available later this fall.

In a related effort, an Office of Engineering memorandum dated June 24 announced the availability of warranty guide specifications for micro-surfacing projects on the National Highway System under Experimental Project 14. If you have any questions or would like to request technical support in the surface rehabilitation area, please call Hassan Raza at 202-366-1338.

William A. Weseman

Ray G. Griffith
FOR: Director, Office of Technology Applications

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject INFORMATION: SP204 - Retrofit Load Transfer

Date FEB 10 1994

From Chief, Pavement Division
Chief, Engineering Applications Division

Reply to Attn of HNG-42
HTA-21

To Regional Federal Highway Administrators
Division Federal Highway Administrators
Federal Lands Highway Program Administrator

Attached are the following documents for your use and information:

1. Current status report - **SPECIAL PROJECT 204** - Retrofit Load Transfer and December 27, 1993 report Retrofit Load Transfer in Jointed Concrete Pavements
2. TRB Preprint 940247, Linda M. Pierce, PCCP Rehabilitation in Washington State (A Case Study)
3. Inspection report by Lynn Porter and Cathy Nicolas on Washington State Load Transfer Retrofit Project
4. Report by Roger Larson of load transfer restrofit field visits in Puerto and Indiana

Until recently, load transfer retrofit had been used only experimentally in the continental United States. In the last ten years, an estimated 300 lane Km of faulted or cracked undoweled jointed plain concrete pavement (JPCP) has been successfully rehabilitated in Puerto Rico. Based on the generally good performance of previously constructed load transfer retrofit experimental sections in the U.S. and the outstanding performance in Puerto Rico, SP-204 was initiated to encourage the development of equipment to construct multiple slots in each wheelpath to increase the production rate for this technique and to reduce the construction cost and road user delays.

Attachment 1 describes the current status and background of this effort. Attachment 2 describes the preliminary engineering and experimental test section construction that led to the 53 km project now underway in Washington State. Attachment 3 describes the major Washington State project currently underway involving 53 km (about 24 km now complete) of retrofit load transfer on eastbound I-90. Attachment 4 describes field visits to Puerto Rico to observe the long term performance of retrofit load transfer projects and to Indiana to observe a demonstration of the feasibility of using carbide milling technology to construct multiple slots in jointed reinforced concrete pavement (JRCP).

Based upon the recent construction of 24 lane km of retrofit dowels (JPCP) in the project currently underway in Washington State and the successful demonstration of milling three slots per wheelpath in one pass on working cracks in a JRCP ramp in Indiana, equipment is now available to economically construct retrofit load transfer at joints or cracks in existing jointed concrete pavements. The bid price to construct retrofit load transfer devices in Washington State was $34.50 per dowel installed (62,000 38 mm dowels in 64 mm wide slots). The average bid price in Puerto Rico is $20 per dowel installed (25 mm dowel in 40 mm wide slots) where this has been done routinely for ten years (slots sawed individually).

This technique should be used with other concrete pavement restoration techniques to rehabilitate existing jointed concrete pavement <u>before</u> serious deterioration is present. Perhaps the most cost-effective initial application of this technique would be to restore load transfer at working cracks developing in under-reinforced JRCP in other wise good condition. If performed early, it would also provide a cost-effective extension of the service life at the joints on undoweled JPCP and at transverse cracks without serious deterioration in either doweled or undoweled JPCP. If serious deterioration is present, full depth patching and/or selective slab replacements should be performed instead.

When properly applied, this technique will result in a cost-effective extension of the service life of existing jointed concrete pavements in good to fair condition. This technique would also be a very effective routine and preventive maintenance technique to reduce the cost and user delays during repairs of working cracks shortly after they develop and before full depth patches or slab replacements become necessary.

If you have comments or questions, please contact Mr. Roger Larson, the project manager of SP 204, at (202) 366-1326. A Technical Working Group will be formed shortly to update guidance reflecting the new equipment developments and other critical technical issues to help ensure success of this promising technique.

Theodore R. Ferragut

Louis M. Papet

4 Attachments

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: ACTION: ISTEA Section 6005
Thin Bonded Overlay and Surface Lamination
Pavements and Bridges
Reply due: October 31, 1994

Date: July 1, 1994

From: Director, Office of Engineering

Reply to Attn of: HNG-32
HNG-42

To: Regional Federal Highway Administrators

We are requesting applications for additional projects for the Thin Bonded Overlay and Surface Lamination (TBO) Program, which is part of the Applied Research and Technology (ART) Program established by Section 6005 of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. A summary of the TBO program and the application procedures are described in Attachment A. The application form is included as Attachment B. A summary of information on technologies is included in Attachment C. A listing of bridge deck and pavement overlay projects and TBO technologies previously approved is included in Attachment D and the evaluation plans developed for these projects are included in Attachment E.

Additional projects are being sought for available fiscal year (FY) 1994 and 1995 funding. Projects proposed for construction in FY 1996 and 1997 are also encouraged. There may be no future solicitations for ISTEA TBO projects if enough candidate projects are available for selection from responses to this request. Please contact the States in your region for candidate projects for the TBO program. Candidate projects proposed by the State highway agencies must be submitted on the application form (Attachment B) and sent with any supporting information to the appropriate Federal Highway Administration Division Office by October 14, 1994, for forwarding to this office by October 31. The Section 6005 funding provided (100 percent for reporting and evaluation and 80 percent for construction and an equal amount of obligation authority for projects approved as a part of this solicitation) is in addition to the individual State's regular Federal-aid. Please also note that priority for funding will be given to the technologies listed in the *New Projects Sought* section of Attachment A.

Your cooperation and attention are greatly appreciated. If you have any questions or comments, please contact Mr. Vasant Mistry, HNG-32, (202) 366-4599 or Mr. Roger Larson, HNG-42, (202) 366-1326. General questions on the ART Program should be addressed to Mr. Richard A. McComb, HTA-2, (202) 366-2792.

for William A. Weseman

5 Attachments