REPORT NUMBER: WI/SPR-05-99

PB99-167207

INVESTIGATION AND APPLICATION OF FRACTURED SLAB TECHNIQUES FOR PCC PAVEMENTS

FINAL REPORT

WISCONSIN DEPARTMENT OF TRANSPORTATION
TECHNOLOGY ADVANCEMENT

APRIL 1999

INVESTIGATION AND APPLICATION OF FRACTURED SLAB TECHNIQUES FOR PCC PAVEMENTS

FINAL REPORT WI/SPR-05-99
Federal Experimental Project #WI 93-06

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APRIL, 1999

The Pavement Research Unit of the Division of Transportation Infrastructure Development, Bureau of Highway Construction, under the direction of the Council on Research, conducts and manages the highway engineering research program of the Wisconsin Department of Transportation. The Federal Highway Administration provides financial and technical assistance for these research activities, including review and approval of publications. This publication does not endorse or approve any commercial product, even though trade names may be cited, does not necessarily reflect official views or policies of the agency, and does not constitute a standard, specification or regulation.

<table>
<tr><td>1. Report Number
WI/SPR-05-99</td><td>2. Govt. Accession No.</td><td>3 Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Investigation and Application of Fractured Slab Techniques for PCC Pavements</td><td>5. Report Date
April 1999</td></tr>
<tr><td>6. Performing Organization Code</td></tr>
<tr><td colspan="2">7. Authors
Owusu-Ababio, Sam and Nelson, Thomas</td><td>8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
University of Wisconsin-Platteville
Civil and Environmental Engineering Dept
1 University Plaza
Platteville, WI 53818</td><td>10 Work Unit No.</td></tr>
<tr><td>11. Contract or Grant No.
WisDOT Project 0093-06</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
Wisconsin Department of Transportation
Div. of Transportation Infrastructure Development
Pavement Research Unit, 3502 Kinsman Blvd.
Madison, WI 53704</td><td>13. Type of Report and Period Covered
Final Report
May 1996 -April 1999</td></tr>
<tr><td>14. Sponsoring Agency Code</td></tr>
<tr><td colspan="3">15. Supplementary Notes</td></tr>
<tr><td colspan="3">16. Abstract
Slab fracture techniques, including break & seat, crack & seat, and rubblization have in recent years gained widespread recognition among pavement engineers as means for eliminating or substantially reducing the potential for reflective cracking in hot mix asphalt (HMA) overlays over portland cement concrete (PCC) pavements. Guidelines for the use of these techniques in Wisconsin, however, have not been clearly established.

This reports examines the PCC rehabilitation techniques of rubblization and crack & seat used in Wisconsin and their performance. The report examines the literature and evaluates the critical issues associated with the use of PCC fracture techniques by various agencies. Several elements pertinent to fracture techniques are also identified and incorporated in a database for in-service fractured overlaid PCC pavements in Wisconsin. In addition, the in-service performance of cracked and seated and rubblized pavements in Wisconsin is evaluated. The construction procedures and equipment performance for some fractured pavement projects in Wisconsin, in addition to their short-term field performance are presented. Finally, the report provides guidelines for the use of crack and seat and rubblization rehabilitation techniques for PCC pavements in Wisconsin.</td></tr>
<tr><td colspan="2">17. Key Words
PCC Pavement, Reflection Cracking, Crack & Seat, Rubblization, Performance, Fracture Methods.</td><td>18. Distribution Statement</td></tr>
<tr><td>19. Security Classif.(of this report)</td><td>20. Security Classif. (of this page)</td><td>21. No. of Pages | 22. Price</td></tr>
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1. Introduction and Background

Reflective cracking in hot mix asphalt (HMA) overlays on portland cement concrete (PCC) pavements has long been recognized to decrease pavement performance. Various research efforts have consequently been made by several agencies in an attempt to minimize the occurrence and damaging effects of reflective cracking. Such efforts have included the use of increased overlay thickness, fabrics, reinforced overlays, interlayers and separation layers of base course, saw and seal, and slab fracture techniques. In 1993, the Wisconsin Department of Transportation (WisDOT) initiated research efforts to examine state-of-the-art fractured slab techniques for controlling reflective cracking in HMA over PCC pavements. The first phase of this research effort, titled " Fractured Slab Techniques for PCC Pavements" yielded a synthesis report which identified two fracture methods to have potential application in Wisconsin namely, **crack and seat** and **rubblization**. Phase II of this research was, therefore , initiated in May, 1996 to provide a better understanding of the performance of these two fractured slab techniques in Wisconsin.

1.1 Problem Statement and Objectives

Reflective cracking of HMA overlays of PCC pavements is aesthetically objectionable, and results in shortened pavement life and increased maintenance costs. Although various fracturing techniques have been utilized to address these problems, WisDOT does not have adequate data to evaluate their performance and define guidelines for their use.

The specific objectives of this research are to:

- Develop procedures and guidelines for the use of crack and seat and rubblization as rehabilitation alternatives for PCC pavements;
- Develop a database for fractured slab projects which WisDOT has performed since the early 1980s;
- Analyze the field performance of fractured slab projects in the State of Wisconsin;
- Survey the pre-construction characteristics, monitor the construction procedures, and evaluate the performance for a rubblization project on State Highway 16 (STH 16); and,
- Broaden the WisDOT knowledge base regarding methods for controlling reflection cracking.

2.0 Synthesis of Slab Fracture Practice for Reflective Crack Control

Reflective cracking in HMA overlays has for years been a perplexing problem for pavement engineers. Attempts to prevent its occurrence in highway pavements have been reported in the literature as far back as 1932[1]. Since that time, advances in the state-of-the-art for controlling reflective cracking have ranged from the experiences gained from trial-and-error experiments on in-service pavements to recent theoretical studies which attempt to develop methods to successfully prevent reflective cracking. It has been widely discussed by many researchers[2-6] that the primary mechanisms leading to reflective cracking development in HMA overlays are the horizontal and differential vertical movements at the joints and cracks in the existing pavement, with the horizontal movements being considered more critical. The damaging horizontal movements are caused by seasonal temperature changes and daily temperature cycles while the differential vertical movements occurring at underlying joints with poor load transfer and at working cracks are due to traffic loading[6,7]. Recent state-of-the-art techniques for restricting such movements in the underlying pavement and hence, controlling the potential for reflective cracking, use one of three fracture methods, namely: crack and seat, break and seat, and rubblization.

A literature review report completed by WisDOT for Phase 1 of this research study recommended the **crack and seat** and **rubblization** techniques as the fracture methods having potential application in Wisconsin. The crack and seat method, according to WisDOT definition, is applied to jointed plain concrete pavements (JPCP) only. This method seeks to minimize or retard reflective cracking by creating concrete pieces that are small enough to reduce horizontal slab movement resulting from thermal expansion and contraction, and yet are interlocked sufficiently to resist the vertical movement induced by traffic. The seating process that follows the cracking assures that the pieces are seated firmly against the subgrade. **Rubblization,** on the other hand, seeks to eliminate reflective cracking in the HMA overlay by the pulverization of the existing slab, creating a layer of variable sized aggregate in place of the former slab. It can be applied to all types of PCC pavements.

Current practice by states using fracture methods to rehabilitate concrete pavements indicate that several important features are associated with fracturing methods for controlling reflective cracking in HMA overlays on PCC pavements. Included are the size of the fractured pieces, type of existing pavement that is fractured, type and operation of equipment, the thickness of the overlay that is placed, the roadbed condition, and the performance of the fractured pavement.

2.1 Type of PCC Pavement and Condition

Pavement type and condition are two fundamental factors which influence the type of fracturing technique to use in PCC rehabilitation. Current practice indicates that most agencies use the techniques on experimental basis or use engineering judgment to determine when and where to use a given technique. However, the trend is to use the crack and seat technique for structurally sound **JPCPs** which exhibit characteristics such as excessive movements, faulted joints, pumping, and excessive cracking. Rubblization, on the other hand, according to a Pavement Consultancy Service study[8], can be applied to **all** PCC pavements with severe deterioration, and with very little potential to retain slab integrity and structure. Rubblization techniques have been successfully used on PCC pavements exhibiting prime distresses such as joint deterioration, displaced patches, transverse and longitudinal cracking, punchouts, and delaminations. For the crack and seat method, however, it is documented in the literature that construction problems are likely to occur if an adequate base does not exist over a fine grained subgrade to support the cracking operation.

2.2 Fracture Pattern and Segment Size

The size requirement for the fractured PCC slab is dependent on the technique being used. The general trend, however, has been to develop a smaller fracture pattern which would reduce reflective cracking and at the same time have adequate structural capacity. **Crack and seat** has been performed with spacing between 300 and 1500 mm (12 and 60 in.), with reports that 600 to 1200 mm (24 to 48 in.) spacings tend to achieve the best results[9]. Crack spacing of 600 mm (24 in.) has been commonly used on Wisconsin projects. A typical crack pattern for a pavement that has been cracked and seated is shown in Figure 2-1. The guillotine drop hammer has been identified as the most effective piece of equipment for crack and seat operations.

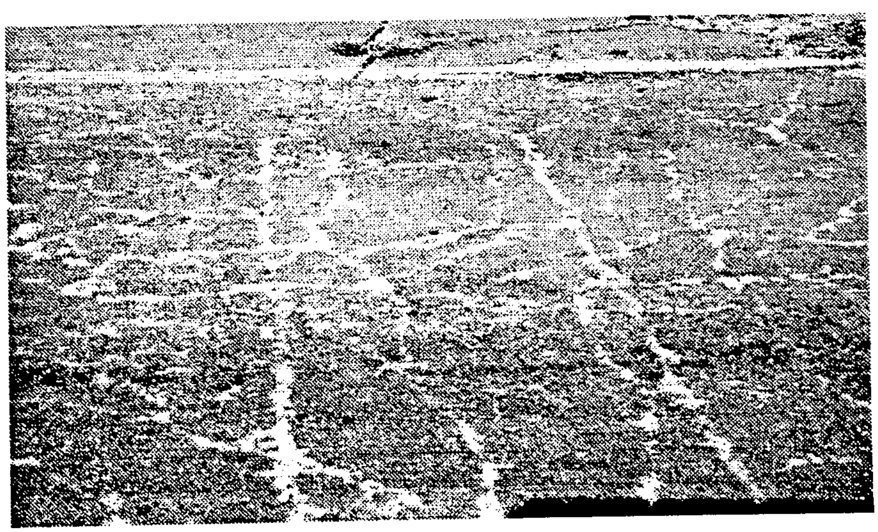

Figure 2-1. Crack Pattern for a Cracked and Seated Pavement.

Rubblization, if performed by a PB4 resonant pavement breaker or Multiple-Head Breaker, generally produces pieces that range in size from 25 to 75 mm (1 to 3 in.) near the surface, with a maximum particle size of 300 to 375 mm (12 to 15 in.), usually found in the lower portions of the pavement. Figure 2-2 shows the typical distribution of particle sizes following a rubblized PCC pavement performed by a PB4 pavement resonant breaker. Although initially tried for rubblization, a drop hammer or whip hammer device has been found to generally produce larger and more inconsistent fractured slab pieces.[10] This has resulted in most agencies not recommending the drop hammer or the whip hammer for rubblization. It is most important to ensure that the final product does not include many oversize pieces, which will not function as aggregate. Oversize pieces are sometimes subject to further breaking using jack hammers, for example, as commonly done in the State of Michigan[11], or oversize pieces are removed and replaced with high quality aggregate. In some instances, where there is the existence of weak spots in the subgrade, the recommended pre-overlay repair is to replace these spots with high quality aggregate material.

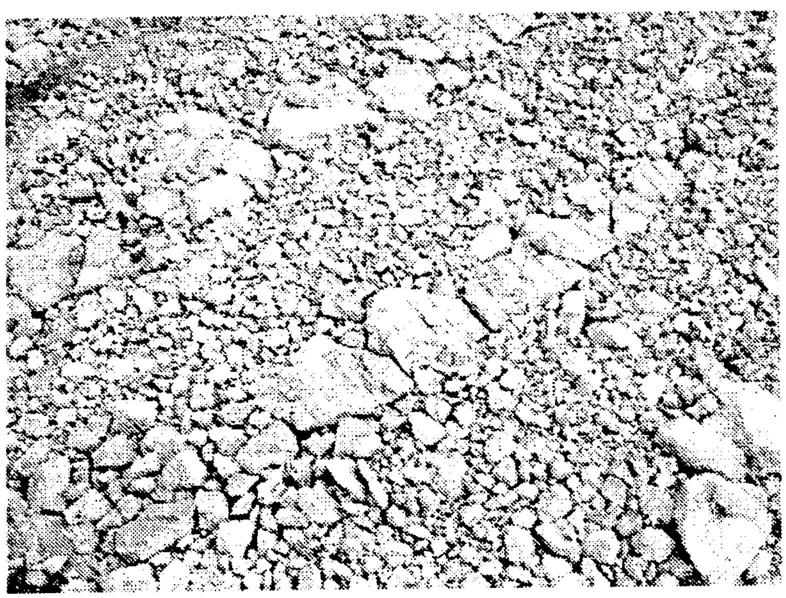

Figure 2-2. Rubblized PCC Pavement Particle Size Distribution Produced by a PB4 Resonant Breaker

2.3 Fracturing Equipment

A large variety of fracturing equipment is available to contractors for the fracturing process, as indicated in Table 1, on page 9. For **rubblization**, the most effective fracturing equipment includes the self-propelled resonant pavement breaker (PB4) and the Multiple Head Breaker (MHB), shown respectively in Figures 2-3 and 2-4. The PB4 operates at 5 - 8 km/h (3-5 mph) and delivers 900 kg (2000 pounds) of force, at 44 cycles per second. It produces a low amplitude oscillation over a 300-mm (12-in.) square or a 175-mm (7-in.) square breaker shoe. Typically, 15 passes are required to cover a 3.6-m (12-ft) traffic lane using the 300-mm (12-in.) shoe, and 20 to 24 passes with the 175-mm (7-in.) shoe. This results in a production rate of 210 to 530 lane-m/hr (690 to 1765 lane-ft/hr). The PB4, due to its heavy weight, has the tendency to shove, distort, or punch through pavements that are built on weak subgrades. The MHB was developed in 1995 by Badger State Highway, Inc., Antigo, WI. It uses two rows of 450 kg (1000 lb) hammers mounted laterally in pairs. The heads are lifted by hydraulic cylinders to adjustable heights and dropped at a rate of up to 35 impacts per minute. The breaking width can be adjusted in 0.6 m (2 ft.) increments, from 0.6 m (2 ft.) to a maximum of 4.0 m (13 ft.). Working speeds range from 3 m per minute to 16 m per minute, resulting in typical production rates from 180 to 960 lane-m/hr (600 to 3170 lane-ft/hr).

The most common devices used for **cracking** include the guillotine hammer, the whip hammer, and the pile-driving hammer with modified shoes. The guillotine (shown in Figure 2-5) is the preferred cracking equipment due to its effectiveness as a breaking device and the high productivity it offers. Measured production rate is in the range of 1.2 to 6.4 lane-km/day (0.75 to 4.0 lane-miles/day). The whip hammer, shown in Figure 2-6, consists of a flexible leaf spring arm with a flat cracking head on one end; the other end is attached to a truck where the arm is hydraulically manipulated to the desired striking position. Its use as a cracking device is being discontinued because it tends to produce inconsistent crack patterns and its operation is time consuming. The pile-driving hammer (shown in Figure 2-7), although an effective cracking device, has a low productivity of 0.43 lane-km/day (0.27 lane-mi/day) compared to the guillotine hammer.

Figure 2-3. PB4 Pavement Resonant Breaker Equipment

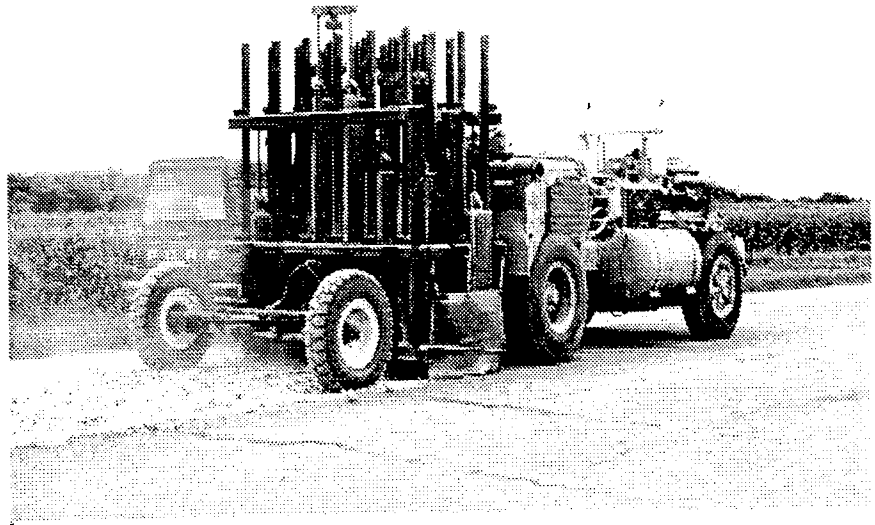

Figure 2-4. Antigo Multiple Head Breaker Equipment

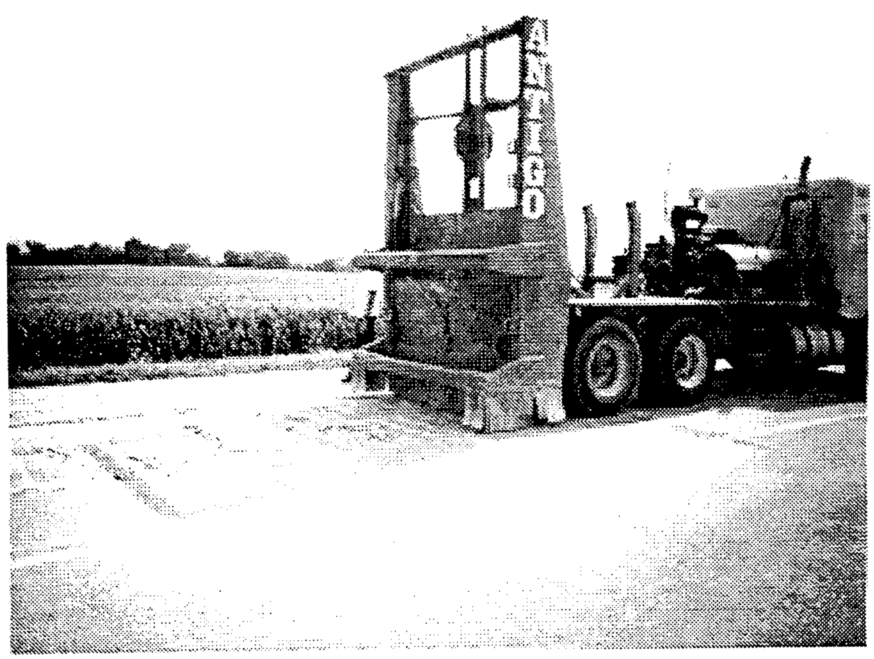

Figure 2-5. Guillotine Drop Hammer Equipment

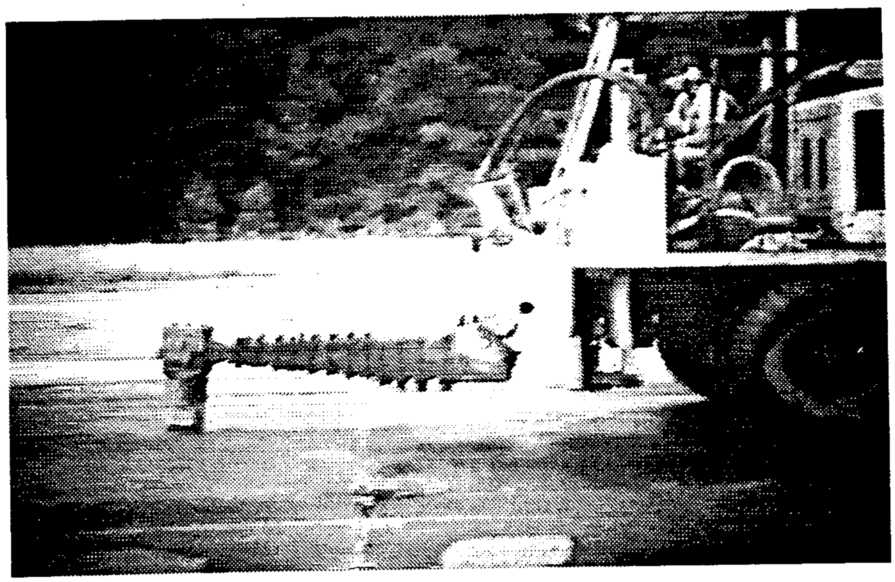

Figure 2-6. Whip Hammer Equipment

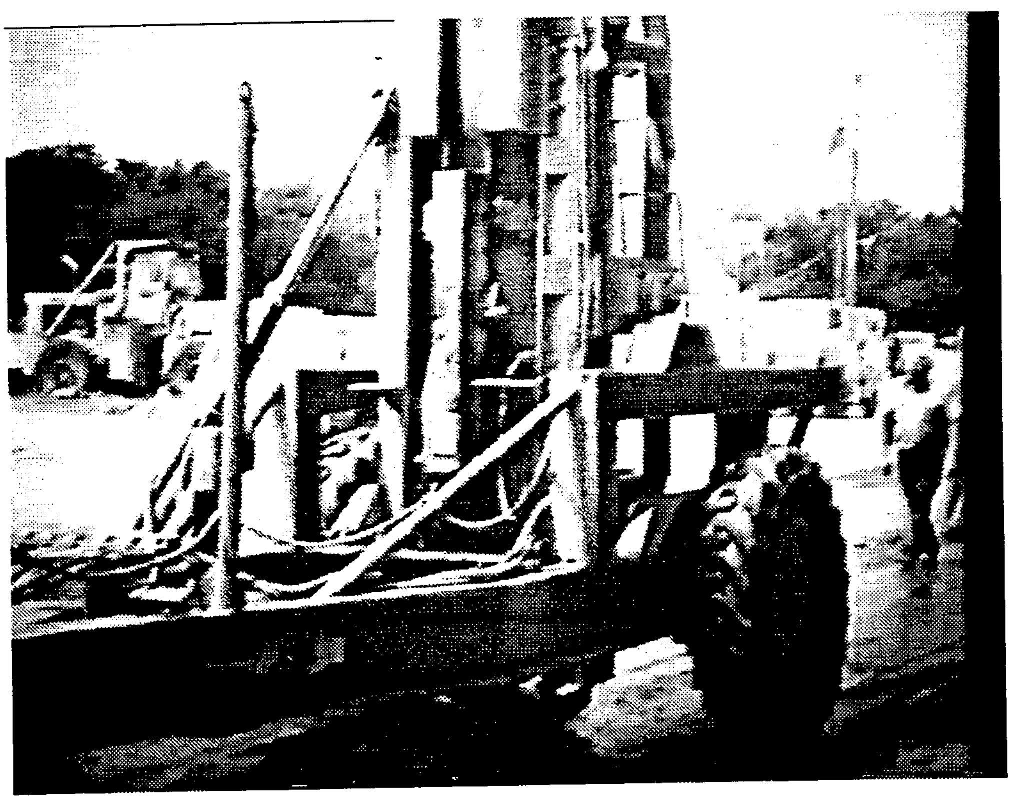

Figure 2-7. Pile-driving Hammer Equipment

2.4 Seating Equipment

A variety of rollers, including pneumatic tired rollers in the load range of 31,750 to 45,360 kg (35 to 50 tons), steel wheeled or vibratory types in the load range of 9,070 kg (10 ton) have been used in seating fractured PCC slabs using a wide range of passes. Experience by other states/agencies has shown that the fractured pavements can be over-rolled, causing weakening of the subgrade and a consequent increase in deflections. The consensus of the states seating fractured pavements is that 3 to 7 passes of a 31,750 to 45,360-kg (35 to 50-ton) pneumatic tired roller are adequate to seat cracked PCC slabs, depending on roller weight and condition of pavement foundation, while at least 2 passes each of a 9,070-kg (10-ton) vibratory and a pneumatic tired roller are adequate for compacting a rubblized pavement. Figure 2-8 shows a vibratory roller fitted with a series of 25 mm (1 in.) wide bars arranged in a herringbone pattern to assist in completing the fracturing process.

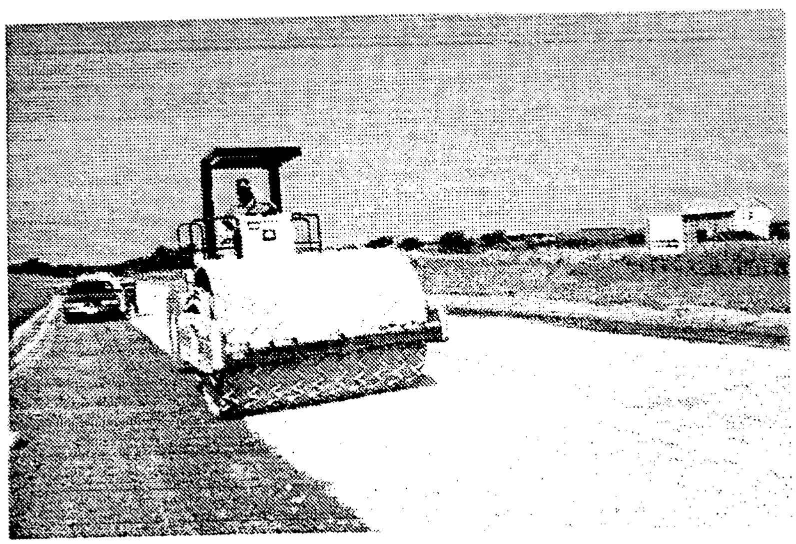

Figure 2-8. Steel-Wheel Herringbone Vibratory Compactor on a Rubblized Project

Table 1. Summary Characteristics of Fracture Equipment Types

Equipment Type	Applications	Energy Range (J)	Production Rate (lane-km/day)	Advantages	Limitations
1. Crane & Wrecking ball	Small areas where specialized equipment is not justified		0.4 -1.2	Equipment generally available	Requires skilled crane operator
2. Whip hammer	JPCP	6,580-20,550	1.6	High productivity, covers full lane width	Not effective on JRCP, slab tenting occurs
3. Guillotine	JPCP & JRCP	16,440-164,400	1.2 - 6.4	Versatile-effective with JPCP and JRCP, preferred by several states; high productivity, covers full lane width	None reported
4. Impact hammer		2740		Rubblized portions of concrete below cracks	None reported
5. Pile driving hammer	JRCP	10,960-157,550	0.43	Covers full lane width	Low productivity
6. Resonant pavement breaker	JPCP, JRCP, CRCP	< 2740	0.8	Excellent for rubblizing	Can shove or punch through pavement on weak subgrade
7. Antigo Multiple-head breaker	JPCP, JRCP, CRCP	2,740-9,590	1.6	Excellent for rubblizing, adjustable width to full lane	None reported

Source: Modified from *Guidelines and Methodologies for the Rehabilitation of Rigid Highway Pavements Using Asphaltic Concrete Overlays*; Pavement Consultancy Services, Beltsville, Md., 1991[8]

2.5 Performance of Fractured Pavements

Performance of pavements that have been fractured and then overlaid has been evaluated in regard to both structural and functional characteristics. The structural performance is evaluated by using measures such as the extent of reflective cracking, rutting, and deflection. These measures give some indication of the pavement's ability to maintain its structural integrity. The functional performance measures the quality of the riding condition, from the point of view of the traveling public, and is measured by the pavement's surface roughness. Current practice suggests that the rubblization technique has not yet been subjected to use over enough time to evaluate the long-term performance experience. However, the short-term or preliminary results, from agencies using the technique, indicate that rubblization has a great potential for providing a pavement that will perform well structurally and functionally. For crack and seat projects, there is the indication that reflective cracking will eventually develop, but proper application of the techniques will delay the development of the cracks. The extent of this delay however, has not been clearly documented in the literature.

Some states, including Michigan, require the installation and proper functioning of edge drains for all PCC pavements prior to rubblizing[11]. This requirement may be due to the fact that the rubblization process results in some form of open-graded PCC base course which would allow water to seep through the base layers, creating a need for drainage system installation. A review of data base records for more than 25 rubblized projects also indicated that the majority of the pavements rubblized in Michigan were constructed on top of sand or select subbases ranging in thickness from 300 to 450 mm (12 to 18 in.), thus requiring the installation of drainage systems to drain any water potentially trapped by the sand subbases. California also requires edge drains for crack and seat projects[12]. Antigo Construction's recent experience with rubblization projects indicates that the installation of edge drains is becoming a standard practice. The impact of drainage improvements on fractured pavement performance and its cost effectiveness, however, have not been reported.

Projects in Michigan also showed that the pre-fracture technique is used on the majority of the projects to achieve a better production rate. Michigan plans for a design service life on rubblized projects of between twelve and twenty years. Of twenty-eight projects undertaken and evaluated from 1989 through 1993, half (14) were on target for the predicted service life as of 1996, and three were inconclusive.[11] Of the eleven not on target, all showed intermittent to extensive transverse cracking, and six of these also had longitudinal wheel path rutting between 6 mm (1/4 in.) and 9 mm (3/8 in.) deep. Ten of the eleven had also experienced difficulties with the rubblization process, to include inadequate base support or non-uniform rubblization.

Despite the crucial importance of factors such as loading, subgrade condition, overlay thickness, drainage quality, and environment (in terms of temperature and precipitation) on pavement performance, relatively little systematic attempt has been made to examine the interrelationship between any of these factors and the overlay's performance.

2.6 Overlay Thickness Design

The state-of-the-art concerning overlay design thickness is limited. Most agencies using the fracturing techniques rely on their past experience and engineering judgment when designing an overlay thickness; hence, the thickness varies considerably from agency to agency. The 1993 AASHTO guide[13] contains suggested procedures for these techniques. In the guide, the seated slab pieces are considered to behave more like a flexible pavement than a rigid pavement. The main critical design element associated with the procedure is the determination of the effective structural capacity of the existing fractured PCC pavement which is dependent on the pavement layer thicknesses, drainage quality, and the structural layer coefficients.

Design of an overlay that incorporates crack and seat or rubblization can follow the equation:

$$SN_{ol} = a_{ol} * D_{ol} = SN_f - SN_{eff}$$

where: SN_{ol} = Required overlay structural number
a_{ol} = Structural coefficient of the overlay
D_{ol} = Required overlay thickness in inches
SN_f = Structural number required for future traffic
SN_{eff} = Effective structural number after fracture

This equation can be rearranged to solve directly for the thickness, D_{ol}:

$$D_{ol} = \frac{(SN_f - SN_{eff})}{a_{ol}}$$

The Structural Number for future traffic can be determined by methods from the AASHTO guides or agency guidelines. It is dependent on the layer coefficients of the broken slab and any layers below forming part of the pavement structure. Layer coefficients currently in use by some agencies are given in Table 2, on the following page.

Table 2: **Fractured Slab Layer Coefficients in Use by Agencies**

Agency Name	Material	Slab Condition	Coefficient Metric (English)
AASHTO[13]	Break & Seat---JRCP	Pieces greater than 0.3 m (1-ft) with ruptured reinforcement or broken steel/concrete bond.	0.0079-0.0138 (0.20-0.35)
	Crack & Seat---JPCP	Pieces 0.3 to 0.9 m (1 to 3 ft)	0.0079-0.0138 (0.20-0.35)
	Rubblized PCC (any pavement type)	Completely fractured slab with pieces less than 0.3 m (1-ft)	0.0055-0.0118 (0.14-0.30)
Michigan DOT[14]	Rubblized PCC (any pavement)	-	0.0087 (0.22)
	Crack & Seat	-	0.0098 (0.25)
WisDOT[15]	Rubblized PCC (any pavement)	-	0.0079-0.0094 (0.20-0.24)
North Carolina DOT[16]	Crack & Seat	-	0.0079-0.0157 (0.2 -0.4)

The effective structural number (SN_{eff}) of the fractured pavement structure can be calculated as:

$$SN_{eff} = a_2 D_2 + a_3 D_3$$

where: a_2 = Structural coefficient of fractured layer
D_2 = Thickness of fractured layer
a_3 = Structural coefficient of existing base
D_3 = Thickness of existing base

2.7 Pavement Foundation and Condition

Very little attention has been devoted to characterizing the condition and type of soil beneath the pavement slab prior to fracturing, despite the crucial importance of these factors on the fracturing and seating operation, and on the overall performance of the fractured pavement after the overlay. Many states have indicated that pavement fracturing operations will not be effective on fine grained subgrades, but have not further studied the soils for which this applies.

Of more than 25 projects rubblized in Michigan, three projects, located respectively on Old US 16, Saginaw road (Old US 10), and M-44, produced inconsistent and incomplete fractured slab pieces using the resonant PB4 due to the presence of weak subgrades. The large pieces had to be broken using jack hammers. A review of soil survey maps for these areas indicated that the pavements lay on top of subgrade material classified as A-6 by AASHTO definition. These were the only projects reported by Michigan to be on an A-6 soil. A portion of a rubblization project on M-68 also had to be discontinued due to the presence of a high water table although the subgrade showed an AASHTO classification of A-2. The Michigan experience[11] indicates that the most difficult areas to satisfactorily rubblize include pavement sections located on poor subgrades such as clay, pavement edges where there is little or no lateral support, and undowelled full depth concrete patches.

Evaluation of rubblization projects in Michigan also indicated that rubblization should not be used if the subgrade has a resilient modulus of less than approximately 24 Mpa (3500 psi). A further evaluation of the subgrades under poorly performing rubblization projects indicates that A-6 and A-7 soils should be avoided, especially if high moisture contents are present. The moisture content that defines this boundary between successful and unsuccessful rubblization has not been identified, however.

The North Carolina Department of Transportation conducted deflection studies on crack and seat pavements and concluded that "crack and seat pavement rehabilitation is likely to perform well when subgrade moduli are consistently above 103.3 MPa (15,000 psi) after cracking"[16].

2.8 Unit Cost of Fractured Pavement Projects

If any meaningful life cycle cost analysis (LCCA) will be performed, realistic cost data on the various fracturing techniques are essential to examine various strategies for controlling reflective cracking. Costs vary widely depending on the type of fracturing technique, project size, and the geographical location.

The total fracture and compaction cost for thirty-six crack and seat projects performed in Wisconsin between 1982 and 1995 ranged from $\$0.19/m^2$ to $\$0.84/m^2$ ($\$0.16/yd^2$ to $\$0.70/yd^2$), with an average cost of $\$0.30/m^2$ ($\$0.25/yd^2$). Illinois' 1983-1987 crack and seat project costs ranged from $\$0.61/m^2$-$\$0.83/m^2$ ($\$0.51/yd^2$-$\$0.70/yd^2$). In 1991, California reported crack and seat costs of $\$0.60/m^2$ -$\$0.89/m^2$ ($\$0.50/yd^2$-$\$0.75/yd^2$).

The cost for twelve rubblization projects performed in Wisconsin between 1989 and 1996 ranged from $\$1.24/m^2$ to $\$2.22/m^2$ ($\$1.03/yd^2$ to $\$1.85/yd^2$), excluding special projects. The average price for the PB4 was $\$1.81/m^2$ ($\$1.51/yd^2$) and $\$1.50/m^2$ ($\$1.50/yd^2$) for the MHB. Michigan reported rubblization costs ranging from $\$1.09/m^2$ to $\$2.98/m^2$ ($\$0.91/yd^2$ to $\$2.48/yd^2$) for projects performed between 1989 and 1995. The average cost was $\$2.38/m^2$ ($\$1.65/yd^2$).[14]

3.0 **Performance of Overlaid Fractured Slab Pavements**

Performance of overlaid fractured pavements is crucial for making comparative assessments of rehabilitation methods for PCC pavements, and ultimately for the overall life cycle cost analysis involving pavement selection options. Various performance model forms exist for most pavement types, but very little systematic modeling has been done regarding the performance of fractured PCC slabs overlaid with HMA.

3.1 Performance Model Building for Fractured Pavements

Three main indicators of performance have been analyzed in this study, as directed by WisDOT. These include pavement distress index (PDI), international roughness index (IRI), and present serviceability index (PSI).

The PDI is a mathematical expression developed by WisDOT for pavement condition rating based on observable surface distresses. It reflects the composite effects of various distress types on the condition of a pavement segment.[17] Distress surveys are conducted biennially on the mainline State Trunk Highways to evaluate the network. Distresses are assigned a value, based upon type of distress, severity, and extent. Distress values are totaled for a pavement, with a value of 0 reflecting no distresses, rising to a maximum value of 100.

The IRI is the standard reference for road roughness based on a specific mathematical model of a longitudinal surface profile in the wheel path of a road profiler. A higher IRI value indicates a poorer ride.

The PSI is a mathematical model developed at the American Association of State Highway Officials (AASHO) Road Test in the early 1960s. It relates subjective panel ratings of a pavement segment to objective measurements of distress and roughness as a measure of pavement serviceability to the road user. PSI values range from a high of 5.0 for a perfect pavement, to a low of 0. A threshold value for repairs/reconstruction is set by each agency at a value ranging from 2.0 to 3.0.

The main factors assumed to influence these performance indicators are the subgrade condition, traffic level, overlay thickness, pavement surface age, PCC pavement type, and the method used in fracturing the PCC slab, i.e., either by cracking and seating, or rubblizing.

Soil condition for each pavement subgrade is represented in this analysis by the soil support value (SSV), which in all cases was estimated using county soil survey maps in conjunction with Geotechnical Bulletin No. 1[18]. This bulletin relates Wisconsin soil support value, group index, and frost characteristics.

Functional classification of pavements was used as a surrogate for traffic loading since actual traffic loading data was unavailable. Four functional classes, including interstate, principal arterial, minor arterial, and major collector were represented in the database. The performance indicator values were extracted from WisDOT pavement management records. The records indicate that IRI values for fractured pavements have been measured since 1990, while PDI and PSI values have been respectively measured since 1985 and 1981 for some fractured pavements.

The first step involved in the analysis was to compute the average value of each of the performance indicators for each project for every year the indicator was measured. The next step was to identify projects by type of PCC pavement prior to fracture, i.e., whether pavement is a jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), or continuously reinforced concrete pavement (CRCP). Projects were further classified by the method of fracture for each pavement type. Only two fracture methods were applicable in this research: crack and seat and rubblization. Crack and seat projects have been performed using guillotine drop hammers, which produce fractured slab pieces approximately 600 mm (24 in.) per side. Rubblization projects have used either a PB4 pavement resonant breaker or Antigo Construction's multiple-head breaker (MHB), to produce pieces less than a nominal 225 mm (9 in.) in largest dimension.

For each pavement type and fracture method combination, a preliminary analysis was conducted to examine the effect of functional classification on each of the indicators of performance. The results indicate that PDI variation on JRCPs is sensitive to functional classification, and that the highest PDI averages occur on interstate highways compared to all other highways combined. There were differences in the mean IRI and PSI values for the various functional classes, however, these differences were not statistically significant to warrant the use of the functional classification variable in further analysis involving PSI and IRI. The next step in the analysis involved observing trends and conducting a series of simple and multiple -linear regression analyses to relate each performance indicator to the main influential factors. Figures 3-1a and 3-1b show respectively the PDI trends for cracked and seated JPCP and JRCP projects; both appear to show increasing PDI values with pavement age in a linear fashion.

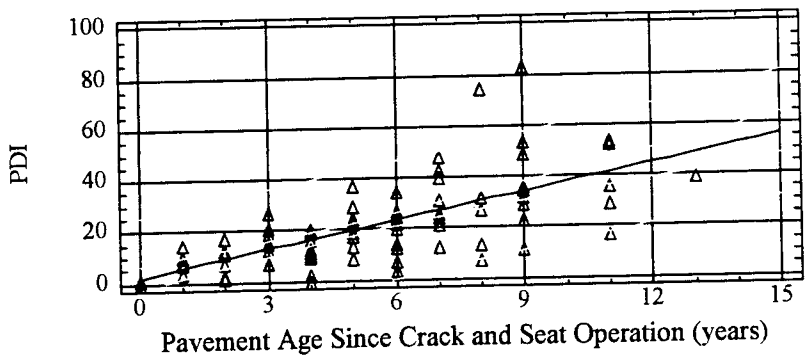

Figure 3-1a: PDI Trend for Cracked and Seated *JPCP*

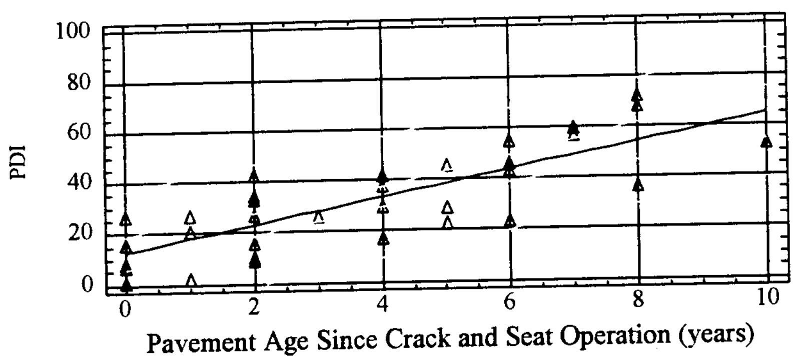

Figure 3-1b: PDI Trend for Cracked and Seated *JRCP*

In Figure 3-1c and 3-1d, the trends for PSI are shown respectively for cracked and seated JPCP and JRCP. Both show declining serviceability with pavement age and initial PSI of approximately 4.5. Figure 3-1e shows increasing IRI with pavement age for cracked and seated JPCP projects with an initial IRI of approximately 1.0m/km for newly overlaid fractured JPCP pavements.

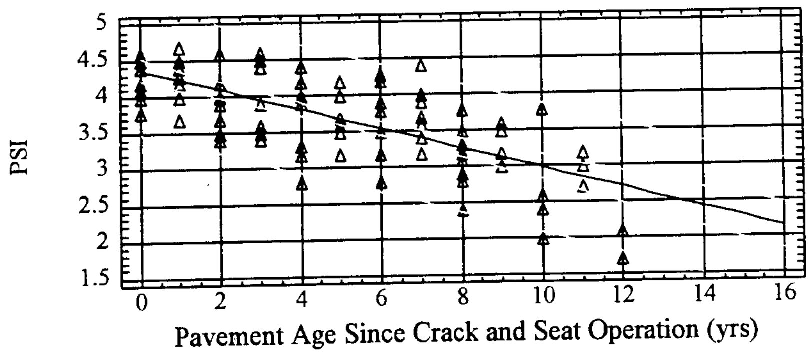

Figure 3-1c: PSI Trend for Cracked and Seated *JPCP*

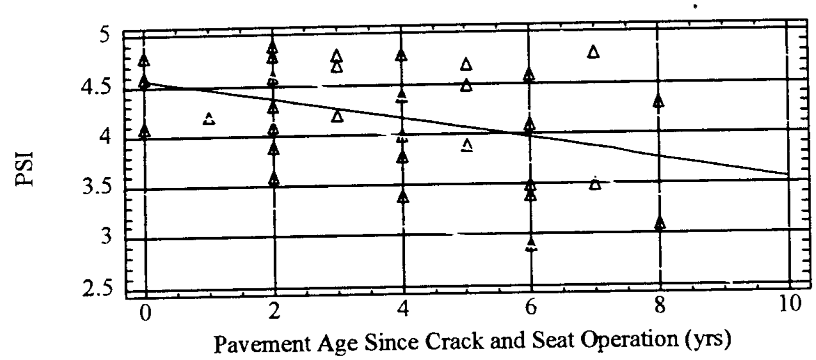

Figure 3-1d: PSI Trend for Cracked and Seated *JRCP*

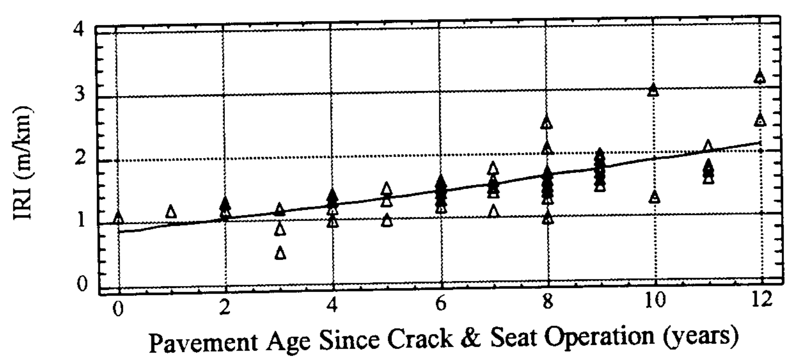

Figure 3-1e: IRI Trend for Cracked and Seated *JPCP*

The overall statistical models developed for each of the performance indicators are summarized in Table 3 by pavement type for all cracked and seated projects. The basic statistical estimates for the model variables are presented in Table 4. Table 3 indicates that pavement age, overlay thickness, and subgrade conditions are the main factors influencing IRI and PSI for cracked and seated JPCP. Subgrade condition and pavement age are the critical factors influencing the deterioration of JRCP pavements when PSI is used as the indicator of performance. The main factors explaining the variation in PDI for JPCP are overlay thickness and age, while PDI variation on JRCP is dictated by age and functional classification

Table 3. **Performance Models for Crack & Seat Pavements in Wisconsin**

Performance Indicator	Pavement Type	Model Form	R^2	S.E	# of obser-vations (n)	# of projects represented
PSI	JPCP	**$PSI_t = 3.45+0.33SSV- 0.13t- 57.79/h$**	**0.680**	0.37	95	22
		$PSI_t = 4.37 - 0.14t$	0.531	0.44	95	22
	JRCP	**$PSI_t = 1.9- 0.1t + 0.6SSV$**	**0.653**	0.34	32	8
		$PSI_t =1/[0.22+0.01t]$	0.197	0.03	32	8
IRI	JPCP	**$IRI_t =1.4 - 0.21SSV + 0.01t^2 + 66.19/h$**	**0.692**	0.27	56	22
		$IRI_t = 1.06 +0.01t$	0.502	0.34	56	22
	JRCP	$IRI_t = 1.02 + 0.07t$	0.425	0.21	20	8
PDI	JPCP	**$PDI_t = -15.4 + 3.02t +1709.8/h$ ***	**0.737**	7.3	85	22
		$PDI_t = 3.1t$	0.562	8.7	85	22
	$JRCP_{Interstate}$	**$PDI_t = 15.4 + 5.8t$**	**0.741**	9.2	23	5
	$JRCP_{other\ hwys}$	**$PDI_t = 5.4 + 4.2t$**	**0.829**	6.5	11	4

t = pavement age in years since the crack and seat operation; h = overlay thickness in mm; SSV = soil support value
S.E.= standard error; * excludes Interstate pavements because of lack of sufficient data

Table 4: **Basic Statistical Estimates of Performance Model Variables**

Performance Indicator	PCC Type	Basic Statistic	Variable				
			PCC Slab thickness (mm/in)	HMA Overlay thickness (mm/in)	Pavement Age (years)	Soil support value	Performance Indicator Estimate
PSI	JPCP (n = 95)	Mean value	220/8.8	95/3.8	4.6	4.5	3.7
		Std. deviation	12.5/0.5	22.5/0.9	3.3	0.4	0.6
		Range	175-225/7-9	50-162/ 2-6.5	0-12	4.1-5.4	1.7-4.7
	JRCP (n =32)	Mean value	222.5/8.9	92.5/3.7	3.7	4.6	4.2
		Std. deviation	25/1.0	22.5/0.9	2.4	0.7	0.6
		Range	150-250/6-10	75-137/ 3-5.5	0-8	3.4-5.4	2.9-4.9
IRI	JPCP (n = 56)	Mean value	222.5/8.9	97.5/3.9	6.6	4.6	1.5
		Std. deviation	7.5/0.3	22.5/0.9	2.9	0.4	0.5
		Range	200-225/8-9	50-162/ 2-6.5	0-12	4.1-5.4	0.5-3.2
PDI	JPCP (n = 86)	Mean value	222.5/8.9	100/4.0	4.9	4.6	19.5
		Std. deviation	7.7/0.3	27.5/1.1	3.0	0.4	17.2
		Range	200-225/8-9	50-162/ 2-6.5	0-13	4.1-5.4	0-82
	JRCP (n= 34)	Mean value	215/8.6	102.5/4.1	3.7	4.9	31.7
		Std. deviation	27.5/1.1	20/0.8	2.8	0.7	18.4
		Range	6-10	3-5.5	0-10	3.4-5.4	1-72

n = number of observations

3.2 Engineering Implications of Crack and Seat Pavement Performance Models

Performance prediction over the life cycle of a pavement is critical for both pavement management and design. The performance models developed for the HMA overlays on crack and seat PCC pavements could be used to determine the timing and, consequently, the type of maintenance or rehabilitation intervention based on critical levels of the performance indicators as determined by WisDOT policy.

The PSI variations with pavement surface age are shown in Figures 3-2a through 3-2c for three overlay thicknesses (75, 100, and 125 mm) under various soil conditions, represented by the soil support value. Figure 3-2d shows a similar variation for JRCP, except that the variation is independent of overlay thickness. Variations based on IRI are also shown in Figures 3-2e through 3-2g, while variations based on PDI are shown in Figures 3-2h and 3-2i. Figures 3-2a through 3-2c and Figures 3-2e through 3-2g indicate that the PSI and IRI immediately after construction are dependent on the subgrade quality and the thickness of the overlay. High initial PSI and low IRI values are associated with pavements located on fair to good soils (soil support value in the range of 4 to 5) compared to those located on poor to fair soils (soil support value in the range of 3 to 4). In addition, the initial PSI and IRI values do improve with overlay thickness. The lower PSI and higher IRI values associated with crack and seat pavements located on weak soils are due to the instability of the soil under construction equipment. This results in pavement unevenness after compaction.

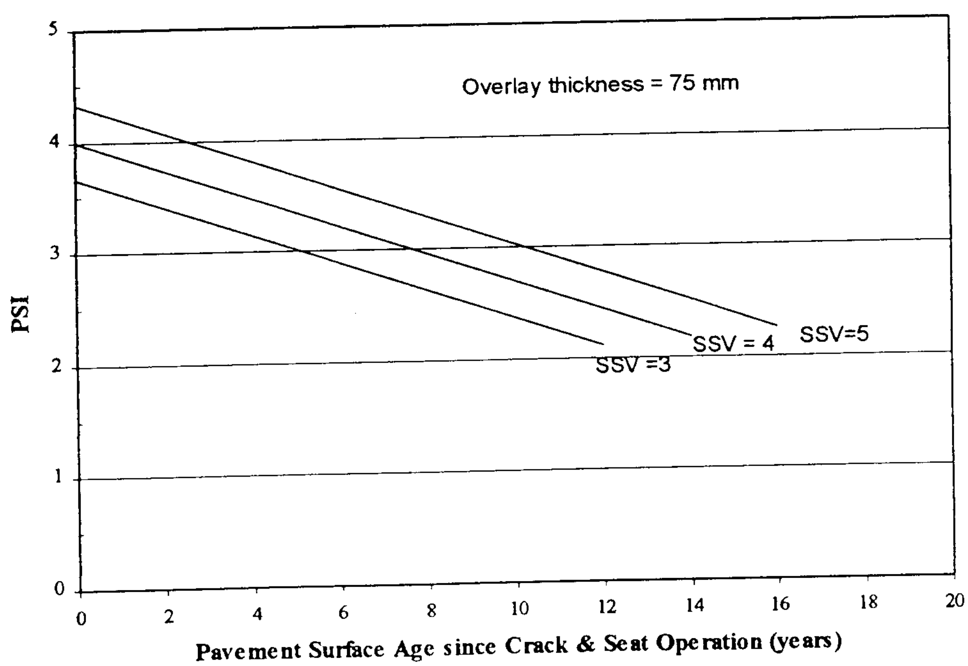

Figure 3-2a: **PSI** Variation with Surface Age for **75 mm** HMA Overlay of Cracked and Seated **JPCP** in Wisconsin

The high initial PSI and lower IRI values associated with thick overlays are due to the fact that, thick overlays have increased structural strength and weight and, hence, the ability to suppress ride quality related defects that would have originated from beneath the pavement structure.

Figure 3-2d indicates that for JRCP, soil support value alone may be adequate for estimating service life, and that overlay thickness is not crucial for service life estimates for JRCP. This may suggest that the support provided by fractured JRCP to the pavement structure appears to be more stable compared to that provided by JPCP, and hence, pavement foundation movement contribution to surface ride quality will be lower for JRCP compared to JPCP which would require thicker overlays to mask the effect of foundation movements in order to improve ride quality. The stability provided by the JRCP foundation may be due to the wire mesh reinforcement acting as a strengthening fabric in the seated fractured pavement.

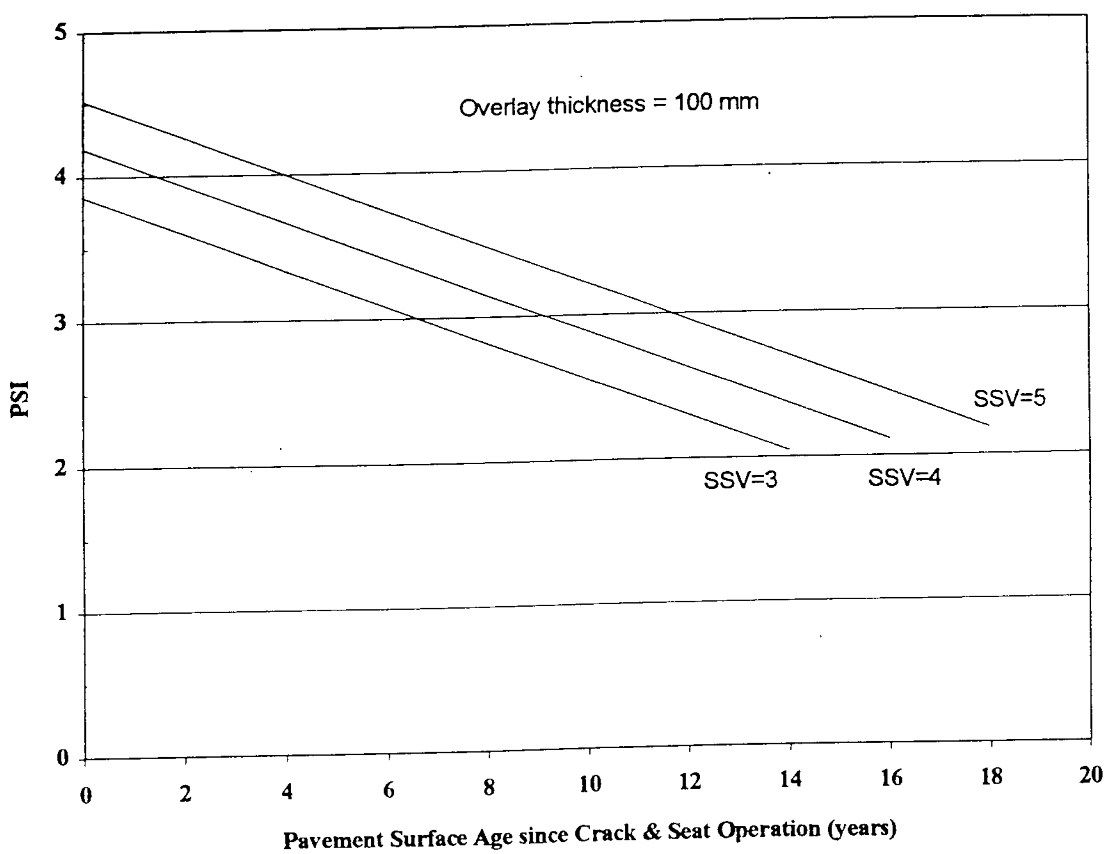

Figure 3-2b: **PSI** Variation with Surface Age for **100** mm HMA Overlay of Cracked and Seated **JPCP** in Wisconsin

The results also indicate that in general, for the same soil condition and overlay thickness, performance decisions based on IRI will trigger slightly higher service life values compared to those based on PSI. The difference could be in the order of approximately 1-3 years especially when soil conditions are poor and overlay thickness is minimum (≤75 mm). The difference tends to be smaller for good soil conditions. Either IRI or PSI inputs can be used to determine performance with little service life difference.

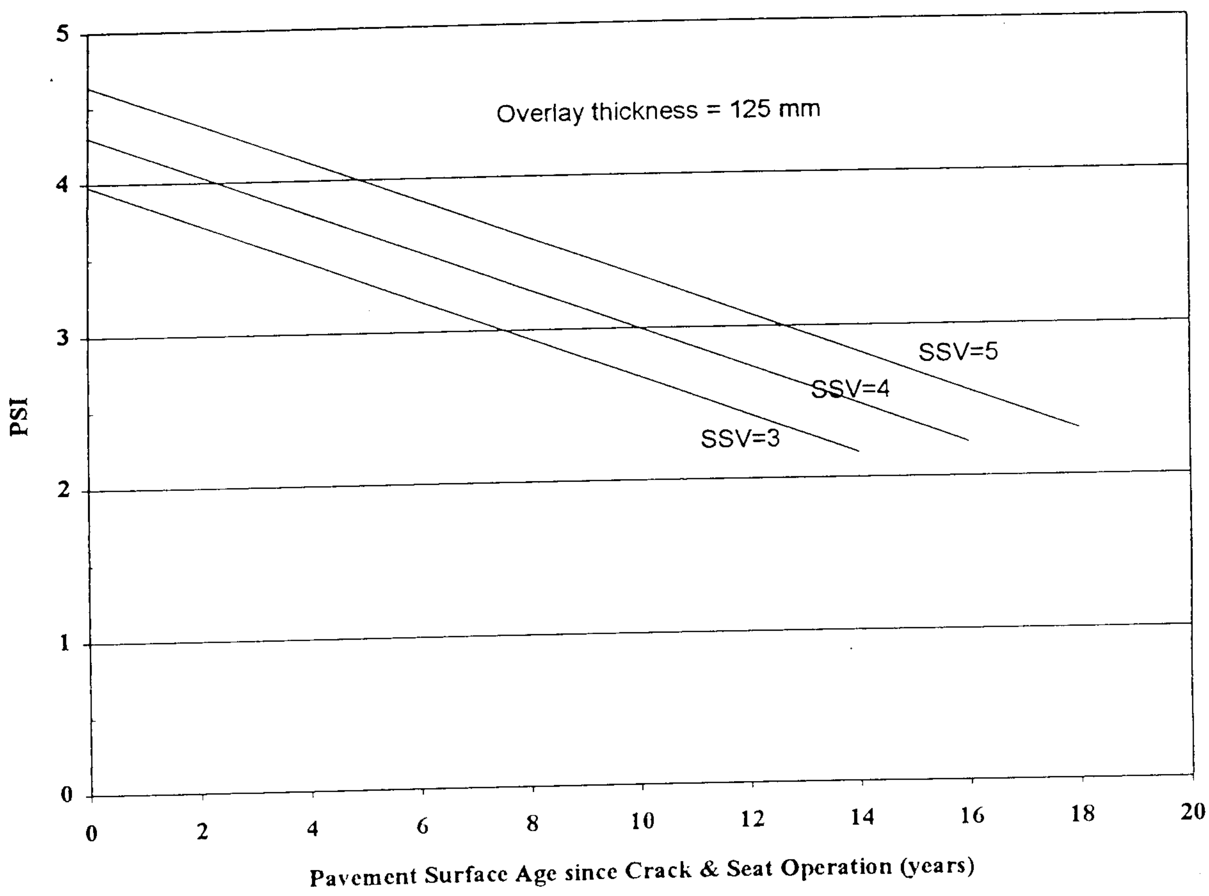

Figure 3-2c: **PSI** Variation with Surface Age for **125** mm HMA Overlay of Cracked and Seated **JPCP** in Wisconsin

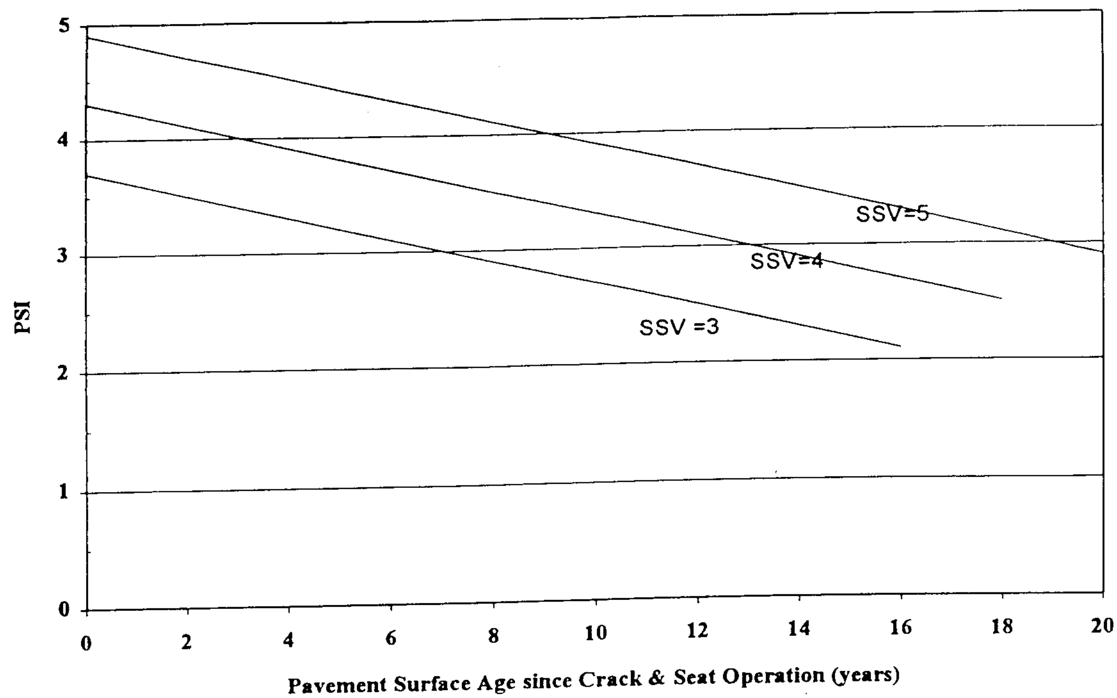

Figure 3-2d: **PSI** Variation with Surface Age for HMA Overlaid Cracked and Seated **JRCP** in Wisconsin

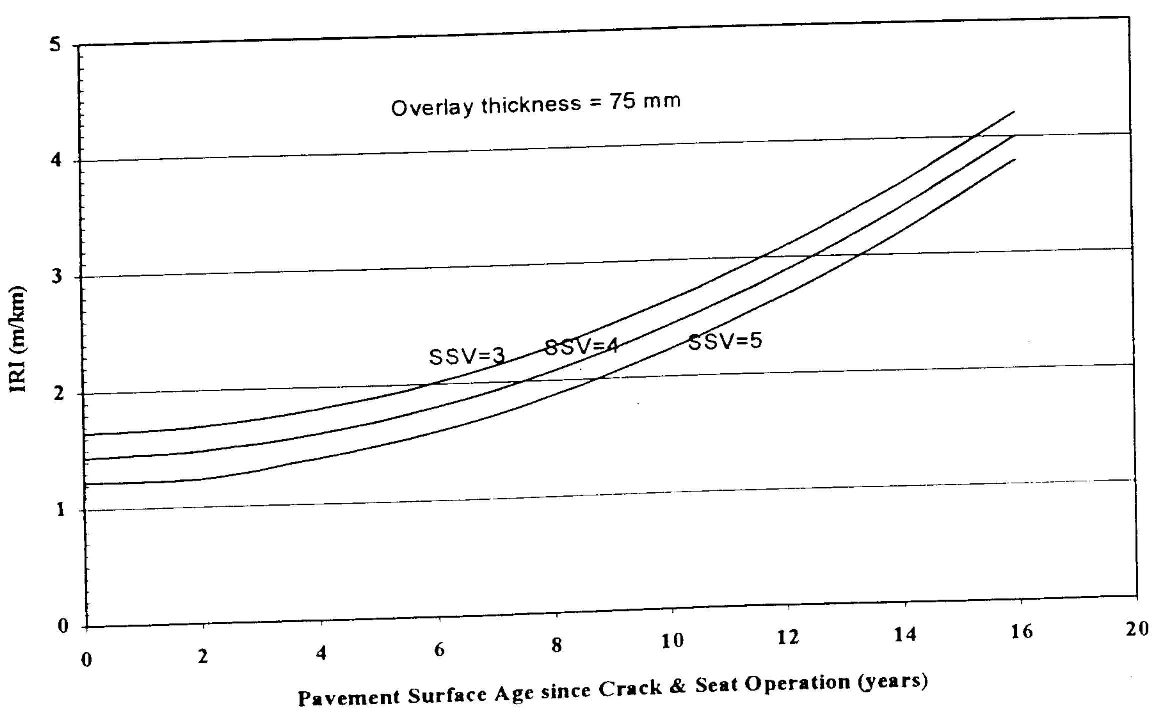

Figure 3-2e: **IRI** Variation with Surface Age for 75 mm HMA Overlay of Cracked and Seated **JPCP** in Wisconsin

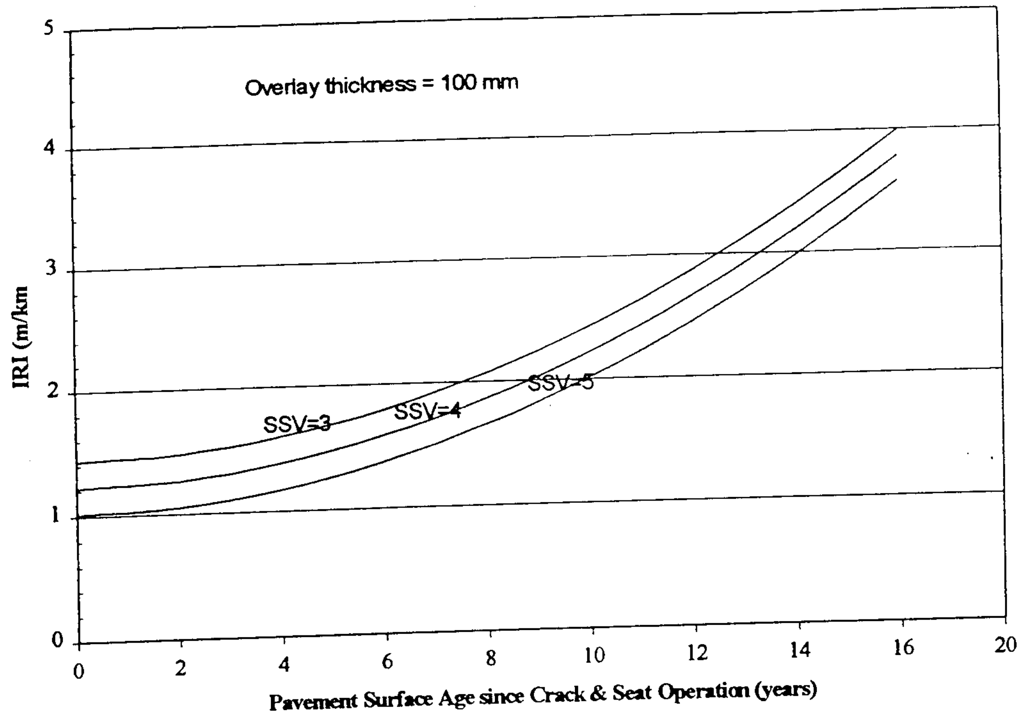

Figure 3-2f: **IRI** Variation with Surface Age for **100** mm HMA Overlay of Cracked and Seated **JPCP** in Wisconsin

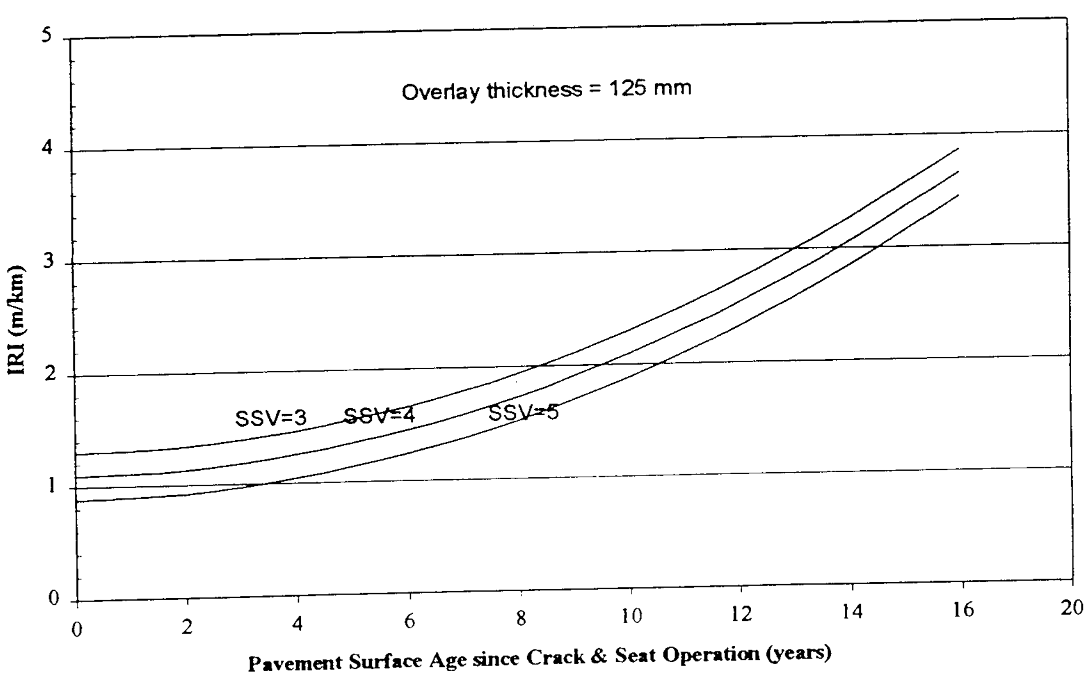

Figure 3-2g: **IRI** Variation with Surface Age for **125** mm HMA Overlay of Cracked and Seated **JPCP** in Wisconsin

Considering PDI as the performance indicator, the relationship between pavement surface age and overlay thickness for HMA on cracked and seated JPCP is shown in Figure 3-2h for all pavement functional classes (principal arterial, minor arterial, and major collector) combined. Thicker overlays on cracked and seated pavements will reach a given PDI level after a longer service life. This may be due to their ability to withstand structural related distresses compared to thin overlays.

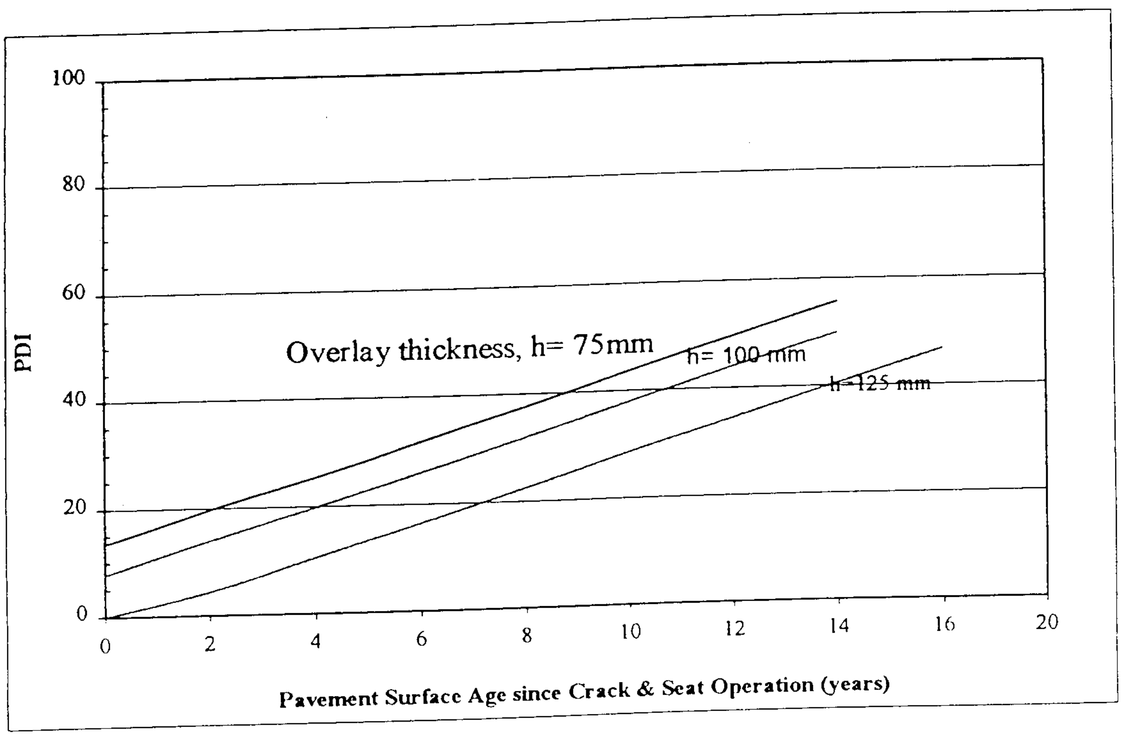

Figure 3-2h: **PDI** Variation with Surface Age for HMA Overlaid Cracked and Seated **JPCP** in Wisconsin

For HMA on cracked and seated JRCP, Figure 3-2i indicates that service life is dependent on the functional classification. For the same critical PDI, Interstate highways have shorter service lives compared to all other highways. This is probably due to structural related distresses resulting from the high level of traffic loading experienced on Interstate highways.

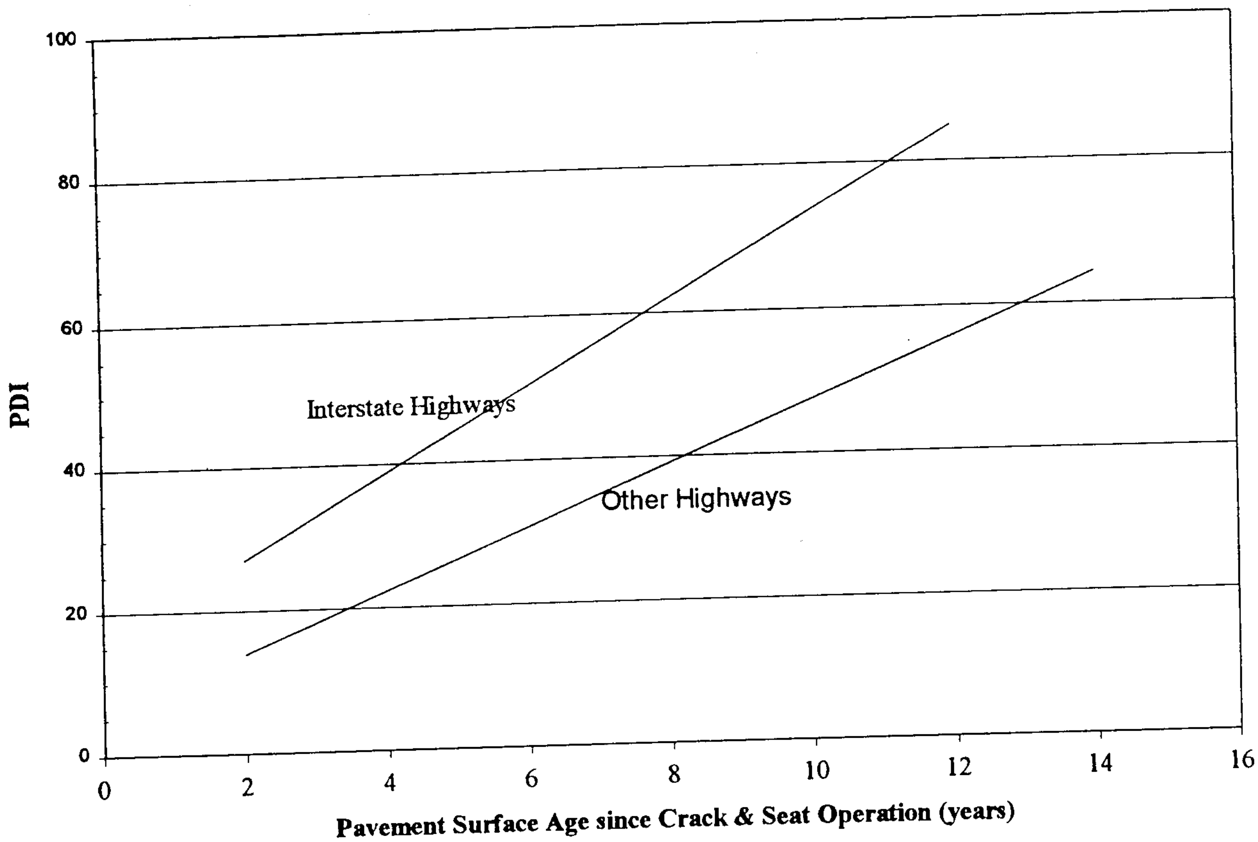

Figure 3-2i: **PDI** Variation with Surface Age for HMA Overlaid Cracked and Seated **JRCP** in Wisconsin

In general, the overall results of these analyses indicate differences in estimated pavement service lives depending on the parameter (i.e. PSI, IRI, PDI) used as the indicator of performance and the threshold values adopted.

3.3 Comparison of Fractured Versus Non-fractured Overlaid PCC Pavements

Figure 3-3 shows the performance variation in terms of PDI versus surface age for fractured and non-fractured overlaid JPCP. Due to data limitations, an overall trend could not be established for rubblized projects. The performance characteristics of five individual rubblized JPCP projects which have been monitored for distresses are shown, together with established trends for crack and seat, and composite pavements (AC/JPCP).

The established trend for the composite pavement (AC overlay on non-fractured PCC) was supplied by Mr. Scot Schwandt, a pavement engineer with WisDOT, while that for the crack and seat is from Table 3 of this report. Most of the rubblized projects are described in Chapter 4 of this report.

The short-term (up to 7 years) performance analysis indicates that the rubblized pavements, with the exception of USH 8, appear to outperform both the crack and seat and the composite pavements. Unlike the other rubblized projects which rested directly on aggregate bases, the USH 8 pavement rested on a 250-mm (10-in.) previously cracked and seated JPCP. The rubblization process did not fracture the underlying layer, but it was debonded from the rubblized 225-mm (9-in.) top JPCP layer. The anomaly in performance may have resulted from an additional cracking progression from the underlying cracked and seated JPCP lower layer reflecting through the fractured layer onto the 100-mm (4-in.)asphaltic overlay.

The trends shown in Figure 3-3, on the next page, do not consider the effects of soil and overlay thickness, due to the lack of sufficient data in these variables. The preliminary results, however, suggest that rubblization has the greatest potential to minimize the effects of early reflective cracking in overlaid PCC pavements.

The composite pavements appear to have worse performance for the short term, but then deteriorate at a lower rate compared to the cracked and seated pavements.

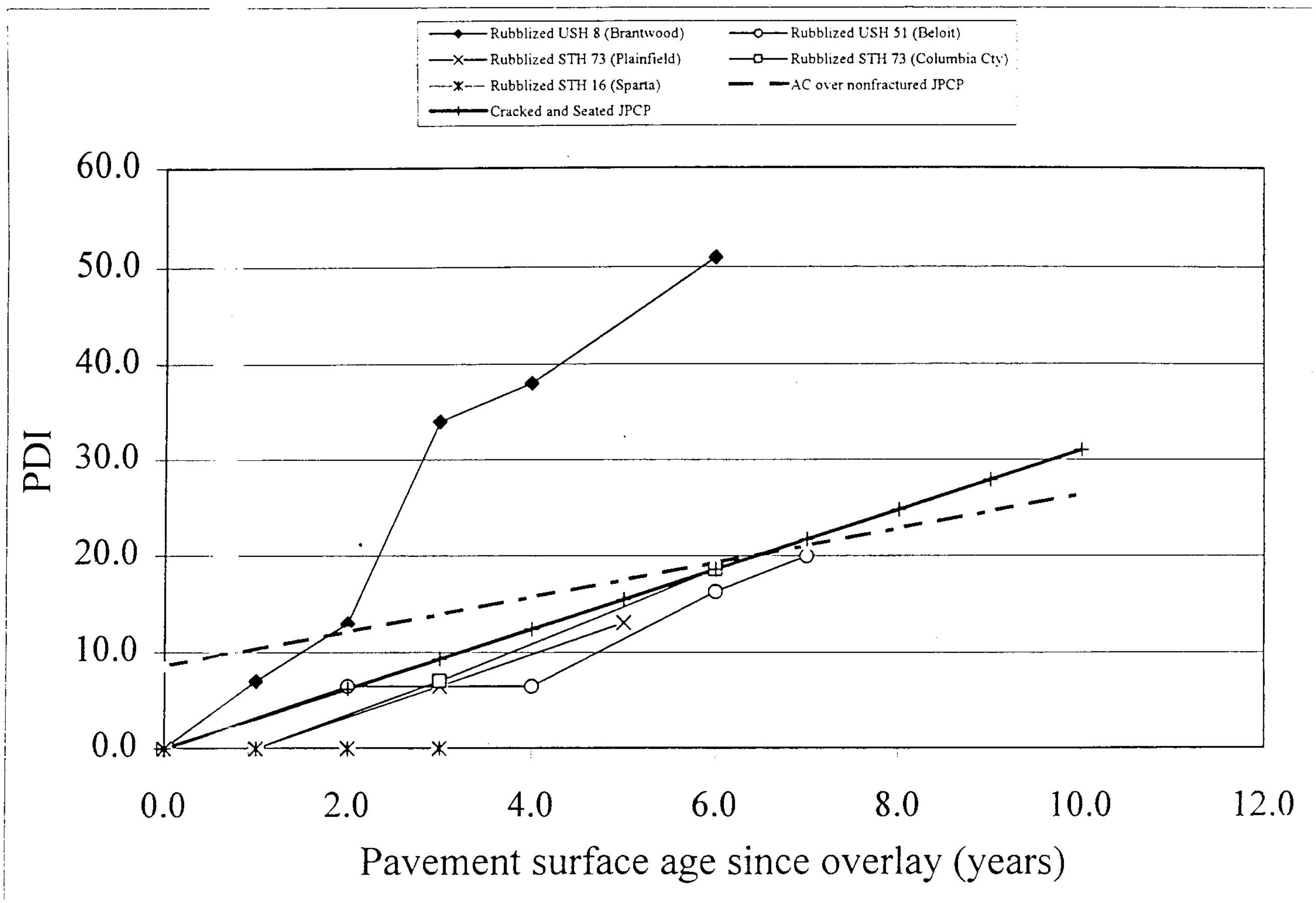

Figure 3-3: Comparison of PDI Trends of Fractured Versus Non-fractured Overlaid JPCP Pavements in Wisconsin

4.0 **Rubblization Projects**

Since 1989, several rubblization projects have been performed in Wisconsin. The database that was developed as a part of this study provides the detailed information on these projects. Some of the key projects that have been monitored by WisDOT are summarized in this section.

4.1 State Highway 16 Rubblization Project

In order to evaluate the effectiveness of rubblization as a slab fracture technique, a project was selected for evaluation. This project included both a control section which was repaired, and a test section that was rubblized in August, 1996.

The pavement section rubblized is located on STH 16 west of Sparta in Monroe County. The project begins at the intersection with County Highway B (5th Avenue) and ends near the intersection of Hamlet Road (Cliff Drive), approximately 4.2 km (2.5 miles) to the west of the starting point. The location is approximately 1 km (0.6 mile) from the western city limits of Sparta.

The original pavement was a nominal 200 mm (8 in.) thick jointed reinforced concrete pavement constructed in 1955 with joint spacing of 24 m (80 ft.) for most of the project length. Joint spacing was 18 m (60 ft.) from station 66+00 to station 91+45. Reinforcement consisted of centerline longitudinal reinforcing of either steel tie bars or a 0.6-m (2 ft.) wide welded wire fabric. There was no reinforcing at transverse joints. The road consisted of two 3.6 m (12 ft.) travel lanes, one in each direction, with a gravel shoulder with a width between 3.0 and 3.6 m (10 and 12 ft.). The original pavement rested on a 150-mm (6-in) stone base layer on top of a subbase ranging from none to 600 mm (24 in.) of sand.

4.1.1 Experimental Design

The project included both a rubblized section and a control section. The control section was 0.45 km (1490 ft.) long, located at the east end of the project. The rubblized section was 3.6 km (2.25 miles) long, with transition sections between the control and rubblized sections. The control section was repaired by performing full depth removal and replacement of slabs at failed joints.

The test section was initially to be rubblized according to the following specifications:

> "The operating speed of the unit shall be such that the existing pavement is reduced into particles ranging from sand sized to pieces not exceeding 150 mm (6 in) in largest dimension, the majority being a nominal 25 to 50 mm (1 to 2 in.) in size. The surface concrete above any reinforcement found shall be reduced to the 25 to 50 mm (1 to 2 in.) size to the extent possible. Additional passes of the

breaker may be required if larger sizes remain above the reinforcement." (Work Proposal)

Prior to rubblization, however, an addendum was issued requiring that the pavement be rubblized as follows:

"The operating speed of the unit shall be such that the existing pavement is reduced into particles ranging from sand to pieces not exceeding 225 mm (9 in.) in largest dimension, the majority being a nominal 25 to 50 mm (1 to 2 in.) in size."

Rubblized material was to be compacted with two passes of a vibratory steel wheel roller with a nominal 9000 kg (10 ton) weight operating at a speed not to exceed 1.8 m/s (6ft/s). Pavement areas over existing culverts were to be removed and replaced with crushed aggregate.

4.1.2 Pre-Rubblization Pavement Evaluation

An evaluation of distresses was performed for the entire control section and selected sections of the rubblized project. Distresses were typically mid-slab fractures, polishing, and joint failure. Pavement Distress Index values for the control and rubblized sections were 68 and 72, respectively, on a scale where 0 is excellent and 100 indicates poor pavements.

Falling weight deflectometer evaluation was performed on the pavement prior to rubblization. A KUAB 2-mass Falling Weight Deflectometer was used for the testing. This testing indicated an average modulus value for the concrete (E_c) of 30,061 MPa (4.36 x 10^6 psi), with values ranging from 15,858 to 86185 MPa (2.3 x 10^6 to 12.5 x 10^6 psi). These values indicate a concrete of reasonable structural capacity. This testing also indicated a base layer mean modulus value of 227.7 MPa (32.3 x 10^3 psi), with values ranging from 122.8 to 439.9 MPa (17.8 x 10^3 to 63.8 x 10^3 psi).

4.1.3 Pavement Treatment

The pavement was rubblized using a multiple head breaker (MHB) developed by Badger State Highway Equipment, Inc., Antigo, Wisconsin. The MHB utilized two 2.4 m (8 ft.) wide rows of six breaking heads, with each head weighing 500 kg (1100 lbs). A bar 25mm (1 in.) wide by 200 mm (8 in.) long, mounted transversely on the bottom of each head did the actual breaking. Drop height used on this project was 1.1 m (44 in.). The MHB operated at a speed of 2.4 m/min (8 ft/min), with heads striking the pavement every 100 mm (4 in.). The purpose of the first row of heads was to provide initial fracture of the pavement, and the second row was designed to complete the pavement fracture. Following rubblization a 10,886-kg (12-ton) vibratory roller was used to seat the fractured pieces. The steel drum of the roller was fitted with a series of 25 mm (1 in.) wide bars arranged in a herringbone pattern to assist in completing the breaking process. Figure 4-1 shows the texture of the rubblized pavement both before and after rolling.

Figure 4-1. Rubblized Pavement Texture.

Rubblization began at the east end of the test section, and proceeded to the west for 1.64 km (5400 ft.). At that point (station 76 +00), the production rate dropped from 418 m^2/hr (500 sy/hr) to approximately 167 m^2/hr (200 sy/hr), and sampling indicated pieces exceeded specified top size. A test trench at that location also showed that the concrete slab rested directly on rock. The rubblization procedure was changed to add a preliminary fracturing of the pavement with a guillotine drop hammer, to sever any possible bonding of the slab to the underlying rock. The guillotine breaker dropped a 5,443 kg (6 ton), 2.4 m (8 ft.) wide hammer from a height of 1.5 m (5 ft.) to impact the pavement every 0.3 m (1 ft.). This procedure produced satisfactory production rates and rubblized particle sizes, and was continued to the end of the project.

Asphalt patches could not be rubblized, due to their absorption of the impacts, and hence, were removed and replaced with crushed aggregate. The pavement over culverts was not rubblized because of the potential damage to the culverts. The cover over the culverts was less than 1.2 m (4 ft.). These sections were removed by a backhoe and replaced with crushed aggregate.

4.1.4 Post Rubblization Evaluation

Following rubblization, a surface visual evaluation and a falling weight deflectometer (FWD) evaluation were performed on the rubblized pavement. Additionally, two trenches were dug to evaluate the breaking effectiveness through the entire depth of pavement. Visual and full depth evaluation of the trenches indicated that the rubblized pavement was broken into pieces with a maximum size of 300 mm (12 in.), ranging to a minimum of sand sized particles less than 4.75 mm (0.19 in.). The percent of pieces above the specification size of 225 mm (9 in.) was approximately 20 percent. The majority of the oversized pieces were located near the bottom of the rubblized pavement, and were found primarily in the section where the rubblization rate slowed, prior to the point where the pre-breaking technique was modified to include preliminary fracturing. Despite the large sized pieces, visual examination revealed that the entire pavement thickness was completely fractured. Centerline reinforcement was debonded from the concrete, and remained below the surface, making removal unnecessary.

FWD testing of the compacted rubblized pavement indicated pavement modulus values of an average of 728 MPa (105.6 x 10^3 psi) (a 97% reduction in modulus after fracture and compaction). High values were 1,444.5 MPa (209.5 x 10^3 psi) and low values were 424.7 MPa (61.6 x 10^3 psi). Using results of rubblization evaluation performed by Witcak and Rada[19], these values fall below what was identified as a critical value of 6895 Mpa (1000 x 10^3 psi), indicating complete fracture of the concrete.

The removed asphalt patch sections and the culvert sections were replaced with crushed aggregate. Local traffic was allowed to operate on the rubblized pavement. Water was applied daily for dust control.

4.1.5 <u>Surfacing Activities</u>

Following rubblization and replacement of culvert and patch sections, a nominal 75 mm (3 in.) thick layer of crushed aggregate was placed on the rubblized material to establish proper transverse slope. This material was compacted and paved with 75 mm (3 in.) of asphaltic concrete. Prior to paving, the aggregate surface was evaluated by FWD testing. The average modulus value was 541.2 MPa (78.5 x 10^3 psi) with a range from 370.9 to 1,827.1 MPa (53.8 x 10^3 to 265 x 10^3 psi). The Control Section was also overlaid with 75 mm (3 in.) of asphaltic concrete.

4.1.6 <u>Short-term Performance of Project</u>

A pavement distress evaluation of STH 16 was performed by the Principal Investigators on July 7, 1997, approximately 11 months following paving. In the control section, 88.6% of the transverse cracks and joints and 95.7% of the centerline longitudinal joint had reflected through the asphalt overlay. The location of these cracks and joints had been mapped prior to the paving in order to ensure that cracks appearing in the overlay were, in fact, reflective cracks. Additionally, edge cracks were evident for 100% of the control section where the overlay extended 0.9 m (3 ft.) past the slab edge. All cracks were less than 3 mm (1/8 in.) in width, and were classified as low severity level.

In the rubblized section, there were no reflection cracks on the entire length. There was only one transverse crack of low severity 2.7 m (9 ft) long in the entire section. This crack appeared in one of the mapped sections of the rubblized section, and was verified as not being a reflection crack. This crack was in a fill section approximately 6.1 m (20 ft.) high. No distresses had appeared at culvert locations where the pavement was removed and backfilled with crushed aggregate base course.

A second pavement distress evaluation on March 19, 1999, approximately two and one half years following rubblization, identified 96.4% of the transverse cracks and 98.2% of the centerline cracks had reflected through the asphalt overlay in the Control Section. Only nine low severity transverse cracks approximately 3.1 m (10 ft.) long were found in the entire length of the rubblized section.

This performance indicates that the rubblization has successfully eliminated early reflection cracking, although longer term monitoring is required in order to determine how long a period rubblization effectively stops reflective cracking.

4.2 US Highway 8

US Highway 8, near Brantwood in Price County, was fractured in 1993. This project included several different pavement treatment sections, each between 0.61 and 0.76 km (2000 and 2500 ft.) in length, except for the crack and seat section, which was 0.2 km (650 ft.) long. Test Section A consisted of cracking and seating, followed by a 75-mm (3-in.) overlay . Test Section 1 consisted of rubblizing and a 100-mm (4-in.) overlay. Test Sections 2 and 3 consisted of crack and joint cleaning and filling, followed by 75 mm (3 in.) and 37.5 mm (1.5 in.) overlays, respectively. Test Sections 4 and 5 consisted of cleaning and filling, followed by application of a waterproof membrane prior to paving with 37.5 mm (1.5 in.) and 75 mm (3 in.) overlays, and Test Section 6 involved no treatment prior to paving with a 37.5-mm (1.5-in.) overlay.

The existing pavement consisted of a nominal 200-225 mm (8-9-in) thick, 9.2 m (30 ft.) wide JPCP pavement built in 1983, over a previously cracked and seated 250-mm (10-in.) thick, 6.1 m (20 ft.) wide JPCP pavement built in 1940. Section A's cracking and seating was performed with a 5,443- kg (6-ton) guillotine drop hammer. Section 1's rubblization consisted of pre-fracture with the guillotine drop hammer, and rubblizing with the PB4 resonant pavement breaker. The pre-fracture was required due to the bonding of the upper layer of concrete pavement to the overlaid layer. Compaction was performed with four to five passes of a 10,886-kg (12 ton) vibratory steel-wheel roller with a herringbone pattern grid on the drum. Pavement over existing culverts was rubblized. A 100-mm (4-in.) overlay was placed over the test sections.

Evaluation of the rubblized pavement through digging of two test holes showed that the entire 225 mm (9 in.) upper layer was pulverized full depth, and hairline cracking was evident in the lower layer.

Distress evaluation of the test sections after one year indicated that the crack and seat and the rubblized sections were performing better than the other sections in reducing reflective cracking (PDI of 7 vs. PDI's of 13 to 28 for non-fractured sections). After the second year, the crack filled test sections began to be indistinguishable from each other as far as amount of reflection cracking , regardless of overlay thickness or membrane treatment (PDI's of 19 to 28), while the fractured slabs continued to perform well (PDI's of 13). The section without any treatment, as expected, performed the worst (PDI of 49). A distress evaluation performed five and a half years after construction showed that the rubblized section had a PDI slightly better than the other treated sections (50 vs. 54-56), yet still better than the control section (PDI of 64).

4.3 State Highway 73

State Highway 73, from Columbus to USH 151 in Dodge County, was rubblized in 1993. The pavement was a nominal 225 mm (9 in) JPCP with 24 m (80 ft.) dowel spacing, on 250 mm (10 in) of crushed aggregate base course, originally constructed in 1955. The underlying subgrade material ranged from an A-2-4 soil to an A-6 soil. The pavement had transverse cracks at approximately 4.6 meter (15 ft.) spacing, and a PDI of 57. The project consisted of a 0.7 km (2320 ft.) long control section that was patched with 0.6 m (2 ft) wide non-doweled patches, and a 1 km (3240 ft.) long test section that was rubblized using a PB4 resonant pavement breaker. The rubblization produced pieces 25 to 75 mm (1 to 3 in.) in diameter in the top 75 mm (3 in.), and pieces ranging from 75 to 125 mm (3 to 5 in.) in the remainder of the slab. The rubblized pavement was compacted with one pass of a 9,072-kg (10-ton) vibratory compactor, one pass of a pneumatic-tired roller, and two additional passes of the vibratory roller. A 50 mm (2 in.) leveling course of crushed aggregate was placed over the rubblized pavement, and compacted. This leveling course was used to seal the pavement structure from rain, allow its use by traffic, and correct the existing substandard cross slope to the standard value of 2%. This was necessary since paving was scheduled for two weeks after rubblization. Due to mix design and plant production problems, the road remained open to traffic for one month between rubblization and paving with 100 mm (4 in.) of asphaltic concrete.

Performance of the sections showed PDI values after one year of 0 for the rubblized section, and 13 for the control section. After two years, the PDI's increased to 7 for the rubblized section, and 27 for the control section. After five and one-half years, PDI's were 20 for the rubblized section and 37 for the control section. The distresses consisted of transverse cracks every 6 to 12 meters (20 to 40 ft.), and longitudinal cracking of the centerline.

4.4 County Highway TT

County Highway TT, east of Madison in Dane County, was rubblized in 1993. The project was 9.2 km (5.7 miles) long, from STH 30 to CTH N. This pavement was a JPCP with a nominal thickness of 200 - 225 mm (8 - 9 in.), after the 50-mm (2-in.)

asphaltic surface layer, placed in 1975, was milled off of the pavement. This pavement, built in 1946, was placed directly on a compacted A-6 subgrade. High moisture contents were evident in the adjacent fields, and in the shoulder. At the time of the rubblization, the pavement was severely distressed at the underlying concrete joints.

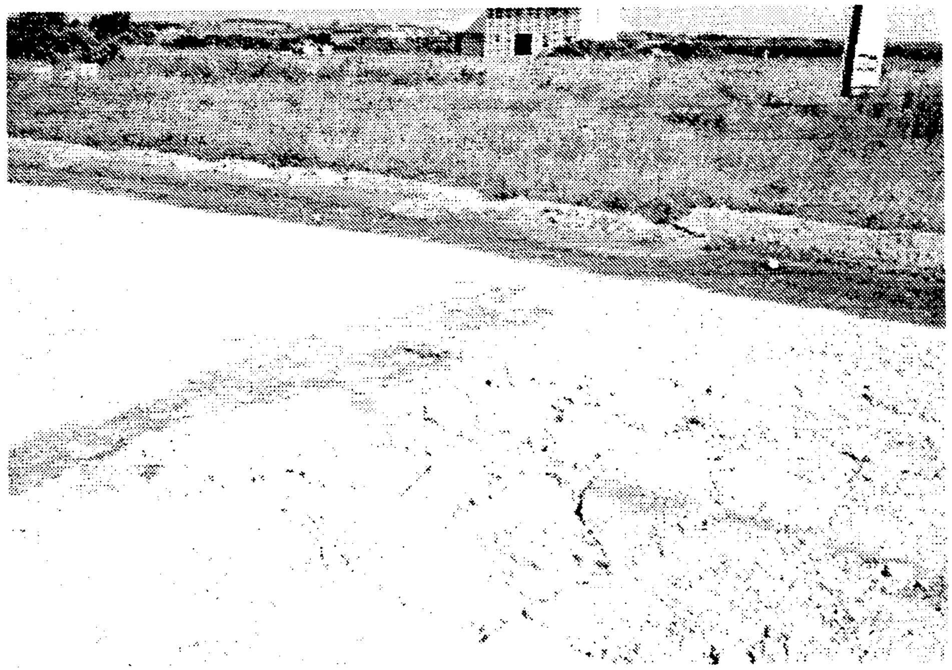

Figure 4-2. Inadequate Rubblization Over Center of Pavement.

This project was rubblized with the PB4 resonant pavement breaker, and compacted with a 10,886-kg (12-ton) vibratory roller with a herringbone pattern grid on the drum. Rubblization operations on the outside 1.8 m (6 ft.) of each lane progressed well, but the middle 3.6 m (12 ft.) resulted in oversize pieces of 0.3 to 0.6 m (1 to 2 ft.) in diameter, that were shoved at an angle into the underlying soft material, as can be seen in Figure 4-2. During rubblization of the middle section, additional moisture came to the surface. Due to surface irregularities caused by the shoved pieces, these areas were removed. A 75-mm (3-in.) thick crushed aggregate leveling course had to be placed over the rubblized pavement prior to paving. This layer was paved with a 125-mm (5-in.) asphaltic concrete surface. During the first year after rubblization, several sections showed thermal cracking. This section of CTH TT was crackfilled and surfaced with a chip seal in 1998, making tracking of reflection cracks difficult at this time.

4.5 <u>USH 51</u>

USH 51 north of Beloit was fractured in1992. The original pavement was 225 mm (9 in.) of PCC over 125 mm (5 in.) of crushed aggregate base course, over 225 mm (9 in.) of granular subbase. Test sections ranging in length from 0.9 km (2950 ft.) to 1.6 km (5150 ft.) included rubblization with a guillotine drop hammer, rubblization with the

PB4 resonant breaker, crack and seat, and concrete patching. Soils along the project length were firm, offering a good working platform. A leveling course of crushed aggregate 37.5 mm (1.5 in.) thick was added over the section rubblized by the drop hammer, to fill valleys and provide a smoother paving surface. A 37.5-mm (1.5 in.) leveling layer of asphaltic pavement was added throughout the fractured slab sections, followed by 100 mm (4 in.) of asphaltic concrete for a total thickness of 137.5 mm (5.5 in.). A 100-mm (4-in.) thick asphaltic concrete overlay was placed over the control (patched) section.

Rubblization proceeded well with the PB4. The guillotine drop-hammer, however, when used for rubblization, displaced pieces vertically and unseated the pieces. A later section with curb and gutter, originally planned for drop-hammer rubblization, was replaced by the PB4, due to this poor performance and the desire to ensure minimum disturbance of the curb and gutter. Following rubblization, the project manager reported some softness at the shoulder in approximately 25 locations. It also appeared that the asphalt binder was sliding or pushing from traffic loads due to this soft shoulder. After rubblizing, there was also a slight dip at the old joint locations in the concrete pavement, which showed up after each lift of binder. The project manager recommended that a 75-mm (3-in.) lift of aggregate base may be required for future rubblization projects.

Distress evaluation of the sections for the first year showed better performance for the sections cracked and seated or rubblized with the PB4 (PDI's of 0), than the section rubblized with the guillotine (PDI of 7). The section rubblized with the guillotine showed the same PDI as the section with only concrete patching. After two years, the PDI values were the same for sections (PDI of 16). A 1999 evaluation showed all sections performing comparably, with PDI values of 19 to 21.

5.0 Conclusions and Recommendations

5.1 Conclusions

Slab fracture techniques, especially crack and seat and rubblization, have in recent years gained widespread recognition among pavement engineers as effective means for controlling reflective cracking in HMA overlays over PCC pavements. The main objective of fracturing the existing deteriorated PCC is to produce slab pieces smaller than the original slab length, thus reducing movement due to temperature changes and load transfer rocking. The dimension of the pieces must also be such that structural capacity is maintained in the existing pavement system. The fractured pavement is rolled or compacted to eliminate rocking of the fractured pieces and any other movements, reestablish structural interlock, and establish firm contact with the underlying layer.

The ultimate objective of this study was to examine and evaluate the use and performance of slab fracture techniques (specifically, crack and seat and rubblization) in Wisconsin, and to develop procedures and guidelines for their use. This objective was accomplished by examining the state-of-the-art methods for slab fracture techniques, developing a database of fractured pavements in Wisconsin, evaluating the performance of in-service fractured pavements from the database, and monitoring the construction procedures and equipment performance for some fractured pavement projects in Wisconsin and other states which have used these techniques.

Based on the results of this study, the following conclusions are reached :

- Pavements located on subgrades classified as A-6 or A-7 with high moisture contents, as well as those located on good soils,but with saturated moisture conditions, do not satisfactorily rubblize. Such soil conditions do not provide adequate support against the breaking force of the rubblizer head/shoe, and create oversize fractured slab pieces which tend to shove into the underlying layers during the fracture operation. Such pavements, in addition, tend to exhibit short term poor performance after project completion. Moisture contents that define this condition have not been well documented in past projects or studies, however.

- If the pavement slab is bonded to an underlying layer, such as rock, an underlying PCC slab, or cement treated base, and is to be rubblized, pre-breaking of the surface with a guillotine drop hammer may be required to debond the concrete pavement from the underlying layer to facilitate the rubblization operation and to preclude movement of the fractured layer with the non-fractured layer.

- The PB4 resonant pavement breaker and the multiple-head breaker (MHB) are effective tools for rubblizing PCC pavements, while the guillotine drop hammer is the preferred tool for crack and seat projects.

- The short term performance (up to seven years) of in-service fractured PCC pavements in Wisconsin indicates that the rubblization technique has the best potential to minimize the effects of reflective cracking in AC overlaid PCC pavements.

- Service life values for crack and seat pavements can be estimated for use in a life cycle cost analysis involving pavement options. The performance results of this study indicate that for in-service cracked and seated JPCP, the service-life (based on IRI or PSI as the indicator of performance) is dependent on the HMA overlay thickness, as well as the pavement foundation condition measured in terms of the soil support value.

- For cracked and seated JRCP, the results show that soil support value alone may be adequate for estimating service life, and that overlay thickness does not have any significant effect on the service life. This observation suggests that the support provided by a cracked and seated JRCP appears to be more stable compared to that provided by a cracked and seated JPCP; and hence, pavement foundation movement contribution to surface ride quality will be minimal for the JRCP compared to the JPCP. This may require thick overlays to mask the effect of foundation movements in order to improve ride quality, and thus extend the overall service life. The stability provided by the JRCP support may be due to some of the reinforcement acting as a strengthening fabric in the fractured pavement layer.

- In this study, three performance indicators, including PSI, IRI, and PDI, were used. The results show that significant differences exist in predicted service lives, depending on the indicator used as the performance measure, as well as the threshold values adopted.

- A database has been developed for 158 fractured slab projects in Wisconsin to allow pertinent information about them to be queried.

5.2 Recommendations

- Weak subbases continue to have a profound negative impact on the construction and performance of fractured pavements, but have not been adequately addressed. The need exists to define clearly the types, moisture content range, and strength characteristics suitable for design and construction of fractured pavements.

- Visual examination of maximum nominal particle size, for the purpose of determining the extent or degree of pavement fracture in a rubblization project tends to be very subjective, yet effective, if the full depth of rubblized pavement is examined. Testing with an FWD could be performed in addition to the visual examination if an objective measurement (a modulus value) is required to minimize the level of subjectivity in this assessment. FWD testing may not be practical from a production viewpoint.

- The need exists to continue to monitor the fractured slab pavements to acquire data for the rubblized pavements to aid in evaluating their in-service long-term performance, and also update the database.

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Appendix A - Database Development for Fractured Slab Pavements

Database Development for Fractured Slab Pavements

To enable WisDOT to have the capability to maintain, update, and evaluate the use of slab fracture techniques, a database was set up for 158 fractured slab projects (19 rubblized and 139 crack and seat projects) that have been performed in Wisconsin since the early 1980s. These projects were identified by contacting the following sources: Mr. Matt Shinners of Antigo Construction Inc., the Pavement Management Unit of the WisDOT Bureau of Highway Construction, and WisDOT district offices.

The database has been created in *Microsoft Access*, and allows the user to maintain and update information in three main areas regarding fractured slab pavements. In addition, it allows the user to perform queries pertaining to specific details about any project. The three main areas included in the database are described as follows:

a) Original PCC Pavement Description and Construction Details.

i) General information for the original pavement: includes the project identification number, route name, nearest city, county, district, beginning and ending reference points, sequence numbers, total length, PCC pavement type, slab thickness, width, reinforcement type, joint spacing, and year of original construction.
ii) Original PCC rehabilitation information: includes the overlay type (if any), the year of the last overlay, and the overlay thickness.
iii) Unbound layer information for the existing pavement: includes the subgrade type, subgrade condition defined in terms of soil support value, base/subbase type and thickness.
iv) Design and Construction data: include the construction year average daily traffic (ADT), the design year ADT, percent trucks, design year, design life, design equivalent 80 kN (18kip) single axle loads (ESAL), design method, overlay thickness, thickness of untreated aggregate over fractured pavement (if any), overlay width, total overlay length, name of contractor, project completion date, fracture technique used, fractured slab size achieved, fracture and compaction equipment characteristics such as equipment type, drop height/speed, weight, and production rate. Cost information is also included.

b) Post Construction Distress and PDI Data.

These provide information on periodic surveys regarding the severity and extent of all distresses associated with each fractured slab project. The distresses include cracking of various forms, rutting, raveling, patching, and surface distortions. The database allows the user to record the distresses in accordance with procedures outlined in the WisDOT pavement distress manual. The pavement distress index (PDI) for each project is also provided for each survey year.

c) *Post Construction IRI Data*.

This provides information regarding the ride quality of each fractured slab project for every year that the ride is measured. The ride is defined in terms of the international roughness index (IRI) value.

Database Queries

Specific information regarding a project or set of projects may be quickly obtained using the queries component of the *MicroSoft Access* database. Sample queries that can be sought through the database include:

> Where are all the rubblized projects in the state?
> How many of them are in District 1?
> What type of PCC pavements were they prior to fracture?
> Who were the contractors for these projects?
> What type of equipment was used?
> What are the pavement's PDI or IRI history etc.?

An electronic copy of the database can be obtained from WisDOT, Division of Transportation Infrastructure Development—Pavement Research Unit.

Appendix B - Draft Slab Fracture Guide Specifications

Draft Slab Fracture Guide Specifications

Crack and seat and rubblization are viable techniques for rehabilitation and/or reconstruction of portland cement concrete pavements. Guidelines for their use are summarized in Table B-1 below and detailed in the sections that follow.

Table B-1. Summary of Guidelines for use in Crack and Seat and Rubblization Projects

	TECHNIQUE	
	CRACK AND SEAT	RUBBLIZATION
Candidate Pavements	JPCP	JPCP, JRCP, CRCP
Typical Distresses	low to moderate faulting, spalling, joint patches, corner breaks, uneven/rocking slabs, lane separation, D-cracking	punch outs, delamination, patch deterioration, transverse/longitudinal cracking, low to moderate faulting, D-cracking
Conditions to Avoid	Must have aggregate base	Moist A-6, A-7 soils, high water tables or saturated base/subgrade
Fracture Equipment	Guillotine drop hammer	Resonant Breaker, Multi-Head Breaker,
Compaction	5 passes of 31,175 kg. (35 ton) pneumatic roller	3 passes of 9000 kg (10 ton) vibratory steel wheel roller
Resulting particle size	0.6 - 0.9 m (2 - 3 ft) spacing	No larger than 225 mm (9 in) Majority between 25 mm and 75 mm (1 to 3 in)

B.1 Crack and Seat

A. *Conditions for Use*: The crack and seat technique shall be used only on structurally sound jointed plain concrete pavements exhibiting low to moderate severity levels of distress. Typical distresses include but are not limited to low to moderately faulted joints and cracks, spalling, joint patches, corner breaks, rocking slabs, uneven slab settlement, lane separation, and D-cracking. These correspond to WisDOT pavement surface distresses of:

- Slab Breakup
- Distressed Joints/Cracks
- Surface Distress
- Longitudinal Joint Distress
- Transverse Faulting.

Pavements exhibiting D-cracking may be cracked and seated if the areas of D-cracking are first blown out with 690 kPa (100 psi) compressed air, and breaking operations should cease approximately 0.6 m (2 ft.) from the D-cracked area. Pavements without crushed aggregate base courses in good condition shall not be cracked and seated.

B. *Pre-construction Activities*: Prior to the start of the crack and seat project, the following activities shall be conducted using procedures outlined by WisDOT:

i) Detailed distress and structural surveys shall be conducted to document the existing surface and structural conditions of the pavement;
ii) Distress conditions such as severe joint spalls may be repaired by full-depth or partial depth methods depending on the severity and extent of the distress;
iii) Asphalt patches and deteriorated sealants (if any) shall be removed to the satisfaction of the engineer.

C. *Cracking and Seating Operations*: Cracking and seating shall be performed by a guillotine drop hammer capable of fracturing the entire thickness of the pavement slab with hairline cracks. A desired cracking spacing of approximately 0.6 to 0.9 m (2 to 3 ft) is to be achieved but acceptance shall be at the discretion of the engineer. Prior to the start of routine cracking operation, a test section of sufficient length shall be established to demonstrate to the engineer that the equipment's breaking force and striking pattern will produce acceptable crack patterns. If conditions change or an unacceptable pattern is being produced, the engineer should adjust the striking energy and/or the striking pattern. Acceptable cracking spacing should be checked by applying water to the pavement surface immediately before it is cracked.

Following cracking, the pavement shall be seated with three to seven passes of a pneumatic tired roller in the load range of 31,750 to 45,360 kg (35 to 50 tons), or a 9,070-kg (10-ton) vibratory compactor.

Any soft areas detected by the rolling/seating procedure shall be undercut and removed as directed by the engineer. Such areas shall be backfilled and compacted with approved untreated granular aggregate. Following compaction, a falling weight deflectometer (FWD) test shall be conducted to determine the structural capacity of the fractured and seated pavement. The pavement shall be cleared of all loose debris prior to overlay.

B.2 **Rubblization**

A. *Conditions for use*: Rubblization shall be applied to any PCC pavement type having little or no potential to retain its integrity or structural capacity. Such pavements may be characterized by severe levels of distresses such as punch outs, delaminations, patching deterioration, wide transverse and multiple longitudinal cracking. Pavements with D-cracking may be rubblized if actions outlined in section B.1A. are performed. Pavements with low or moderate levels of transverse faulting are also suitable candidates for rubblization, if the faulting is ground one or two times before rubblizing. These distresses correspond to WisDOT pavement surface distresses of:

- Slab Breakup
- Distressed Joints/Cracks
- Surface Distress
- Longitudinal Joint Distress
- Transverse Faulting.

In addition, the pavement shall have a base course or be built on a coarse-grained subgrade. If constructed immediately above a moist, fine-grained subgrade, rubblization may not fracture the slabs to desired size particles, and may shove pieces into the subgrade. Subgrades that have been identified as being unsuitable for rubblization include AASHTO classes A-6 and A-7. Pavements showing evidence of high water tables or saturated subgrades/subbases should also be avoided. Standing water in adjacent ditches or field, wet shoulders, or the ejection of water during rubblization are indicators of these unsuitable conditions.

B. *Pre-construction Activities*: Prior to the start of the rubblization project, the following activities shall be conducted using procedures outlined by WisDOT:

i) Detail distress and structural surveys shall be conducted to document the existing surface and structural conditions of the pavement;
ii) Any asphalt patches, loose joint fillers (if any) shall be removed and replaced with approved untreated aggregate.
iii) Culverts should be delineated to allow the contractor to decide whether they should be rubblized or removed and replaced with crushed aggregate.

C. *Rubblization and Compaction Operations*: Rubblization shall be performed using either a PB4 resonant pavement breaker or a multi-head breaker (MHB). The PB4 should be capable of producing low amplitude 900 kg (2000 lb.) force blows at a rate of not less than 40 cycles per second. Approximately fifteen passes will be required to cover a 3.6-m (12-ft.) traffic lane for a PB4 equipped with a 300 mm (12 in.) square breaker shoe, ans 20 to 24 passes are required for the 175-mm (7-in.) breaker shoe. The MHB should be capable of producing a breaking energy between 2740 and 9590 J (2000 and 7000 foot-pounds). A test section of sufficient length shall be used to determine the energy and rubblizing pattern that will produce acceptable results. Rubblized pieces should be no larger than 225 mm (9 in.) in size, with the majority of the pieces being between 25 and 75 mm (1 and 3 in.) in size. If reinforced concrete pavement is being rubblized, all pieces above the reinforcement shall be less than 75 mm (3 inches) in size. Any large pieces of rubblized pavement or patches that may be detected on the rubblized pavement surface shall be removed and the resulting depression filled with approved filler aggregate. Steel shall be debonded from the concrete and left in place. However, any steel exposed at the surface shall be cut off below the surface and removed.

In areas where portland cement concrete pavements or cement treated bases lie under the pavement to be rubblized, or where the pavement is located over a strong subgrade or rock layer, pre-breaking of the surface with a guillotine drop hammer may be required to debond the concrete pavement from the underlying layer. Any other conditions that result in unacceptable rubblizing should require adjustments to the energy level and/or breaking pattern by the engineer.

Pavement at culvert locations may be removed and replaced with approved untreated crushed aggregate, at the discretion of the contractor. Testing of required cover over culverts has not been conducted, but projects have successfully rubblized pavements with a minimum of 0.6 m (2 ft) between the bottom of the slab and the top of the culvert.

Following rubblization, the surface shall be compacted with not less than three passes of a 9,000-kg. (10-ton) vibratory steel wheeled roller. A pass shall be defined as forward and back in the same path. In order to enhance the breaking of surface pieces, the drum of the roller may be equipped with a raised "Z" or herringbone pattern grid approximately 25 mm (1 in.) high.

If the project engineer is unsure of the success of the fracture, an FWD test may be conducted after rubblizing and compaction to determine whether the elastic modulus is lower or higher than the 6890 Mpa (1000 x 10^3 psi) value used to predict satisfactory rubblization.

Limited traffic may be allowed on the rubblized pavement prior to surfacing operations. A layer of crushed aggregate may be placed over the rubblized pavement prior to surfacing operations to correct non-uniformities in the surface or to provide a satisfactory cross slope. Water may be applied to the surface for dust control.