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Investigation of Early Distress in Wisconsin Rubblized Pavements

FINAL REPORT

July 2011

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Research Study # WI-10-02

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16. Abstract

This study investigated premature distress formation in Wisconsin HMA overlays of rubblized concrete pavement. The premature distress was tented transverse cracking, which formed during the winter season and significantly affected pavement ride. The intent of this study was to determine the mechanism behind the failure and recommend changes in design and construction of rubblized pavements to prevent future failures. Design parameters, soil properties, historic distress levels, and several additional factors were analyzed for 19 good- and poor-performing pavements. Service life and construction practices were also evaluated.

Tented transverse cracking was a result of non-uniform support of the rubblized layer. Weak support occurred at previous locations of deteriorated joints and cracks in the concrete pavement, causing a reflective crack in the HMA layer. Water entered the crack, froze and expanded in the winter, and caused tenting of the crack. It is recommended that deteriorated joints be repaired with base patching to create a more uniform pavement foundation. Test rolling of the rubblized layer was also suggested to locate additional weak areas. It is recommended that these weak areas be excavated below subgrade and backfilled with aggregate material.

Using predictive models based on pavement distress information, the service life of the 11 pavements in the good performance category was forecast to be 17 to 25 years. The eight pavements in the poor performance category were projected to have a service life of 13 to 15 years.

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Commonly Used Abbreviations

Initializations

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
BAD	Base aggregate dense
CABC	Crushed aggregate base course
CRCP	Continuously reinforced concrete pavement
CTH	County trunk highway
DOT	Department of Transportation
EBS	Excavation below subgrade
FDM	Facilities Development Manual
FWD	Falling weight deflectometer
HMA	Hot mix asphalt
IRI	International roughness index
JPCP	Jointed plain concrete pavement
JRCP	Jointed reinforced concrete pavement
M-E	Mechanistic-empirical
MEPDG	Mechanistic-empirical pavement design guide
NB	Northbound
PCC	Portland cement concrete
PCI	Pavement condition index
PDI	Pavement distress index
PDR	Pavement design report
SB	Southbound
SMA	Stone matrix asphalt
SN	Structural number
STH	State trunk highway
STN	State trunk network
TSR	Transportation Synthesis Report
USH	United States highway
WisDOT	Wisconsin Department of Transportation

Units

ksi	kips per square inch
in	inch/inches
psi	pounds per square inch
m/km	meters per kilometer
ft	foot/feet
ft^2	square feet
CY	cubic yards

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1. Introduction

Rubblization, a practice that breaks existing concrete pavement full-depth, is commonly used in the State of Wisconsin as an alternative to full pavement reconstruction. The concrete pavement is broken in place and overlaid with new hot mix asphalt (HMA) or concrete pavement; HMA pavement is typically used in Wisconsin. The rubblized layer provides a strong pavement foundation. The pavement is broken into small pieces (2 to 12 inches), which ideally eliminates slab action and reflective cracking in the HMA surface. Various equipment is capable of pavement breaking; Wisconsin pavements are typically rubblized with a multi-head breaker (MHB).

Rubblization has been used in Wisconsin for over 20 years, and good performance has typically been observed. [*1, 2, 3*] Recently, however, some rubblized pavement systems have shown premature distresses that lead to early failure. The early distresses noted were tenting of transverse cracks in the HMA pavement layer. This distress has a significant effect on pavement smoothness and occasionally is severe enough to impact driver control.

The purpose of this study was to evaluate the pavements that have demonstrated this distress and investigate the root cause of the failure.

2. Literature Review

2.1 Performance Issues

A 2003 study performed for the Michigan DOT investigated the performance of over 80 rubblized pavement systems in the state. Many projects had good or excellent performance. Other under-performing projects had unacceptable levels of joint reflective cracks, top-down cracking, rutting and/or raveling. The study concluded that reflective cracking was a direct result of inadequate rubblization of the old concrete pavement. Top-down cracking, rutting and raveling were due to segregation and other issues related to the HMA mixture. It was determined that all distress types could have been prevented with better quality control during construction. [*4*]

Drainage of the rubblized material was also investigated on eight projects in the Michigan DOT study. Results were variable. Three projects had good or excellent drainage, and four had poor drainage in the rubblized layer. In the poor drainage projects, water ponded in the test trench that was excavated. It was therefore concluded that water seeping through a crack in the HMA layer would be left standing in the rubblized layer, which could lead to softening of the rubblized material and asphalt stripping in the HMA layer. [*4*]

A Transportation Synthesis Report (TSR) prepared for WisDOT surveyed states on rubblization performance and challenges during the rubblization process. Eighty-three percent of responding states noted good performance of rubblized pavements. Ohio found that rubblized pavement systems performed comparably to newly constructed HMA pavement. Michigan also noted good performance of rubblized pavement systems. When properly maintained, rubblized pavements in Michigan can be expected to have a service life greater than 20 years. The expected service life decreases to 15 years if routine maintenance is not performed. [*5*]

Despite the accounts of generally good performance, 75 percent of responding states reported that there are challenges associated with rubblization. Equipment issues, soft subgrades, and traffic-related problems during construction were the most commonly noted challenges. [*5*] These responses indicated that while good performance of rubblized pavements can be expected, the rubblization process must be carefully planned and executed.

A 2004 study for the Alabama DOT looked closely at distresses present in nine rubblized pavement sections. An average of eight transverse cracks per mile were recorded. The researchers reported that for a car traveling at 70 miles per hour, a transverse crack would be encountered on average every 6.4 seconds. It was also noted that the transverse cracks did not correspond to the original joint spacing and therefore were not a result of reflective cracking. The study also found that greater distresses were present in rubblized continuously reinforced concrete pavement (CRCP) and thick concrete pavements (greater than 10 inches). [*6*]

Rubblized pavement performance was evaluated for several projects constructed on Illinois Interstates in the 1990s. The control sections included HMA overlays of two 8-in CRCPs and of a patched 10-in jointed reinforced concrete pavement (JRCP). The test sections were rubblized and overlaid with HMA of varying thicknesses. Roughness data (International Roughness Index - IRI) and condition rating survey values were collected. The data indicated that the rubblized sections performed at approximately the same or better level overall than the control sections. The rubblized sections with thicker HMA had better ride values but similar cracking performance compared to the thinner HMA over rubblized concrete sections. [*7*]

A study conducted for the Colorado DOT investigated the performance of a rubblized pavement system with the following structure: 6 inches of HMA over 8 inches of rubblized jointed plain concrete pavement (JPCP) over 2 inches of asphalt-treated base. The pavement structure was a 20-year design for 6.5 million equivalent single axle loadings (ESALs). Falling weight deflectometer (FWD) testing prior to and immediately following rubblization found that load transfer at the concrete joints was reduced from a range of 83 to 95 percent to a 64 to 69 percent range. The subgrade resilient modulus was in the 16 to 20 ksi range. Two types of rubblization equipment were used on the project: MHB and resonant frequency breaker (RFB). Subgrade and pavement moduli were similar for both breaking methods. [*8*]

The pavement performance investigation found that three of the four 1000-ft test sections had zero to ten ft of transverse cracking; the fourth section (RFB) had 125 ft of transverse cracking. It was not noted whether these cracks occurred at locations of previous concrete joints. Limited longitudinal cracking was noted in the test sections and was determined to be top-down cracking. The cracking pattern and overall pavement moduli were similar for both the MHB and RFB breaking methods. [*8*]

2.2 Design and Construction Considerations

A 2007 study for the Wisconsin DOT provided guidelines for the design and construction of rubblized concrete pavements. Design recommendations included a minimum HMA thickness of four inches and a foundation layer elastic modulus of at least 10 ksi. For pavement design using AASHTO methods, a layer coefficient of 0.22 for the rubblized layer was suggested. It was noted that this value was conservative and could be increased (i.e., increase the rubblized layer's structural contribution) with further research. If M-E design principles were used, a 65 ksi rubblized layer modulus was recommended. However, it was noted that the "correct" value to use in design was highly site-specific. [*2*]

Service life for rubblized pavements was also estimated. With a pavement distress index (PDI) failure threshold of 75,[1] predicted service life was 21 years. Other studies of Wisconsin pavements reported lower service lives, but it is the author's opinion that overlay thicknesses have increased since the time of those studies, and that will contribute to longer service life. It was concluded that the 22-year design life currently used for rubblized pavement systems was appropriate. [*2*]

A nationwide overview of rubblization design and construction practices prepared in 2005 provided guidelines on pavement breaking and other construction considerations. The general suggestion for breaking of the concrete pavement was that the slab should be reduced to rubble such that reflective cracking is eliminated, but not so fractured that poor base support results. In the lower half of the slab, it was concluded that pieces 12 to 18 inches were acceptable and did not affect the likelihood of reflective cracking. It was specifically stated that the "…intent of rubblizing is to produce a structurally sound base which prevents reflective cracking by obliterating the existing pavement distresses and joints. The intent is not to meet a gradation requirement." [*9*]

It was recommended that, prior to rubblization, all loose material be removed from the pavement surface. Small, sound concrete and HMA patches could remain in place, but large, deteriorated HMA patches should be removed and base patched with HMA or aggregate. Underdrain systems were recommended, and it was noted that water would likely flow in the drainage system during rubblization. [*9*]

Another recent TSR prepared for WisDOT provided information on states' design of rubblized pavements. Fifteen states reported layer coefficients typically used in design, as shown in Table 1. Delaware and Ohio reported the lowest layer coefficients of 0.14, which is in the range typically used for aggregate base course layers. Ohio researchers determined this value by comparing the deflection of rubblized layers to the deflection of aggregate base layers. [*10*]

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[1] The PDI scale is zero to 100, with the zero value representing a pavement with no distress. The Department's PDI rating system is further described in Section 7.1.

Table 1. Nationwide Rubblized Layer Coefficients (Summarized from [*10*])

State	**Rubblized Layer Coefficient**
Delaware	0.14
Ohio	0.14
Michigan	0.18
Colorado	0.19
Maryland	0.20
Connecticut	0.20
Kansas	0.20
Maine	0.20
Wisconsin	0.20 to 0.24
Alabama	0.22
Florida	0.22
New Jersey	0.22 to 0.25
South Dakota	0.24
Louisiana	0.25
North Carolina	0.28
Arkansas	0.29

California and Illinois reported using rubblized pavement systems but use design methods that do not require a layer coefficient. California uses the Hveem design method and assumes the rubblized layer to be similar to a high quality granular base. Illinois uses a mechanistic-empirical design procedure with a design chart used to select HMA pavement thickness. [*10*]

The Michigan DOT study mentioned in the previous section also investigated design parameters of the state's rubblized pavements. Using data from FWD testing, pavement layer moduli were backcalculated for 18 of the study's test sites. Backcalculated modulus values for the rubblized concrete slab ranged from 46 to 1,064 ksi. This variation was due to differences in the original concrete pavement strength and in the rubblization process (e.g., MHB versus RFB). Using the modulus results, layer coefficients were estimated using AASHTO design methods. Calculated layer coefficients ranged from 0.06 to 0.45; the average layer coefficient for the rubblized concrete slab was 0.20. [*4*]

This study also examined the rubblized concrete pavement as two separate layers: one as completely broken material resembling CABC and one as fractured concrete pavement. Using AASHTO procedures, the designer could analyze the pavement system using these two layers rather than one rubblized pavement layer. If the two-layer approach was used, the study's recommended layer coefficients were 0.14 and 0.26 for the broken material and fractured concrete layers, respectively. Using these numbers produced a pavement design similar to the one-layer approach with a layer coefficient of 0.20. [*4*]

It was also concluded that: [*4*]

- Concrete pavements should not be rubblized if they have relatively weak support (3,000 psi or less), or if the existing pavement has "extensive" cracking.
- If, during rubblization, large pieces of concrete rotate and penetrate the base layer, all material to the roadbed soil should be excavated, replaced with aggregate, and compacted.

A report was prepared in 2006 for Antigo Construction, a Wisconsin-based company that performs a significant amount of rubblizing nationwide. This study evaluated performance of rubblized pavements in Michigan that used MHB and RFB rubblizing equipment. Most pavements rubblized with the RFB had excellent, good or fair ride quality. In the MHB category, all but one pavement had a good distress index (DI) rating and good or excellent ride quality. High distresses noted on the under-performing project were attributed to subgrade issues, including poor drainage. In addition, wet weather conditions during construction exposed the rubblized layer to moisture. [*11*]

For projects broken with the MHB, DI data were available up to age 6 years. These data were plotted against pavement age to create a pavement failure prediction model, with failure at DI = 50. The second-order polynomial model predicted failure at age 15.4. Data for fully drained and partially or non-drained pavements in the MHB were then analyzed separately. Second-order polynomial models for the fully drained and the partially or non-drained pavements predicted failure at 21 and 9.4 years, respectively. [*11*]

For projects broken with the RFB, DI data were available up to age 13 years. A second-order polynomial model predicted failure for these pavements at age 17.1. It was determined that rubblization equipment (MHB or RFB) was not a significant factor for distress prediction, and a predictive model using all performance data estimated the service life of Michigan's rubblized pavements to be 16.5 years. [*11*]

Low correlation values ($R^2 < 0.5$) were calculated for these predictive models. It was noted that this variability was expected given differences in construction materials and practices, underlying soil characteristics, and construction workmanship. [*11*]

2.3 State Specifications

Several states' rubblization specifications were reviewed. Many similarities exist among these and Wisconsin's provisions. For instance, test pits to determine rubblized particle sizes are required in all of the reviewed specifications. Most states do not allow traffic on the rubblized layer prior to HMA paving. The states have slightly different particle size requirements, with acceptable sizes ranging from 2 to 15 inches in the top half of the slab and 6 to 18 inches in the bottom of the slab. [*5, 12*]

Other notable provisions were encountered in the various states' specifications. In Michigan, it is required that existing concrete pavement joints and surface cracks with a width greater than ¼ inch be indistinguishable after rubblization, and no HMA patch material should be visible on the surface. [*13*]

The Oklahoma specification includes a provision that "if the pavement cannot be rubblized, the Department will pay for removal of pavement and replacement with Aggregate Base Type A …" [*14*]

Several states mandate that the rubblized layer be exposed for no more than 48 hours prior to HMA paving. These provisions include wording to allow the project engineer to extend this open period if rain events occur, to allow for drying of the rubblized layer. [*15, 16, 17*] The Oklahoma specification states that the first HMA layer be placed immediately following rubblization. [*14*]

In Illinois, design of HMA overlays over rubblized pavement is completed using a mechanistic-empirical approach. The HMA pavement thickness is determined based on traffic levels and location in the state. HMA thickness increases with design traffic and northern location in the state. A minimum HMA thickness of 6 inches is specified. In addition, the rubblizing method (e.g., MHB or RFB) is selected based on the thickness of the concrete pavement section to be rubblized and the immediate bearing value (IBV)[2] of the subgrade. Rubblization is not performed on concrete pavements less than 6 inches thick. These guidelines are currently being reviewed using updated testing of Illinois DOT pavements. [*12, 18*]

3. Study Description

3.1 Background and Objectives

This study was motivated when winter pavement ride on some rubblized pavement systems was significantly affected by transverse cracking in the HMA pavement layer. The transverse cracks exhibited tenting, a condition where the HMA pavement was shoved upwards on either side of the crack. Tenting typically subsided in the spring, but posted speed limits had to be temporarily reduced in some cases.

Other rubblized pavement systems did not develop this type of distress and exhibited adequate performance overall. Many examples of rubblized pavements with excellent performance were identified. These pavements had been in service for up to 12 years at the time of this study.

The primary objective of this research effort was to determine why some rubblized pavements failed in the manner described above while others had excellent performance. Pavement design parameters, construction practices and performance data were reviewed to evaluate the root cause of failure and make recommendations on changes that could be made to avoid this type of failure in the future. An additional objective was to evaluate current design parameters and pavement service life assumptions and determine their suitability.

[2] The IBV is a measure of soil strength similar to the test for California Bearing Ratio (CBR).

3.2 Scope of Study

The scope of this evaluation included analysis of project details for 19 rubblized pavements in Wisconsin. The following documents and databases were reviewed:

- Pavement design reports
- Soils investigation reports
- Project plans and as-built files
- Construction diaries
- Pavement distress data
- WisDOT rubblization guidelines and specifications

Notes and photographs from field surveys were also examined. Additional field work, such as pavement and materials testing, was not included in this study.

3.3 Methodology and Report Organization

<u>Project Identification</u>

A set of 19 rubblized pavement projects were identified for analysis. Projects with both good and poor performance were selected. Each of the five WisDOT regional pavement engineers subjectively identified pavements that had met performance expectations and some that were showing early signs of failure. Eleven projects with "good" performance and eight projects with "poor" performance were identified for analysis. Basic information for these 19 projects is outlined in Section 5.

<u>Information Collection</u>

Detailed project information was gathered for each of the 19 study projects. Pavement design reports, soils reports, and as-built construction plans were reviewed to gather information such as original concrete pavement structure, HMA pavement type and thickness, soils parameters, traffic information, and construction staging. In addition, construction diaries kept by project engineers in the field were reviewed. Noteworthy events during rubblizing and paving were recorded, along with any problems encountered in the field.

Information was collected on each pavement's condition history. WisDOT pavement information records were reviewed for distress (PDI) and roughness (IRI) data. These parameters were collected for both the existing concrete pavement and for the HMA pavement after rubblizing. Field reviews were conducted by WisDOT staff in 2011 for each pavement in the analysis.

Current WisDOT practices relating to the design and construction of rubblized pavements were reviewed. A summary of these guidelines and specifications is presented in Section 4.

Analysis

Three analyses were performed for the pavements in this study:

1. **Project-specific analysis.** Projects were reviewed individually. General project details and information gathered from construction plans and diaries, field reviews, and pavement distress data were used for this qualitative review of construction practices and pavement performance. Discussion for each project is presented in Section 6.
2. **Overall comparison.** Projects in the "poor" performance category were compared as a group to projects in the "good" performance category. Distress information and detailed project parameters were used to perform this quantitative comparison. This analysis was performed to determine if any project parameter(s) commonly led to good or poor rubblized pavement performance. This analysis is presented in Section 7.
3. **Design considerations.** Pavement service life and design input parameters were evaluated. This analysis is presented in Section 8.

Results and Conclusions

Discussion of results are presented at the end of each of the three analysis sections. Conclusions from the analyses are presented following the discussions and are repeated in Section 9 for ease of reference.

Recommendations

Recommendations are presented in Section 10.

4. Current WisDOT Practices

Guidance for selection and design of rubblized projects is provided in the WisDOT Facilities Development Manual (FDM). [*19*] Construction practices are outlined in the Department's Standard Specifications for Highway and Structure Construction and in the WisDOT Construction and Materials Manual (CMM). [*20, 21*]

4.1 Project Selection

Rubblization is to be considered as a pavement replacement strategy when the existing concrete pavement has no remaining service life. The current guidelines for when a concrete pavement has reached terminal serviceability are as follows:

- Greater than 20 percent of the roadway is in need of concrete joint repair
- Greater than 20 percent of the roadway surface has been patched
- Greater than 20 percent of the roadway has slab breakup or longitudinal joint distress greater than 4 inches in width
- The PDI is greater than 60

Rubblization may be used on all concrete pavement types (e.g., JRCP, JPCP and CRCP).

Installation of drainage features such as edge drains should be considered but is not required for rubblized systems. Guidance states that projects with fine-grained underlying soils are more likely to need drainage, while projects with coarse-grained soils typically do not. If used, drainage systems should be installed and functional prior to rubblization.

A subgrade investigation is recommended. FWD or DCP testing are proposed methods to determine existing support conditions, although it is noted that conditions may vary between time of testing and construction. A more detailed soils investigation is recommended if the underlying soils have a DGI greater than 12, or if the water table is less than 4 ft from the top of the subgrade.

4.2 Design

WisDOT pavement design utilizes the AASHTO method of computing a structural number from the sum of the products of individual layer coefficients and thicknesses. The layer coefficient for the rubblized layer is to be in the range of 0.20 to 0.24. A minimum HMA thickness of 4 inches is required from a constructability standpoint.

HMA overlays of concrete pavement must be removed prior to rubblizing. Thin layers of HMA left by the milling machine may remain in place. Good-condition full-depth HMA patches of concrete pavement need not be removed. Guidelines state that "full-depth concrete repair is not necessary."

Traffic staging must be planned such that the rubblized layer does not carry traffic. Traffic is allowed on intermediate HMA layers of "a sufficient HMA thickness." In this case, it is suggested that an undistributed amount of base and/or surface patching be included in the contract to correct any problems prior to paving of the final (surface) HMA layer.

4.3 Construction

The WisDOT standard specification requires that pavement be rubblized with self-contained, self-propelled breakers. All pavements in this study were broken using Antigo Construction's MHB Badger Breakers®. [*22*]

The concrete pavement must be broken uniformly, with all particles less than or equal to 12 inches as a maximum dimension. In addition, 75 percent of the broken particles must conform to the dimensions shown in Figure 1. The particle sizes must be verified by excavating two 9-ft^2 test holes during the first half day of construction with one test hole per lane-mile thereafter.

Exposed reinforcing steel must be cut below the rubblized surface and removed. In addition, the FDM states that for adequate performance, steel must be debonded in JRCP and CRCP.

The rubblized layer must be compacted with two passes of a vibratory steel roller. Loose material such as asphaltic patching and joint filler must be removed from the surface. Depressions should be filled with aggregate and re-compacted if necessary. Prior to paving, additional compaction passes with a pneumatic-tire roller and the vibratory steel roller are required.

The CMM notes that rubblization during light to medium rain is acceptable. However, a rubblized layer over a moisture-sensitive subgrade should not be exposed during rain events.

The specification allows for modifications deemed necessary by the project engineer, such as particle size, excavation of test holes, rolling and compaction. Little additional guidance is provided for problems encountered during rubblization, such as an area of weak support. The FDM proposes use of excavation below subgrade (EBS) or an increased HMA thickness to address localized areas of poor stability.

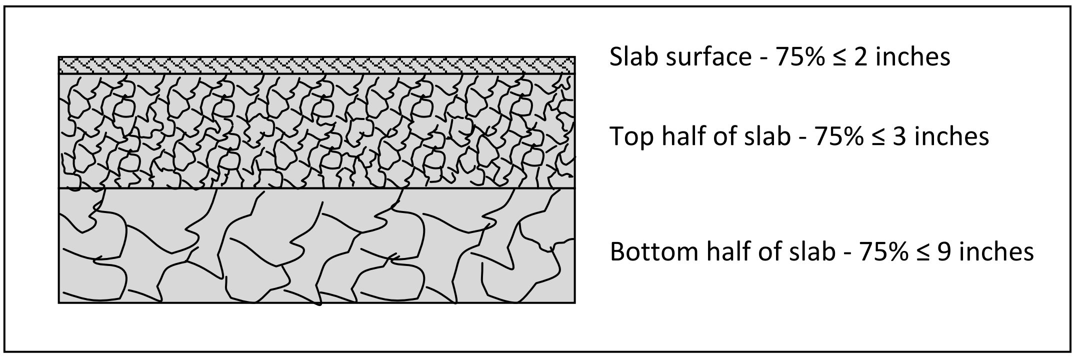

Figure 1. Rubblized particle size requirements. From [*3*]

5. Overview of Rubblized Pavement Projects

A total of 19 pavements were analyzed in this study. Eleven were in the "good" performance category, and eight were in the "poor" performance category. A map of the project locations is provided in Figure 2. Basic details, including project limits and construction year, are outlined in Tables 2 and 3 for good- and poor-performing projects, respectively. Projects marked with an asterisk had features that warranted analysis as two separate projects in the overall analysis (Section 7). These features will be described in more detail in Section 6.

Table 2. Rubblized Pavements - Good Performance Category

Study ID	Highway	WisDOT Project ID	County	Project Limits	Construction Year
1 Good	USH 12	1080-03-70	Walworth	Illinois state line – STH 50	1998
2 Good	STH 67	3160-00-70	Walworth	CTH B – W Main St	1999
3 Good	I-894	1090-14-70	Milwaukee	Belton RR OH – Mitchell IC	2003
4 Good	STH 42	4140-10-71	Door	STH 57 – CTH T	2001
5 Good	USH 41	1150-20-71	Oconto	Abrams – Octonto	2002
6 Good	STH 23	1440-14-71	Sheboygan	STH 32 – I-43	2003
*7 Good	USH 12	5300-04-74	Dane	Gammon Rd – Whitney Way	2005
*8 Good	STH 11	1700-04-72	Green	Hiltbrandt Rd – Airport Rd	2000
9 Good	USH 53 NB	1191-09-75	Chippewa	STH 64 – North county line	2005
10 Good	USH 53 SB	1191-09-76	Chippewa	STH 64 – North county line	2007
11 Good	USH 8	1595-10-70	Oneida	Rhinelander Beltline	2003

*Evaluated as two separate projects in the Overall Analysis

Table 3. Rubblized Pavements - Poor Performance Category

Study ID	Highway	WisDOT Project ID	County	Project Limits	Construction Year
1 Poor	CTH E	2090-13-70	Milwaukee	N 124th St – N 89th St	2009
2 Poor	STH 20	2440-03-70	Racine	West Blvd – Marquette St	2002
3 Poor	USH 41	1130-12-73	Brown	CTH F – CTH G	2002
4 Poor	I 43	1093-03-74	Rock	STH 140 – East county line	2007
5 Poor	STH 113	5420-02-71/72	Dane	Waunakee – Madison	2004
*6 Poor	USH 53	1191-09-73	Chippewa	40th Ave – CTH B	2006
7 Poor	USH 53	1199-11-71	Douglas	Kent Rd – USH 2	2002
8 Poor	USH 51 SB	1178-07-70	Lincoln	CTH S – USH 8	2006

*Evaluated as two separate projects in the Overall Analysis

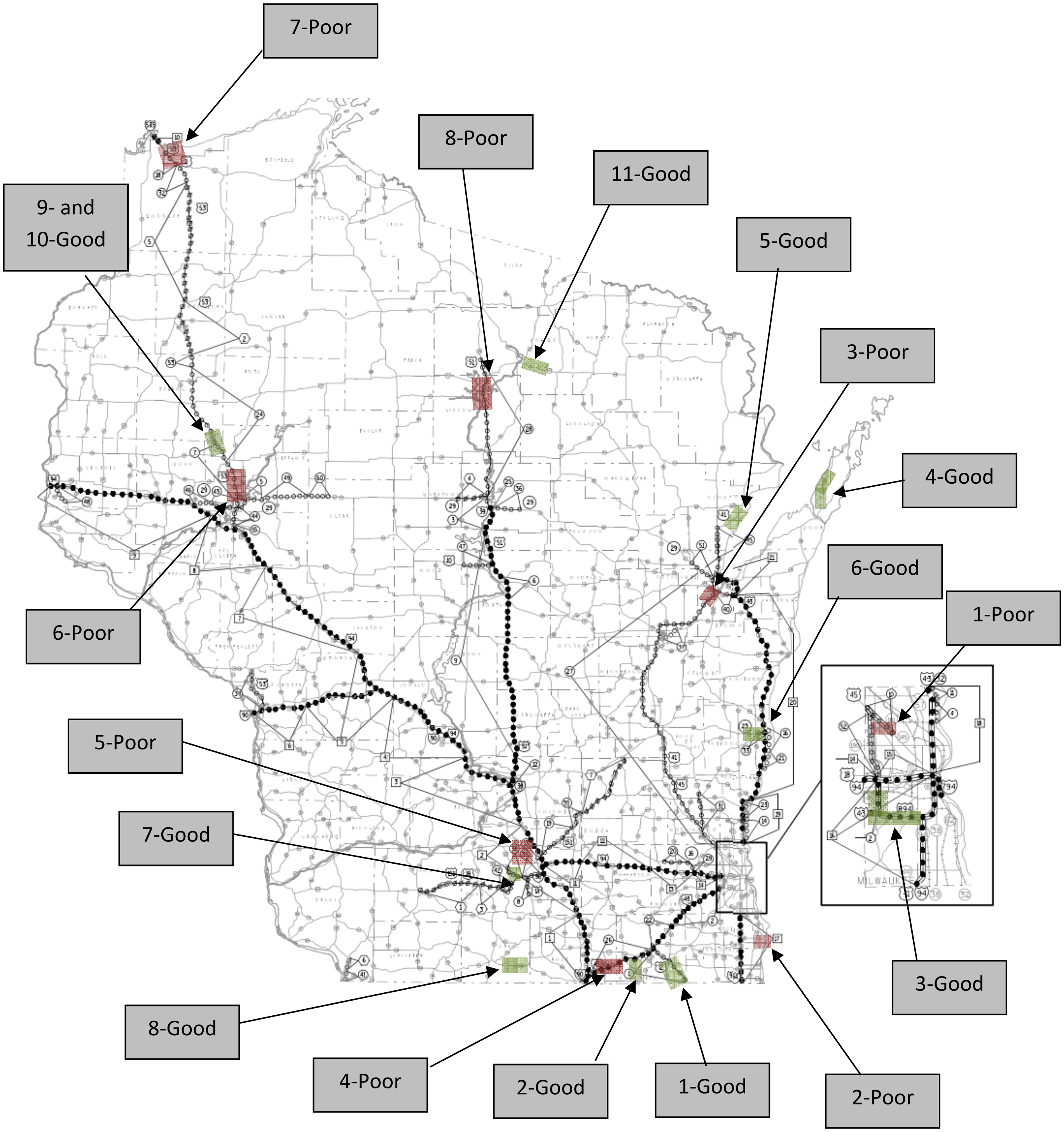

Figure 2. Project location map.

6. Project-Specific Analysis

In this analysis, each of the 19 rubblized projects were evaluated separately. Design and construction details are provided, along with historic rehabilitation measures, if available. Information from soils reports is summarized. In addition, results from a 2011 field analysis are provided for each pavement.

6.1 USH 12, Walworth County - 1 Good

This rubblization project, completed in 1998, extends from the Illinois state line north to STH 50 in Lake Geneva. The roadway is a four-lane divided highway.

The original concrete pavement was a 9-in JRCP paved in 1965. According to the pavement design report, numerous maintenance activities were performed when the roadway had a concrete surface, including base patching, joint repair, joint sealing and grinding. Specific dates for these projects were not available, but pavement distress data suggest that a joint repair and/or diamond grinding operation was completed in portions of the concrete pavement circa 1986.

Based on historic WisDOT pavement distress survey information, the existing JRCP was in poor condition at the time of rubblizing. The pavement had longitudinal joint distress along with one to four distressed transverse joints in every 528-ft survey segment. The distress at the joints was severe, with loss of material in the wheel paths. The average PDI for the roadway was 77.

The project's soils report described the underlying material as dense, silty sand and gravel with medium resistance to boring. Edge drains were installed prior to rubblizing. Rubblization took place in July 1998, and 7 inches of type HV HMA pavement was paved in three layers during July and August 1998. Paving followed the rubblization operation, and lanes were opened to traffic after the lower layer of HMA was paved.

During the first week of HMA paving, the following notes were made in the project diary:

- A test hole was excavated in the rubblized layer on July 1, 1998 (first day of HMA paving). The pavement breaking size met specification requirements.
- Rubblization passes over gas lines were observed closely.
- A 200-ft section had "serious flexing" after traffic opened on the lower 3.5-in HMA layer. An additional HMA layer was placed in this area.
- An area of soft subgrade was noted near the middle of the project. The rubblization pattern was modified to break the concrete into bigger pieces in this area.

The March 2011 field review noted some tenting at transverse joints and areas of longitudinal joint distress. However, all of the HMA cracks were sealed, which should prevent water penetration and delay further crack deterioration. Overall, it was noted that the pavement is in good condition for its age (nearing 13 years).

6.2 STH 67, Walworth County - 2 Good

This 0.6-mile, two-lane urban segment between Theatre Rd. and Congress St. in Williams Bay was part of a larger rehabilitation project on STH 67. The segment of interest was rubblized in 1999.

The 1962 JRCP had not received a major rehabilitation during its 37-year service life. The 9-in JRCP was constructed over 6 inches of gravel. WisDOT pavement distress survey information noted distress at the longitudinal joint and greater than five distressed transverse joints in a 528-ft survey segment, all with the highest (worst) distress severity rating. The PDI of the existing pavement was 93.

The 1995 soils investigation reported that the subgrade was predominately silty sand and gravel, and the water table was estimated to be greater than five feet below the surface. The soil had medium resistance to boring.

One inch of the JRCP was removed prior to rubblization, which took place in August 1999. Four inches of HMA pavement (type MV) were paved later that year (exact timeframe unknown).

The following notes were recorded in the construction diary during the first day of rubblization:

- "Rolling and compaction operations were observed and found to be adequate. The existing base has not shown any failure so far."
- "A test hole was opened to determine the size of the concrete particles. Some of the particles were larger than the spec. The rubblizing machine was slowed to decrease the particle size."
- A rubblizing contractor representative "was on site to observe the process."

Transverse and longitudinal cracking were observed during the March 2011 field review, but acceptable ride quality was noted. Cracks were sealed. Cracking was noted but was not severe, which was typical for the age of the pavement (nearing 12 years).

6.3 I-894, Milwaukee County - 3 Good

Eight miles of I-894 (from the Belton railroad structure to Loomis Road, excluding the Hale Interchange) were rubblized in 2003. This roadway is a 6-lane divided freeway. The original pavement, constructed in 1963, was a 9-in JRCP over 8 inches of CABC. The pavement was overlaid with HMA in 1972, milled and overlaid with HMA in 1982, and milled and overlaid with stone matrix asphalt (SMA) in 1994. All asphaltic pavement was milled in 2002, along with approximately one inch of the original concrete pavement. The pavement was rubblized in April through June 2003, and 5.25 inches of E-30 HMA pavement was paved in April through July 2003. According to the project's soils report, underlying materials included clay and silt with a varying amount of sand and gravel.

Prior to rubblization, the existing HMA surface had banded transverse cracks every 50 to 80 ft along with distressed longitudinal cracking. The average PDI of the HMA surface was 63. Distress data were not available for the concrete surface prior to the initial HMA overlays.

The construction diary for this project was quite detailed (see Appendix 1). During construction, close attention was paid to the condition of the rubblized pavement layer, as described in the following notes from the construction diary:

- A 100-ft test area was rubblized prior to major construction. A test hole was dug at this location to determine particle size. Some minor adjustments were made to the breaking pattern at this time to fine tune the process for the remainder of the I-894 project area.
- Immediately following all rubblization activity, the base was observed under the action of loaded trucks (test rolling). Areas with significant deflection were marked and repaired with concrete base patching. The concrete base patch areas were lightly cracked prior to HMA paving. In some areas, an additional ¼ inch of HMA was paved. The lower HMA layers were profile milled prior to paving the surface layer.
- It was noted that weak areas discovered during test rolling were typically near highly deteriorated joints. The project manager stated that locations of the bad joints "are fairly clear and defined. They either yield or they don't." In some areas, water was noted seeping up through cracks after rubblization.
- At several points during the construction timeframe, the rubblization process was observed by project staff, WisDOT design staff, and technical experts on rubblization. They were "impressed with the thoroughness of the processes."
- FWD testing verified that the pavement structure met the design structural number.

This close attention to detail, along with efforts to locate and address areas of weaker subgrade support, led to excellent performance of this rubblized pavement system. Very few distresses were noted during the March 2011 field review. This rubblized pavement system has had good performance for nearly eight years and is considered one of the premier rubblized pavement systems in Wisconsin.

6.4 STH 42, Door County - 4 Good

This 12-mile segment of STH 42 was rubblized in 2000. The original two-lane pavement was constructed in 1932. The driving lanes were 6-in JRCP, placed directly on native soils. The roadway was widened in 1978 with HMA placed on either side of the JRCP. Several HMA patches were placed between 1978 and 2000. These patches were milled prior to rubblization.

Before rubblization, the surveyed segments of the existing concrete pavement had PDI values ranging from 3 to 67 and averaged 30. Transverse joint faulting (¼- to ½-inch) was noted, and several areas had low-severity transverse joint distress.

The soils for this project, as characterized in 1992, are well-drained and nearly level with a sandy loam or loam subsoil. The subsoil sits on well-drained fine sandy loam till or bedrock. In some locations, however, water was observed seeping up through centerline longitudinal joints of the concrete.

The concrete was rubblized in September 2000, and 3.5 inches of HMA pavement (type MV) were placed in two layers that same month. The final 1.5-in HMA pavement layer was constructed in May 2001.

As noted in the project's construction diary, the first 100 ft of rubblizing resulted in pavement that was over-broken. The rubblizing intensity was reduced. Three soft spots were noted during rubblization, and the contractor placed "wedges" in these areas.

The March 2011 field review noted formation of some distresses (transverse, longitudinal and wheel path cracking), along with localized rough spots. It was noted that close attention to future maintenance will likely result in long-term performance.

6.5 USH 41, Oconto County - 5 Good

In 2002, portions of this USH 41 project were rubblized and overlaid with HMA. The roadway is a four-lane divided highway. Rubblization took place in the NB lanes between Brookside Cemetery Rd. and Kreuger's Quarry Rd. The SB lanes between Froelich Rd. and the USH 141 interchange were also rubblized. Remaining portions were rehabilitated with an HMA overlay of previous HMA overlays.

The existing 1968 concrete was a 9-in non-doweled JPCP over 6 inches of CABC over 6 inches of granular subbase. Prior to rubblization, the pavement had severely distressed joints and cracks, along with faulting at transverse joints. The PDI in these areas was approximately 70. One segment of concrete pavement had been repaired in 1999 or 2000. The PDI in this location was 3.

A 1989 soils report prepared for expansion of the highway to four lanes characterized the soils as follows: the southernmost 3.7 miles of the project have well-drained fine sandy loam to silt loam soils that become clay-like at a depth of 2-3 feet. The remainder of the project was characterized as having poor- to very poor-drained loamy fine sand soils. Excavation below subgrade was recommended for transitions between cuts and fills to avoid "textural differences that may cause frost heaves."

Very few details were provided in this project's construction diary. Paving of 6.5 inches of E-10 HMA took place in July 2002. According to project plans, locations of asphaltic base patching were designated within the rubblized portions of the project.

The March 2011 field review concluded that the project was in good shape overall. The NB lanes had a smooth ride and very few cracks. The SB lanes showed more distress (transverse and longitudinal cracking) than the NB lanes, but were still performing well.

6.6 STH 23, Sheboygan County - 6 Good

This four-lane divided section of STH 23 was rubblized in 2003. The existing concrete pavement, constructed in 1978, was a 10-in JPCP built over 9 inches of CABC. Rehabilitation between 1978 and 2003 included patching and diamond grinding. Prior to rubblization, the pavement's transverse joints and/or cracks had minor faulting and moderate to severe distress every 100 to 200 ft. The average PDI for the pavement was 56.

The soils report for this project, prepared in 2002, classified the soils along STH 23 as well-drained to moderately well-drained loams and silt loams. Permeability of the soils was described as moderate, and the available water capacity was high.

The existing concrete pavement was rubblized in June 2003, and 4.75 inches of warranted HMA pavement were placed later that month. Paving was completed in stages, with traffic allowed on the lower HMA layer.

Several trouble areas were noted in the construction diary during the rubblization phase. The rubblization frequency was modified on the first day of pavement breaking due to soft subgrade. Soft subgrade was also addressed with repairs at several 50- to 200-ft long segments. (These segments are documented in Appendix 1.) The soft subgrade was removed and backfilled with rubblized concrete and CABC.

The May 2011 site review noted several areas of premature distresses: tenting at transverse cracks (presumably over previous concrete pavement joints), longitudinal joint deterioration, and random cracking. The subgrade repair areas noted above were reviewed in greater detail. Some of these locations had slightly better performance than the rest of the roadway. Improved performance was noted in one area in particular where soft spots in the shoulder had been excavated and backfilled with rubblized material and CABC.

6.7 USH 12/14 (Madison Beltline), Dane County - 7 Good

In 2005, the 1.5-mile segment of the Madison Beltline between Gammon Rd. and Whitney Way was rubblized. The roadway is a four-lane divided highway. The original pavement had been constructed in two stages (eastern half in 1965, western half in 1969), both with 9-in JRCP over 6 inches of base course over 9 inches of granular subbase. Joint spacing was 80 feet. The WB lanes of the project were diamond ground in 1998. Because the WB lanes were diamond ground, the EB and WB lanes were analyzed separately in the overall analysis (Section 7). The EB project was designated as 7-Good a, and the WB project as 7-Good b.

Prior to rubblization, the existing concrete pavement had a PDI of 36. Minor faulting (< ¼ in) was noted at some transverse joints.

Soils on the project ranged from sands and gravel to silts and clays. The project's soils report stated: "Few problems are predicted… presence of base course and a granular subgrade should provide an adequate platform for breaking."

Construction was completed under night time closures. The pavement was rubblized in September 2005, and the lower HMA layer was paved directly after. Traffic was allowed on lower layers. A total of 5.5 inches of warranted HMA pavement, placed in three layers, was finished in early October 2005. No other notes on the rubblization process were recorded in the construction diary.

The May 2011 site review found the pavement to be in good condition. Some longitudinal cracking was noted in the wheel paths. Most of these cracks had been sealed.

6.8 STH 11, Green County - 8 Good

The original concrete pavement in this project was constructed in two stages and had different rehabilitation activities; therefore it was analyzed as two separate segments in the overall analysis (Section 7). The pavement in the western portion of the project (8 miles between Hiltbrandt Road and CTH G) was 9-in JRCP constructed in 1972 (8-Good a). The pavement had 80-ft doweled joints and was constructed over 9 inches of CABC. The eastern portion (3 miles between CTH G and Airport Road) was constructed in 1975 with 8-in JPCP over 6 inches of base course over 9 inches of granular subbase (8-Good b). The joints were non-doweled and randomly spaced. This pavement was diamond ground in 1997.

Prior to rubblization, the existing concrete pavement in the western portion of the project had minor to moderate joint distress at transverse joints and/or cracks and minor faulting at transverse joints (< ¼ in). The average PDI was 53. The eastern portion had minor faulting at transverse joints (< ¼ in), and the average PDI was 6.

The soils investigation found fine to medium sands with some silt, sandy silts, and shallow bedrock. The majority of the borings had a sandy subgrade. In a WisDOT memo following the soils investigation, it was stated that "the likelihood is very high for a successful project if the rubblization is scheduled for late summer/early fall, when subgrades are expected to be at the driest they will be… I can say this is one of the better candidates for rubblization, from a soils perspective, that I have seen."

Both segments were rubblized in July 2000 and overlaid with 5 inches of HMA pavement in August and September 2000. Prior to rubblization, edge drains were installed at "low points and critical areas." On the first day of rubblization, two test holes were dug, and size requirements were met. Two additional test holes were excavated the following day when a soft subgrade area was encountered. The breaking pattern was modified in these soft areas. One area of concern was test rolled, and it was determined that an 8-in layer of CABC and drainage installation would benefit the pavement structure.

Some transverse cracking with tenting was noted during the March 2011 field review. However, the smoothness was acceptable, and the pavement was in good condition overall.

6.9 USH 53 NB and SB, Chippewa County - 9 Good and 10 Good

These two USH 53 projects were rubblized in 2005 (NB lanes) and 2007 (SB lanes). The highway is four-lane divided. The pavements were part of a 24-mile pavement replacement/maintenance program on USH 53 between Chippewa Falls and New Auburn. This corridor replacement program also included this study's project 6-Poor.

The existing conditions, rubblization and reconstruction details were identical for the NB and SB lanes but were analyzed separately because they were completed in different years (2005 and 2007 for projects 9- and 10-Good, respectively).

The existing concrete pavement was an 8-in CRCP built over 2.5 inches of asphalt-stabilized base course over 6 inches of CABC over 9 inches of granular subbase. The CRCP was originally constructed in 1973 and was overlaid with HMA pavement in 1996. This overlay was milled prior to rubblization. The native soils are primarily sandy loams and silt loams.

The condition of the HMA overlay prior to the rubblization project was good; the average PDI value for both the NB and SB lanes was 12. Prior to the 1996 HMA overlay, the CRCP exhibited pavement deterioration and delamination. The average PDI values of the CRCP for the NB and SB lanes were 81 and 82, respectively.

For the NB project (9 Good), the HMA overlay was milled and subgrade drainage outlets were installed in July 2005. The pavement was rubblized and HMA was paved in August 2005. Three layers of warranted HMA pavement (6.5 inches total) were placed prior to opening to traffic; work was completed in the lane adjacent to live traffic. No problems or details relating to rubblization were documented in the project's construction diary.

A similar schedule was followed for the SB lanes construction (10 Good). Rubblization, subgrade drainage outlet installation, and paving took place in May 2007. Three layers of warranted HMA pavement (6.5 inches total) were placed prior to opening to traffic; work was completed in the lane adjacent to live traffic. No major problems or events were reported in the construction diary.

In March 2011, the NB lanes were reviewed. The pavement was in good condition with no major distresses noted.

6.10 USH 8, Oneida County - 11 Good

This project was 6.2 miles in length and was rubblized in 2003. The original concrete pavement was a two-lane, 9-in doweled JPCP constructed in 1991 over 6 inches of CABC. At the time of rubblization, the existing JPCP had mild distress at joints and/or cracks and minor faulting at transverse joints (< ¼ in). The PDI ranged from 19 to 53, and the average over the project length was 40.

The soils investigation found fine to medium silty sands and sands with a trace to some gravel. Nine areas were identified as susceptible to frost heave. In these areas, the soils were silty sands and silt, with shallow ground water. The frost heave segments were 50 to 500 ft in length. Eight of these segments were repaired with concrete pavement removal and 4-ft EBS. In one location, a 4-ft wide underdrain system was installed.

The drainage system described above was installed in August 2003. The pavement was also rubblized in August, and 5.5 inches of Type E-3 HMA pavement were placed in three layers during August and September 2003.

No problems were noted in the construction diary relating to rubblizing. After rubblizing and paving of the lower HMA layer were completed, however, concerns were raised about the rubblized pavement settling. The planned HMA thickness was 4.5 inches. To address the settling issue, an additional 1 inch of HMA was placed, for a total of 5.5 inches.

The March 2011 field review concluded that the pavement was performing well overall. Transverse cracks were noted, but were spaced at 50 to 100 ft or more. The cracks had been sealed; however in some locations cracks were starting to open again. In these locations, slight tenting was apparent.

6.11 CTH E (West Silver Spring Drive), Milwaukee County - 1 Poor

The western portion of this urban four-lane, median-divided roadway (124th St. to 107th St.) was constructed in 1963-64. The eastern portion (107th St. to West Fond Du Lac Ave.) was constructed in 1967. The existing pavement was the same for both portions: 9-in JRCP over 6 inches of gravel base. In 1990, the EB lanes were resurfaced with 3 inches of HMA pavement. As this pavement is not part of the WisDOT State Trunk Network (STN), historic distress data were not available.

A 2007 soils investigation reported that the majority of the project had silty clays with a "relatively high" moisture content (17 percent). It was noted in the soils report that "drainage measures should be incorporated into the pavement design." However, no subsurface drainage was included in the final design.

Provisions were included in the plans for identification and repair of soft or yielding areas noted during rubblizing. Test rolling of the rubblized surface was specified for the majority of the project, and yielding areas were to be removed and replaced with base aggregate dense (BAD). In addition, specific locations were marked on the plans for concrete removal and replacement with BAD; these were presumably areas of severe distress noted prior to construction. An example of a distressed joint exposed after milling and prior to rubblization is shown in Figure 3. This joint was removed and repaired with high early strength concrete prior to rubblizing. Additional subgrade stabilization efforts were included in the plans by means of undistributed EBS (8,200 CY), along with test rolling, geogrid reinforcement and breaker run.

The EB overlay was removed prior to rubblization, along with approximately 2 inches of the existing JRCP (EB and WB). Rubblization took place in June through August 2009. Six inches of type E-10 HMA pavement were paved in June through September 2009.

A test hole was excavated on the first day of rubblization, and the breaking pattern was modified to a "heavy cracking." As rubblization and paving progressed, soft and yielding areas were identified and repaired with EBS and replacement with BAD, as directed in the plans. Areas of HMA failure (rutting and shoving) were noted at several locations.

During the pavement's first winter (2009-2010), tented transverse cracks formed. The tenting settled in the spring, but additional cracking was observed, along with shoving and tearing of the HMA. A test area was excavated in September 2010. No visible signs of failure were noted in the rubblized pavement after it was exposed, nor after it was test rolled. Former concrete pavement transverse joints were apparent, however, and water was observed seeping up through them the next day. It was concluded that the observed failures were a result of a weak, wet subgrade. Areas exhibiting failures were milled and patched, as addressing the subgrade issue would have been too extensive and costly a repair. [*23*]

A field review the following March (2011) noted similar issues, along with potholes in the HMA pavement. In one pothole, reinforcing mesh was observed on top of the rubblized layer.

Figure 3. Existing concrete pavement transverse joint on CTH E (a) after milling existing HMA overlay and (b) during repair with HES concrete prior to rubblization. [*23*]

6.12 STH 20, Racine County - 2 Poor

This 4-lane urban project was 1.64 miles in length and was rubblized in 2002. The existing concrete pavement was constructed in 1955. It was a 9-in JRCP over 3 inches of granular crushed stone. Native soils consist primarily of loams, silt loams and sandy loams. Prior to rubblization, the concrete had moderate deterioration at cracks and/or joints, along with transverse faulting (¼ to ½ inch). The average PDI for the pavement was 60.

Two inches of the concrete were ground prior to rubblization, leaving seven inches of rubblized material. Rubblization and placement of 4.5 inches of E-10 HMA pavement took place in June 2002. No problems were noted in the construction diary during rubblization. However, after the lower HMA layer was placed, noticeable settlement was observed. An additional one inch of HMA was placed in the center two travel lanes for the purpose of providing a leveling layer prior to placement of the surface layer. An additional 1,400 tons of E-10 mix was change ordered for this purpose. A pay deduct was also imposed for 2,250 tons of HMA that did not meet density requirements. The location of this pavement is unknown.

The March 2011 field review noted some areas of longitudinal joint distress and patching. The patching was mainly near utilities.

6.13 USH 41, Brown County - 3 Poor

This 3-mile portion of USH 41 (four-lane divided highway) was rubblized in 2002. The existing concrete pavement, built in 1965, was a 9-in JRCP constructed over 8 inches of single aggregate bituminous mix over 4 inches of crushed stone. Short segments (< 0.6 miles) had been overlaid with HMA; this material was milled prior to rubblization. The predominant soil type is Oshkosh silt loam.

Prior to rubblization, the existing JRCP had severe joint and/or crack distress, along with significant slab breakup. Some pavement patches were in poor condition. The pavement's average PDI value was 88.

The concrete pavement was rubblized in August 2002. 6.5 inches of E-30 HMA were also placed in August 2002. No problems were noted in the construction diary relating to rubblization or paving.

Field reviews were conducted in the spring of 2010 and 2011. Transverse cracks were present at the location of previous JRCP joints (80-ft spacing). In 2010, cracks in the northern part of the project had not been sealed, and many were tented. Cracks in the southern portion had been sealed and were not tented. Tented areas were milled and sealed in 2010. In 2011, remaining cracks that had not been sealed were now tented. Cracks where sealant had ruptured were also tented, as shown in Figure 4.

Figure 4. Tented crack on USH 41, Brown County.

6.14 I-43, Rock County - 4 Poor

This four-lane divided Interstate highway project was 5.4 miles long and was rubblized in 2007. The existing concrete pavement was constructed in 1976 as a 10-in non-doweled JPCP over 6 inches of CABC. In 1994, the pavement was patched and diamond ground.

Prior to rubblization, severely distressed joints and/or cracks were documented in the NB lanes. Severely distressed patching was noted in the SB lanes. All lanes had minor faulting (< ¼ in) at transverse joints. The average PDI for the pavement was 73.

The 2005 soils investigation found deep, well-drained and moderately well-drained soils that have silty clay loam to sandy clay loam subsoil over sandy loam glacial till. It was noted that the silty sand subgrade soils should be more than adequate to support the rubblization forces. The profile of the road was higher than the seasonal high water table, but it was recommended that rubblization take place in late June to August to reduce the risk of soft subgrade. The investigation also found that the rocking action of non-doweled slabs "promotes capillary action of the water within the road subgrade, which literally pumps ground water up through the pavement structure. Although much of the subgrade is sandy, there is enough silt mixed in where it will support the upward movement of water through it."

A drainage system was not originally planned for the project. However, temporary traffic on the shoulders during rubblization of the NB lanes resulted in pavement failure suspected to be due to trapped water and saturated subgrade. Edge drains were subsequently installed along both the NB and SB lanes.

The NB lanes were rubblized in June and early July 2007. Three lifts of warranted HMA (total of 6 inches) were paved in July. The SB lanes were rubblized in August 2007, and 6 inches of warranted HMA were placed in September 2007.

Achieving a desirable rubblized layer was difficult. A modified breaking pattern was implemented, as the rubblizer operator suspected poor subgrade quality. Several inspections by WisDOT staff and rubblization experts were performed, and the rubblized layer was determined to be an acceptable paving platform.

The March 2011 field review noted longitudinal paving joint distress. These cracks had been sealed. Transverse cracks were also apparent, with some showing signs of tenting and movement. These cracks had not been sealed.

According to the 2009 WisDOT pavement distress files, transverse cracking was evident in the NB lanes and not in the SB lanes. As mentioned previously, severely distressed joints and/or cracks were present in the NB lanes, but not in the SB lanes. It is therefore likely that the tented transverse cracks formed at the location of the concrete pavement joints with severe distress.

6.15 STH 113, Dane County - 5 Poor

Portions of this project, located north of Madison, were rubblized in 2004. The original concrete pavement, constructed in 1964-65, was a 9-in JRCP over 6 inches of gravel over 9 inches of granular subbase. The concrete pavement was patched and diamond ground in 1990.

Prior to rubblization, moderately distressed patching and minor transverse joint faulting (< ¼ in) were present in the JRCP. The average PDI of the rubblized sections was 35.

The majority of soils in the rubblized pavement area (Kennedy Rd. to Division St.) consist of silt loam. These soils have a seasonal high water table of 3 to 5 ft. The soils closer to CTH M have a seasonal high water table of 0 to 1 ft. The pavement design report (PDR) therefore recommended that rubblization take place in late July or early August, when the subgrade would likely be driest. If rubblization was difficult in this marshy area, it was further recommended that the breaking pattern be adjusted and that a crushed stone base course leveling layer be placed prior to HMA paving (this was not included in project plans).

The pavement was rubblized in June, July and August 2004. Some rubblizing took place earlier than recommended in the PDR, as noted above. Paving of 4.5 inches of HMA pavement (type E-3) took place in June through September.

During rubblization, a test hole noted that effective breaking was taking place. However, it was also noted that there were "depressed areas that will require an [HMA] leveling course." A few soft spots noted by the rubblizer operator were excavated and filled with broken concrete and base material.

Wet conditions were noted during rubblizing. A portion of the project near Kennedy Rd. and River Rd. was a marsh area. In addition, several rain events occurred while rubblizing the portion of the project that coincides with STH 19.

The May 2011 field review noted transverse cracking, some wheel path cracking, and occasional random cracking. The greatest number of distresses were present on STH 19 into Waunakee. Many transverse cracks were tented. None of the tented cracks had been sealed.

6.16 USH 53, Chippewa County - 6 Poor

This 7.4-mile, four-lane divided highway project was rubblized in 2006. The pavement was part of a 24-mile pavement replacement/maintenance program on USH 53 between Chippewa Falls and New Auburn. This corridor replacement program also includes this study's projects 9-Good and 10-Good.

The first two miles of the southern end of the project was an 8-in JRCP built in 1970 (6-Poor a). It was constructed over 6 inches of CABC over 9 inches of granular subbase. The remainder of the project was originally an 8-in CRCP built over 2.5 inches of asphalt-stabilized base course over 6 inches of CABC over 9 inches of granular subbase (6-Poor b). This portion was constructed in 1972 and overlaid with HMA pavement in 1995; this overlay was milled prior to rubblization.

Prior to rubblization, the JRCP had moderately distressed joints and/or cracks. The average PDI in this segment was 38. The HMA overlay of the CRCP had transverse and longitudinal cracking, some with severe deterioration. The average PDI in this segment was 41. Prior to overlay, the CRCP had some pavement deterioration and delamination. The average PDI was 45.

This project's paving diary was not available; the rubblizing and paving timelines are not known. Seven inches of warranted HMA pavement were paved over the rubblized concrete pavement.

The March 2011 field review noted tented transverse joints. The tenting has been observed to settle during the spring season, but damage to the pavement is permanent and affects the pavement ride. This pavement is scheduled for rehabilitation during the 2011 construction season; it will be five years old. The pavement will receive a two-inch mill and inlay as negotiated during conflict resolution under the pavement's warranty provisions.

6.17 USH 53, Douglas County - 7 Poor

This 4-lane divided highway (approximately 6 miles in length) was rubblized in 2002. The existing concrete pavement was an 8-in CRCP constructed over 6 inches of CABC over 18 to 30 inches of select borrow. The pavement was built in 1981, and a portion of the pavement's centerline longitudinal joint was removed partial depth and patched circa 1998. Soils in the area are generally sandy and underlain by clay materials.

Prior to rubblization, the CRCP had cracks with minor distress. The pavement also had moderate to severe distress along the longitudinal joint. It is assumed that this deterioration involved the concrete pavement adjacent to the patched longitudinal joint, as the pavement was rated as having good-condition patching. The average PDI of the pavement was 83.

No problems concerning rubblization were noted in the construction diary. The concrete pavement was rubblized in June 2002. Five inches of warranted HMA pavement were paved; the paving timeline is unknown.

Severe distresses developed in this pavement after three to five years in service. The distress was primarily transverse cracking with tenting. In 2010, the existing 5 inches of HMA were milled full-depth and replaced with 2 inches of SMA over 3 inches of HMA (type E-3). The pavement warranty had expired, and therefore this work was performed at the Department's cost.

Several site reviews were conducted for this project. A forensic investigation was conducted prior to re-paving the project in 2010. A section of HMA was removed full-depth at a tented transverse crack location (Figure 5). A transverse crack was exposed in the underlying rubblized layer; it had not been obliterated during rubblization. Loss of HMA material was noted at the bottom of the pavement layer, and the remaining loose aggregate was stripped of its asphalt layer. Similar distress was also noted in Michigan. [*4*]

It was hypothesized that because the existing CRCP crack had not been obliterated during rubblization, reflective cracks propagated through the HMA layer. Water seeped through the open cracks and did not drain through the rubblized layer. During the winter months, this ponded water froze and expanded, resulting in tenting of the HMA cracks. Tenting settled during the spring thaw, but damage remained in the pavement. HMA cracks had been sealed in 2007, but excessive stress on the cracks caused them to open again.

The new pavement was surveyed in March 2011, when it was nearing one year old. Longitudinal joint distress was evident, likely caused by poor joint construction. Some transverse cracking was noted, and the ride was slightly "wobbly." It is likely that the same failure mechanism is occurring, as the former CRCP cracks were not repaired prior to repaving the roadway.

Figure 5. Removal of HMA pavement at transverse crack location to expose rubblized CRCP layer.

6.18 USH 51 SB, Lincoln County - 8 Poor

This 8.8-mile, four-lane divided highway project was rubblized in 2006. The existing pavement was 9-in non-doweled JPCP constructed in 1983. The JPCP was built over 6 inches of CABC.

Prior to rubblization, the JPCP had moderate distress at joints and/or cracks and minor transverse faulting (< ¼ in). The PDI of the pavement segments ranged from 10 to 63 and averaged 31.

The project soils were defined as predominantly fine to medium sands with a trace of silt and little to some gravel. During the planning phase, five areas were identified as susceptible to frost heave, and a 2-ft wide pipe underdrain system was specified.

During construction, the subgrade in three of these areas was identified as weak, and 4-ft EBS was performed. The trenches were lined with geotextile fabric, backfilled with 1.25-in BAD, and a transverse discharge flume was installed to the median ditch. One additional area of weak support was identified during construction and was treated in the same way. The remaining two frost heave areas were treated with the 2-ft wide pipe underdrain system, as specified. The treated segments were 90 to 360 ft in length.

Prior to paving, it was documented that some of the EBS areas later became wet, and attempts were made to dry them out. However, heavy rain occurred, and some wet/soft areas were paved over without drying out.

The construction timeline was as follows: the concrete pavement was rubblized in late April and early May 2006, and 7 inches of warranted HMA pavement were paved in three layers during May and June 2006. The work was completed under full closure with crossovers to the northbound lanes.

It was observed in the March 2011 field review that the most severe distresses occurred in a one-mile segment of the roadway, where the ride was rough and tented cracks were noted. A significant amount of longitudinal cracking was also recorded.

6.19 I-39, Portage County

This rubblization project was not investigated in detail in this analysis but has been monitored by WisDOT after showing premature failures. The 4-lane divided Interstate highway was rubblized in 1999. The pavement consisted of 5 in of warranted HMA over 9 in of rubblized JRCP (built in 1970) over 2.5 in of asphalt treated base over 6 in of dense graded aggregate base course.

This was one of the first major rubblization projects in Wisconsin, completed in 1999. Concrete pavement joints with significant deterioration were base patched prior to rubblization. The construction project included several test sections with thicker HMA overlays. [*3*] In the winter of 2009-2010, several unsealed cracks tented, and the effect on ride was still noticeable in May 2010. In March 2011, additional cracks had heaved. Most of these cracks had not been sealed. The portion of the project north of the test area (USH 10 EB to the north) is planned to be milled and overlaid with 2 inches of HMA in the summer of 2011. The portion to the south will be crack sealed.

6.20 Summary

Specific factors that potentially led to good or poor performance of the rubblized pavements have been described in the project descriptions above. These factors are summarized in Tables 4 and 5 for projects in the good and poor performance categories, respectively. Discussion follows of notable observations. It should be noted that many of the factors in Tables 4 and 5 were compiled based on project construction diary entries. The level of detail kept in diaries varied from project to project, and it is possible that a significant factor occurred on a project but was not recorded in the diary.

Table 4. Summary of Factors Leading to Good and Poor Pavement Performance - Good Performance Category

Project	Factors Potentially Resulting in Good Performance	Factors Potentially Resulting in Poor Performance	Tenting of Transverse HMA Cracks, 2011
1 Good	• Test hole excavated • Close observation of the rubblization operation • Modifications to plan in problem areas (additional HMA placed, modification of breaking pattern) • Adequate HMA pavement crack sealing	• Severe concrete pavement joint distress - not documented as repaired prior to rubblization	Minor
2 Good	• Test hole excavated • Close observation of the rubblization operation • Modifications to plan in problem areas (modification of breaking pattern)	• Severe concrete pavement joint distress - not documented as repaired prior to rubblization	No
3 Good	• Test hole excavated • Close observation of the rubblization operation • Test rolling of rubblized layer • Modifications to plan in problem areas (modification of breaking pattern, base patching in weak areas, additional HMA placed, profile milling of lower HMA layers) • FWD testing to verify structure capacity	*No notable factors*	No
4 Good	• Close observation of the rubblization operation • Modifications to plan in problem areas ("wedges" placed)	*No notable factors*	Minor
5 Good	• Asphalt base patching prior to rubblization	• Severe concrete pavement joint distress - not documented as repaired prior to rubblization	No
6 Good	• Close observation of the rubblization operation • Modifications to plan in problem areas (excavation of soft subgrade areas)	• Severe concrete pavement joint distress - not documented as repaired prior to rubblization	Yes
7 Good	• Adequate HMA pavement crack sealing	*No notable factors*	No

Table 4. continued, Summary of Factors Leading to Good and Poor Pavement Performance - Good Performance Category

Project	Factors Potentially Resulting in Good Performance	Factors Potentially Resulting in Poor Performance	Tenting of Transverse HMA Cracks, 2011
8 Good	• Several test holes excavated • Close observation of the rubblization operation • Test rolling of rubblized layer in area of concern • Modifications to plan in problem areas (modification of breaking pattern, CABC layer placed, additional drainage installed)	*No notable factors*	Minor
9-Good	• Adequate HMA pavement crack sealing and distress repair	*No notable factors*	No
10 Good	*No notable factors*	*No notable factors*	No
11 Good	• Close observation of the rubblization operation • Modifications to plan in problem areas (additional HMA thickness) • Adequate HMA pavement crack sealing • Potential frost heave areas identified prior to construction	*No notable factors*	Minor

Table 5. Summary of Factors Leading to Good and Poor Pavement Performance - Poor Performance Category

Project	Factors Potentially Resulting in Good Performance	Factors Potentially Resulting in Poor Performance	Tenting of Transverse HMA Cracks, 2011
1 Poor	• Test hole excavated • Close observation of the rubblization operation • Test rolling of rubblized layer • Modifications to plan in problem areas (modification of breaking pattern, base patching in weak areas) • Potential problem areas identified prior to construction	• High moisture content noted during soils investigation; no drainage measures installed • Reinforcing steel remaining on surface of rubblized layer prior to paving	Yes
2 Poor	*No notable factors*	*No notable factors*	No
3 Poor	*No notable factors*	• Severe concrete pavement joint distress - not documented as repaired prior to rubblization • Inadequate HMA crack sealing	Yes
4 Poor	• Close observation of the rubblization operation • Modifications to plan in problem areas (modification of breaking pattern, addition of edge drains)	• Severe concrete pavement joint distress - not documented as repaired prior to rubblization • Inadequate HMA crack sealing	Minor
5 Poor	• Modifications to plan in problem areas (modification of breaking pattern, addition of French drains)	• Wet conditions noted during rubblization • Inadequate HMA crack sealing	Yes
6 Poor	*No notable factors*	• Moderate concrete pavement joint distress - not documented as repaired prior to rubblization • Inadequate HMA crack sealing	Yes
7 Poor	*No notable factors*	• Inadequate HMA crack sealing	Yes
8 Poor	• Modifications to plan in problem areas (modification of breaking pattern, base patching in weak areas) • Potential frost heave areas identified prior to construction	• Moderate concrete pavement joint distress - not documented as repaired prior to rubblization • Wet subgrade not allowed to dry prior to paving	Yes

6.21 Discussion

Excavation of rubblization test holes.

- "Good"-performing projects: 4 of 11 (36 percent)
- "Poor"-performing projects: 1 of 8 (13 percent)

Excavation of test holes during the first day of rubblization (at minimum) is a requirement outlined in the standard specifications. [*20*] Results of this procedure were documented in less than half of the construction diaries reviewed. It could be assumed that if test hole excavation was not mentioned in the diary, no problems were observed and specification requirements were met. However, clear documentation should be made to indicate that care was taken during this critical step in the rubblization process.

Close observation of the rubblizing process.

- "Good"-performing projects: 7 of 11 (64 percent)
- "Poor"-performing projects: 2 of 8 (25 percent)

Notes in the construction diaries indicated that close observation of the rubblizing process took place more often in projects with good ultimate performance. "Close observation" includes activities such as

- Communication with the rubblizer operator
- Observation of the rubblizing process by project staff, Department personnel, contractor representatives, and other experts
- Inspection of the rubblized layer for particle size requirements
- Inspection of the rubblized layer for areas of settlement or voids

Careful assessment of the rubblized layer and other field conditions is critical and contributes to achieving an adequate paving platform. This conclusion has also been presented in previous studies. [*4, 24*] Current guidelines in the FDM and CMM, along with standard specification requirements, emphasize the importance of monitoring the rubblizing process and inspecting the rubblized layer. [*19, 20, 21*] These provisions should be strictly followed.

Test rolling of the rubblized layer.

- "Good"-performing projects: 2 of 11 (18 percent)
- "Poor"-performing projects: 1 of 8 (13 percent)

Test rolling with a loaded truck after rubblizing was not performed often in the analyzed projects. However, test rolling was performed for the entire rubblized layer on I-894 and contributed to the excellent performance of this pavement. Test rolling allowed project staff to identify areas of high deflection, indicating weak support. These areas were repaired prior to HMA paving. If the weak areas had not been identified and repaired, settlement and other damage to the pavement was likely to have occurred under traffic loading. It was specifically documented in the I-894 project diary that test rolling was beneficial in detecting weak areas.

Plan modifications enacted during construction if problems were noted during rubblizing.

- "Good"-performing projects: 7 of 11 (64 percent)
- "Poor"-performing projects: 4 of 8 (50 percent)

Changes to the plan if subgrade or rubblization problems were noted occurred at a slightly higher rate for projects in the "good" performance category. These changes included:

- Modification of the breaking pattern
- Placement of additional HMA thickness
- EBS and/or base patching
- Installation of additional drainage features
- Profile milling of lower HMA layers
- Addition of a CABC leveling layer

Modification of the breaking pattern was the most common adjustment if a rubblization problem was noted. EBS and/or base patching were also commonly employed. Drainage systems were change ordered on projects 8-Good, 4-Poor and 5-Poor. The drainage features likely improved the subgrade support but did not ultimately result in good performance for the latter two projects.

Paving additional HMA thickness in problem areas occurred on three of the good-performing projects. On projects 1- and 3-Good, a nominal thickness was added as a leveling layer in spot locations. On project 11-Good, an additional one inch of HMA was paved on the entire project. The latter remedy is a costly revision to plan, and has not been shown to appreciably extend the pavement system's service life. [*3*] Localized additions of HMA (e.g., ¼ inch thick) is a potentially valid treatment that could boost the structure of spotty areas with weak support.

Wet subgrade and/or rubblized layer noted during paving.

- "Good"-performing projects: 0 of 11
- "Poor"-performing projects: 3 of 8 (38 percent)

On three projects (1-, 5- and 8-Poor), HMA pavement was placed despite the presence of a wet paving platform. All three of these projects were categorized as having poor performance. These projects might have benefited from the installation of drainage systems.[3] Project 1-Poor stands out in particular, as the subgrade was determined to have a high moisture content during the pre-construction soils investigation. This problem was not addressed, and failure was noted one season after construction.

[3] Project 5-Poor had French drains installed, but not in all areas with wet conditions.

Heavy rain events occurred following rubblizing of projects 5- and 8-Poor. Wet conditions were noted in the exposed rubblized layer, and HMA pavement was placed in some areas before the base had adequate drying time.

Potential problem areas identified prior to construction and addressed prior to rubblization.

- "Good"-performing projects: 2 of 11 (18 percent)
- "Poor"-performing projects: 2 of 8 (25 percent)

Prior to construction, potential problem areas were identified for repair on four of the analyzed projects. These repairs included asphalt base patching, aggregate base patching, and drainage installation in known frost heave locations. There were not enough projects in the good performance category to conclude that this type of repair led to improved performance. However, if time and funding are available for pre-construction analysis of foundation support (using FWD analysis), and if weak areas are addressed, better performance would likely be noted. This type of monitoring was also suggested by Von Quintus et al. [*2*]

Moderate or severe concrete pavement joint and/or crack distress noted in PIF prior to rubblization.

- "Good"-performing projects: 4 of 11 (36 percent)
- "Poor"-performing projects: 4 of 8 (50 percent)

The level of concrete pavement joint/crack distress prior to rubblization was evaluated using the Department's network pavement performance evaluation. At the time that this study's pavements were rubblized, the PDI rating system was used.[4] The severity rating for distressed joints and cracks is described in Table 6. [*25, 26*]

Table 6. Distressed Concrete Pavement Joints and Cracks PDI Severity Rating Description

Severity Index	**Description**
1 (Slight/Low)	• Slight loss of material within the joint/crack • Distress in wheel path is 2 to 4 inches wide
2 (Moderate/ Medium)	• Moderate loss of material within the joint/crack • Slight effect on ride or safety • Distress in wheel path is 6 to 10 inches wide
3 (Severe/High)	• Significant breakup and loss of material within the joint/crack • Major effect on ride or safety • Frequent patching of the joint • Distress in wheel path is greater than 10 inches wide

[4] The Department currently evaluates pavements using the PCI rating index.

Inadequate HMA pavement crack sealing.

- "Good"-performing projects: 0 of 11
- "Poor"-performing projects: 5 of 8 (63 percent)

Adequacy of HMA pavement crack sealing was assessed during late winter 2011 site reviews. An inadequate rating denoted either complete lack of sealing, or sealing that had reopened, exposing the crack to water penetration.

Tenting of transverse cracks in HMA pavement.

- "Good"-performing projects: 1 of 11 (9 percent)
- "Poor"-performing projects: 6 of 8 (75 percent)

Tenting of transverse cracks in the HMA pavement was also evaluated during the late winter 2011 site reviews. The pavement in a tented crack had been pushed up on either side of the crack, sometimes to a height of an inch or more above the natural pavement grade. This type of distress resulted in extremely rough ride, and sometimes posed a safety hazard at high-speed travel. The presence of tented cracks is listed in Tables 4 and 5. Tented cracks classified as minor meant that the distress was just beginning to be apparent, and pavement ride was not significantly affected.

Discussion of joint/crack distress, HMA crack sealing, and tenting of transverse cracks

Half of the poor-performing projects had moderate or severe distress at concrete pavement joints and/or cracks prior to rubblization (3-, 4-, 6- and 8-Poor). Tented transverse cracks in the HMA pavement were later noted on all of these projects. Three of the projects did not have adequate sealing. A late winter 2011 field review was not conducted for the fourth project (8-Poor) to evaluate sealing.

Four of the projects in the good performance category had concrete pavement joint/crack distress (1-, 2-, 5- and 6-Good). Tented transverse cracks in the HMA pavement were later observed on one project (6-Good). This project did not have adequate sealing, and it was noted that acceptable performance would not continue without proper maintenance. Project 1-Good had minor tenting and was adequately crack sealed. The other projects (2- and 5-Good) did not have tenting, but transverse cracks had developed in 2010. These two projects were adequately crack sealed.

Based on the above observations, it is apparent that existing concrete pavement joint/crack condition, HMA crack sealing, and development of tented transverse cracks are related factors. It has been observed that transverse cracks develop in the HMA pavement at the location of previous distressed concrete pavement joints and cracks. Water seeps into the HMA crack, expands as it freezes in the winter months, and causes tenting of the HMA layer (as with projects 6-Good, 3-, 4- and 6-Poor). Adequate sealing of the HMA cracks aids in keeping the water out of the crack, limiting the likelihood of tenting (as with projects 2- and 5-Good).

Other projects had tented transverse cracks but were not reported as having distressed concrete pavement joints or cracks prior to rubblization (1-, 5- and 7-Poor). These projects are discussed below.

Project 1-Poor is not part of the Department's STN and pavement distress data is not collected. However, project plans noted that this pavement did have distressed concrete pavement joints, some of which were removed and aggregate base patched prior to paving. This project also had wet subgrade, which was the suspected primary cause of pavement failure.

No major joint/crack distress was recorded in the concrete pavement in project 5-Poor. Wet conditions were also noted during rubblizing and paving of this project, which suggests that trapped water plays a role in the tenting distress.

Project 7-Poor was originally a CRCP. This pavement had moderate to severe longitudinal joint distress prior to rubblization. Wide cracks (CRCP cracks open more than ¼ in) were also recorded. Some forensic investigation of the rubblized layer was performed when the project's HMA pavement was removed and replaced in 2010. Cracks were apparent in the rubblized layer (they had not been obliterated during rubblization). Loose, sandy material was noted in these cracks. This type of material would not be expected to provide the same level of support as the surrounding rubblized concrete, and this likely contributed to formation of the HMA cracking.

In summary, the root cause of tented transverse cracks in the HMA pavement was non-uniform support of the underlying rubblized layer. The failure mechanism was as follows: A poor-condition joint or crack resulted in a localized area of non-homogenous material within the rubblized layer. Reflective cracking in the HMA pavement was more likely to occur at this location of non-uniform support. Water seeped into the HMA crack, expanded under freezing temperatures, and pushed up the HMA pavement. The presence of trapped water also indicated that the rubblized layer did not provide adequate drainage.

6.22 Conclusions

The following bullet points provide conclusions based on results from the project-specific analysis described in Sections 6.1 through 6.18. Recommendations for design and construction practices are provided in Section 10.

- Close observation of the rubblizing process and inspection of the rubblized layer are critical to good performance. Documented excavation of test holes and test rolling are useful ways to evaluate the condition of the rubblized layer.
- If areas of weak support are noted during construction, the following modifications could result in better performance: modification of the breaking pattern, EBS and/or base patching, and spot placement of additional HMA thickness.
- If wet conditions exist (e.g., the subgrade has a high moisture content, there is a local high water table), drainage features could benefit the pavement system. If a significant rain event occurs while the rubblized layer is exposed, the material should be given time to thoroughly dry prior to paving.
- Tenting of HMA cracks, the most prevalent distress noted in poor-performing projects, was a result of non-uniform support at previous concrete pavement joint and crack locations.

Reflective cracks developed in the HMA pavement, and water seeped through to the rubblized layer. The water froze and expanded during the winter months, leading to tenting of the HMA cracks.

7. Overall Performance Analysis

In the Overall Performance Analysis, projects in the "good" performance category were compared as a group to projects in the "poor" performance category. This analysis was more quantitative in nature than the project-specific analysis presented in Section 6. Statistical comparisons were used as described below.

Analysis categories are presented as a series of questions relating each analyzed factor (e.g., rubblized layer thickness) to the ultimate rubblized pavement performance.

7.1 General Notes

<u>PDI, PCI and IRI Data</u>

To quantitatively compare the performance of "good" and "poor" pavements, the Department's PDI, PCI and IRI data were used. The former two indices measure the presence of distress in the pavement; individual distresses such as those described in Section 7.7 are measured and weighted to obtain the PDI or PCI value. PDI data were typically collected biennially between 1985 and 2008. PCI data, also measured biennially, were available for 2009 and 2010 and were converted to PDI for this analysis. The PDI scale is zero to 100, with the zero value representing a pavement with no distress.

A pavement's ride quality is evaluated using the IRI measurement. IRI data were also collected biennially and were available from 1990 to 2010. IRI values are reported in units of m/km.

For each in-depth study project, PDI and IRI data were compiled for both the rubblized pavement system and for the concrete pavement prior to rubblization. The values were plotted versus pavement age; these plots are available in Appendix 2 and Appendix 3.

<u>Statistical Comparisons</u>

For analysis categories where numerical data were evaluated (e.g., PDI, IRI, etc.), a test was performed to determine if there was a statistical difference between projects categorized as "good" and "poor." For this comparison, an unpaired t-test was used to obtain the two-tailed p-value. If the p-value was less than 0.05, the good and poor projects were considered statistically different (see Table 7).

Table 7. Parameters for Statistical Difference Test (t-test)

p-value	Statistical difference between "good" and "poor" projects?
> 0.05	No
< 0.05	Yes

7.2 Good Versus Poor Pavement Comparison

Question: Was there a statistical difference in PDI and IRI between pavements in the "good" and "poor" categories?

Answer: Yes

Projects for in-depth analysis were qualitatively categorized by WisDOT staff as having "good" or "poor" performance. PDI and IRI data were analyzed to determine if there was also a quantifiable difference among the "good" and "poor" pavements. Two pieces of information were investigated: (1) the PDI and IRI value for the pavement at age four, and (2) the rate of increase in overall PDI and IRI. These values are shown in Table 8. Age four was chosen for comparison because most pavements had been in service for at least four years at the time of analysis.[5] The rate of increase was determined by plotting PDI and IRI versus pavement age and determining the slope (rate) of a linear fit for the data. The PDI and IRI plots are provided in Appendix 2 and Appendix 3, respectively.

The PDI data were statistically different for "good" and "poor" projects (Table 8). The average age-four PDI values for good and poor pavements were 6.8 and 14.4, respectively, and the respective rates of increase were 2.1 and 5.0 PDI units per year. These data show that "poor" pavements had more distress after a relatively short time in service (four years), and that the extent and/or severity of distresses increased more quickly than for "good" pavements.

There was not a statistical difference in IRI between "good" and "poor" pavements (Table 8). The average IRI values at age 4 were very similar (1.01 and 0.93 m/km), as were the average rates of increase. This is likely due to the fact that the prevalent distress in the poor-performing pavements was transverse joint tenting, which occurred during the winter. IRI data is typically collected during the summer, when the tenting subsides.

A statistical difference in PDI was shown between the performance categories, demonstrating that the "good" and "poor" pavements have a quantitative difference as well as a subjective difference in performance. Further quantitative comparisons are therefore justified.

[5] Projects 10-Good and 4-Poor were three years old at the time of analysis; the most recent available data points were used in these cases.

Table 8. PDI and IRI Rate of Increase and Value at Age Four

Project	PDI		IRI	
	Age 4	**Rate**	**Age 4 (m/km)**	**Rate**
1 Good	13.7	3.16	1.00	0.034
2 Good	32.0	3.17	2.48	0.126
3 Good	8.1	2.03	1.10	0.044
4 Good	0.0	1.25	0.78	0.014
5 Good	2.3	1.29	0.93	0.046
6 Good	5.3	5.19	0.92	0.006
7 Good EB	0.0	0.00	1.12	0.091
7 Good WB	6.5	1.82	1.09	0.085
8 Good a	4.9	1.95	0.62	0.059
8 Good b	4.7	2.04	0.71	0.025
9 Good	8.2	2.11	0.84	0.000
10 Good	3.2	1.71	0.75	0.046
11 Good	0.0	1.76	0.76	0.048
1 Poor	†	†	†	†
2 Poor	15.0	7.50	1.05	0.026
3 Poor	11.0	2.99	1.31	0.123
4 Poor	4.3	4.34	0.72	*
5 Poor	19.8	6.87	0.94	0.125
6 Poor a	11.1	3.53	1.13	0.000
6 Poor b	10.7	3.50	0.78	0.000
7 Poor	17.5	4.66	0.78	0.047
8 Poor	25.9	6.64	0.87	0.000
Average (Good)	6.8	2.11	1.01	0.048
Average (Poor)	14.4	5.00	0.95	0.042
p-value	0.045	<0.001	0.734	0.783
Different?	Yes	Yes	No	No

Notes: † Distress data not available
* Insufficient data to calculate rate

7.3 Existing Concrete Pavement Type

Question: Does the existing concrete pavement type influence the ultimate performance of the rubblized pavement system?

Answer: No

Existing concrete pavement types are shown in Table 9 for the projects analyzed in this study. The type of pavement did not correlate with a project's categorization as "good" or "poor." Both performance categories had a similar number of each pavement type analyzed. Therefore, the concrete pavement type did not predict the ultimate performance of the rubblized system.

One thing to consider is that very few doweled JPCPs were analyzed in this study. The pavements that have been rubblized thus far in the state of Wisconsin were typically non-doweled; doweled JPCPs became the primary concrete pavement type during the mid-1980s. These doweled JPCPs will soon reach the end of their initial service life and will be future rubblization candidates. Their performance in rubblized systems should be evaluated when several years of data are available.

Table 9. Number of Concrete Pavement Types

Concrete Pavement Type	Good Projects	Poor Projects
JRCP	5	4
JPCP	3	2
JPCP-doweled	1	0
CRCP	2	2

7.4 Year of Existing Concrete Pavement Construction

Question: Does the year that the existing concrete pavement was constructed influence the ultimate performance of the rubblized pavement system?

Answer: No

The concrete pavement construction year was examined; a statistical difference could indicate that various construction practices or standards requirements influenced the behavior of the concrete pavement in a rubblized system. The concrete pavement construction years are reported in Table 10. No specific timeframe stood out in the "good" or "poor" categories, and no statistical difference was noted.

Table 10. Existing Concrete Pavement Construction Year

Project	Year	Project	Year
1 Good	1965	1 Poor	1965
2 Good	1962	2 Poor	1955
3 Good	1963	3 Poor	1965
4 Good	1932	4 Poor	1976
5 Good	1968	5 Poor	1965
6 Good	1978	6 Poor a	1970
7 Good EB	1969	6 Poor b	1972
7 Good WB	1969	7 Poor	1981
8 Good a	1972	8 Poor	1983
8 Good b	1975		
9 Good	1973		
10 Good	1973		
11 Good	1991		
	p-value	0.732	
	Different?	No	

7.5 Age of Concrete Pavement at Rubblization

Question: Does the age of the existing concrete pavement at the time of rubblization influence the ultimate performance of the rubblized pavement system?

Answer: No

The age of the concrete pavement at the time of rubblization was analyzed to evaluate the plausibility that an older pavement is not suitable for rubblization. The concrete pavement ages are reported in Table 11. The average age of the concrete pavement for the "good" and "poor" projects was essentially the same (34 and 35 years, respectively), and there was not a statistical difference between the categories. The range of pavement ages in the "good" category was 12 to 69 years, further demonstrating that a concrete pavement of any age has potential to perform well in a rubblized system.

Table 11. Concrete Pavement Age at Rubblization

Project	**Age**	**Project**	**Age**
1 Good	33	1 Poor	44
2 Good	37	2 Poor	47
3 Good	40	3 Poor	37
4 Good	69	4 Poor	31
5 Good	34	5 Poor	39
6 Good	25	6 Poor a	36
7 Good EB	36	6 Poor b	34
7 Good WB	36	7 Poor	21
8 Good a	28	8 Poor	23
8 Good b	30		
9 Good	32		
10 Good	34		
11 Good	12		
Average	34	*Average*	35
p-value		0.942	
Different?		No	

7.6 Overall Condition of Concrete Pavement

Question: Does the overall condition of the existing concrete pavement at the time of rubblization influence the ultimate performance of the rubblized pavement system?

Answer: No

The overall condition of the concrete pavement prior to rubblization was evaluated using the final PDI values available for each project as a concrete pavement surface. For most projects, this was the final year before rubblization. For projects that had been overlaid with HMA (9- and 10-Good; 6-Poor b), PDI data from the final year before overlay was used. PDI data were not available for projects 3-Good and 1-Poor.

An unpaired t-test was performed to determine if there was a statistical difference in final PDI between the good and poor projects. The final PDI value of each surveyed segment was used in the analysis; these data are available in Appendix 4. The p-value calculated was 0.556 (> 0.05), indicating no statistical difference in final concrete pavement PDI between the good and poor projects. Additionally, the average final PDI values for the good and poor projects were very similar: 59 and 56, respectively. Therefore the overall condition of the existing concrete pavement at the time of rubblization did not predict the ultimate performance of the rubblized system.

This result is expected, as a concrete pavement would not be a candidate for rubblization unless its overall condition (as measured by PDI) was poor. However, it is interesting to note that the average PDI values were slightly less than the minimum PDI threshold of 60 stated in the FDM, over which a pavement is considered a rubblization candidate.

7.7 Severity of Distresses Present in Existing Concrete Pavement

Question: Does the severity of a particular distress present in the existing concrete pavement at the time of rubblization influence the ultimate performance of the rubblized pavement system?

Answer: No

The final year of concrete pavement distress data was analyzed for each project. For most projects, this was the final year before rubblization. For projects that had been overlaid with HMA (9- and 10-Good; 6-Poor b), concrete pavement distress data was collected from the final year before overlay. Project 3-Good was not included because pavement distress data was not available for the final year its surface was concrete. Project 1-Poor was not included because pavement distress data is not routinely collected for highways off the STN.

Each concrete pavement distress (described below) was analyzed separately, and an average distress level was calculated for the entire length of each project. To quantify the data, PDI coefficients were then calculated for each project and each concrete pavement distress type. These coefficients are typically combined to calculate the single-value index (PDI) that rates overall pavement distress. However, for this analysis it was useful to look at the distress coefficients separately to rank the severity of each type of concrete pavement distress. A coefficient of 1.0 indicated no distress present; the closer the coefficient was to zero, the more severe and extensive the distress. The coefficients are reported in Table 12.

Concrete pavement distresses analyzed:

- Slab breakup - fracturing of a slab due to crack development
- Distressed joints or cracks - any distress within two feet of the joint or crack
- Patching - presence and condition of patches are rated

- Surface distress - spalling, scaling, chipping, etc.
- Longitudinal joint distress - faulting, and any distress within two feet of the joint or crack
- Transverse joint faulting
- Wide cracks - any crack greater than ¼-inch in width
- CRCP distresses:
    - Punch-outs
    - Pavement deterioration
    - Patching
    - Delamination of steel

Within each concrete pavement distress category, the coefficients for "good" and "poor" projects were compared using an unpaired t-test. The p-values are reported in Table 12. For all but one concrete pavement distress category, no statistical difference was shown between "good" and "poor" projects. This indicated that the severity of a particular distress in the existing concrete pavement to be rubblized did not influence the ultimate performance of the rubblized pavement system.

The only distress category that showed statistical difference between good and poor projects was CRCP deterioration. CRCPs in the "good" category had more deterioration than those in the "poor" category. This is counterintuitive, as less pavement deterioration logically should not lead to worse performance. However, there were only four CRCPs in the analysis, so a strong conclusion cannot be drawn from this particular pavement distress.

Based on the discussion in Section 6.21, it was expected that a statistical difference would exist between performance categories for distressed joints and/or cracks. It was hypothesized in Section 6.21 that joint and crack deterioration is a factor leading to the occurrence of tented transverse cracks in the HMA layer. However, three of the four "good"-performing projects with distressed joints/cracks also had adequate HMA crack maintenance, which likely prevented or delayed tenting.

Table 12. Statistical Comparison of Good and Poor Projects Using Concrete Pavement Distress Coefficients

Project	Slab Breakup	Distressed Jts./Crks.	Patching	Surface Distress	Long. Jt. Distress	Transverse Faulting	Wide Cracks	Punch-Outs	Pavement Deterioration	%Patching CRCP	Delamination
1 Good	0.875	0.423	0.867	1.0	0.760	1.0	1.0	N/A	N/A	N/A	N/A
2 Good	0.876	0.250	0.448	1.0	0.760	1.0	1.0	N/A	N/A	N/A	N/A
3 Good	†	†	†	†	†	†	†	†	†	†	†
4 Good	0.836	0.917	1.0	1.0	1.0	0.902	1.0	N/A	N/A	N/A	N/A
5 Good	0.840	0.614	0.943	0.922	1.0	0.939	1.0	N/A	N/A	N/A	N/A
6 Good	1.0	0.492	0.942	1.0	1.0	0.966	1.0	N/A	N/A	N/A	N/A
7 Good EB	0.848	0.933	0.814	1.0	1.0	0.939	1.0	N/A	N/A	N/A	N/A
7 Good WB	0.850	1.0	0.814	1.0	1.0	0.966	1.0	N/A	N/A	N/A	N/A
8 Good a	0.814	0.820	0.814	0.972	0.954	0.939	1.0	N/A	N/A	N/A	N/A
8 Good b	0.989	1.0	1.0	1.0	0.984	0.966	1.0	N/A	N/A	N/A	N/A
9 Good*	N/A	1.0	N/A	0.972	0.954	1.0	0.900	No	0.495	0.496	Yes
10 Good*	N/A	1.0	N/A	0.928	0.989	1.0	0.850	No	0.558	0.393	Yes
11 Good	0.790	0.785	0.985	1.0	1.0	0.966	1.0	N/A	N/A	N/A	N/A
1 Poor	†	†	†	†	†	†	†	†	†	†	†
2 Poor	0.863	0.835	0.683	1.0	0.861	0.902	1.0	N/A	N/A	N/A	N/A
3 Poor	0.614	0.423	0.579	0.884	1.0	0.966	1.0	N/A	N/A	N/A	N/A
4 Poor	1.0	0.649	0.638	1.0	1.0	0.891	1.0	N/A	N/A	N/A	N/A
5 Poor	0.849	1.0	0.748	0.942	1.0	0.966	1.0	N/A	N/A	N/A	N/A
6 Poor a	0.907	0.805	0.914	1.0	1.0	0.966	1.0	N/A	N/A	N/A	N/A
6 Poor b*	N/A	1.0	N/A	0.972	0.954	1.0	0.920	No	0.807	0.960	Yes
7 Poor	N/A	0.884	N/A	0.972	0.861	0.966	0.800	No	0.820	0.388	Yes
8 Poor	0.989	0.881	0.963	1.0	1.0	0.939	1.0	N/A	N/A	N/A	N/A
p-value	0.980	0.715	0.213	0.466	0.801	0.319	0.609	‡	0.012	0.512	‡
Different?	No	No	No	No	No	No	No		Yes	No	

Notes: *Concrete pavement distress level prior to HMA overlay
† Distress data not available
‡ p-value not calculated
N/A - Distress not applicable for this project's pavement type

7.8 Ride Quality of Concrete Pavement

Question: Does the ride quality of the existing concrete pavement at the time of rubblization influence the ultimate performance of the rubblized pavement system?

Answer: No

The difference in ride quality for good and poor projects was evaluated using IRI data of the concrete pavement prior to overlay. For most projects, IRI data was taken from the final year before rubblization. These data are presented in Appendix 5. For projects that had been overlaid with HMA (9- and 10-Good; 6-Poor b), data from the final year before overlay were used. IRI data were not available for projects 3-Good and 1-Poor.

An unpaired t-test was used to check for a statistical difference in pre-rubblization IRI between the good and poor projects. The p-value calculated was 0.128 (> 0.05), indicating no statistical difference in IRI between the good and poor projects. Therefore, the ride quality of the existing concrete pavement at the time of rubblization was not an indicator of the ultimate performance of the rubblized system. The average final IRI value for the good projects (2.34 m/km) was slightly higher than the average IRI for the poor projects (2.10 m/km).

7.9 Thickness of Rubblized Pavement Layer

Question: Does the thickness of the rubblized concrete pavement influence the ultimate performance of the rubblized pavement system?

Answer: No

The thickness of the rubblized concrete pavement determines that layer's contribution to the system's total structural number (SN), so rubblized pavement thickness is accounted for in the pavement design phase. However, it is possible that the existing concrete pavement thickness affects the quality of the rubblization and thus influences the eventual pavement performance. This hypothesis was tested herein.

The thicknesses of each project's rubblized concrete pavement layer are shown in Table 13. For some projects, the concrete pavement was removed partial depth (1 to 2 inches) prior to rubblization. The final rubblized layer thickness is shown here.

Thicknesses ranged from 6 to 10 inches and averaged approximately 8.5 inches for both "good" and "poor" projects. This is a typical range for concrete pavement thickness in Wisconsin. No statistical difference in thickness was shown between pavements with "good" and "poor" performance. Thickness of the rubblized concrete pavement is therefore not a likely factor in the ultimate performance of rubblized pavement systems.

Table 13. Rubblized Concrete Pavement Thickness (in)

Project	Thk.	Project	Thk.
1 Good	9	1 Poor	7
2 Good	8	2 Poor	7
3 Good	8	3 Poor	9
4 Good	6	4 Poor	10
5 Good	9	5 Poor	9
6 Good	10	6 Poor a	8
7 Good EB	9	6 Poor b	8
7 Good WB	9	7 Poor	8
8 Good a	9	8 Poor	9
8 Good b	8		
9 Good	8		
10 Good	8		
11 Good	9		
Average	8.5	*Average*	8.3
p-value		0.766	
Different?		No	

7.10 Base and Subbase Properties

Question: Does the thickness and type of the base and subbase layers influence the ultimate performance of the rubblized pavement system?

Answer: No

The existing base and subbase layer(s) for each analyzed project are summarized in Table 14. The average thickness of the existing base layer(s) was 6.5 inches for both the "good" and "poor" category projects. Project 4-Good had no base (concrete paved directly on subgrade) and still had good performance. Therefore the thickness of the existing base layer was not a critical factor in the ultimate rubblized pavement performance.

Likewise, the type of material used in the base was similar for both "good" and "poor" projects. Asphalt-stabilized base material is not commonly used in the state, but four projects analyzed here included asphaltic material. Two were good-performing projects and two had poor performance.

The presence of an existing subbase layer did not correlate with good or poor performance. Approximately half of the projects in each category had an existing subbase layer.

Table 14. Existing Base and Subbase Layer Information

Project	Base		Subbase	
	Thickness (in)	Type	Thickness (in)	Type
1 Good	6	CABC	6	Granular
2 Good	6	G or CS		
3 Good	8	CABC		
4 Good	None			
5 Good	6	CABC	6	Granular
6 Good	9	CABC		
7 Good EB	6	G	9	Granular
7 Good WB	6	G	9	Granular
8 Good a	9	CABC		
8 Good b	6	CABC		
9 Good	2.5 6	ASBC CABC	9	Granular
10 Good	2.5 6	ASBC CABC	9	Granular
11 Good	6	CABC		
1 Poor	6	G or CS		
2 Poor	3	CS		
3 Poor	4 8	SAB CS	6	LS
4 Poor	6	CABC		
5 Poor	6	G or CS	9	Granular
6 Poor a	6	CABC	9	Granular
6 Poor b	2.5 6	ASBC CABC	9	Granular
7 Poor	6	CABC	18-30	SB
8 Poor	6	CABC		

Abbreviations: CABC - Crushed aggregate base course
G - Gravel
CS - Crushed stone
ASBC - Asphalt stabilized base course
SAB - Single aggregate bituminous
LS - Lime stabilized subbase
SB - Select borrow

7.11 Underlying Soil Properties

Question: Does the native soil type and properties influence the ultimate performance of the rubblized pavement system?

Answer: Possibly

Prior to most Wisconsin pavement designs, a soils analysis is conducted to establish the condition and properties of the native soils. A summary of the prevailing soil types (e.g., silt, clay, sand, etc.) is provided, along with the following design parameters: modulus of subgrade reaction (k), DGI, SSV, and the frost index. The latter three parameters are used in HMA pavement design. These parameters and the projects' soil types are shown in Table 15.

The SSV and DGI measure the ability of a soil to support traffic loadings. A higher SSV indicates a higher quality subgrade, while DGI decreases with increasing soils quality. The DGI values correlated somewhat with location in the state, with higher values reported in the south and east and lower values in the north and west. However, the DGI and SSV for the study projects' soils were similar for both performance categories (see average values in Table 15). No statistical difference was shown between the two performance groups; therefore, DGI and SSV are not indicators of future performance.

Current WisDOT rubblization guidance states that difficulty with rubblization could be encountered when the DGI is greater than 12. [*19*] However, four of the eleven projects in the "good" performance category had a DGI of 14 (Table 15). It is therefore possible to achieve good rubblized performance with a DGI of 14.

The frost index indicates a soil's susceptibility to frost action, with F-1 soils being least susceptible and F-4 being most susceptible. Most of the soils in the projects analyzed were categorized as F-3 soils (Table 15). The frost index was not an indicator of performance for these pavements.

The prevailing native soils did not show a strong correlation with rubblized project performance, but one general trend was noted. Subgrades are ideally composed of well-drained soils (e.g., sand, gravel), which provide a stable, solid base for rubblization. Projects with well-drained soils were identified in both pavement categories. However, of the projects for which soils information was available, a greater percentage of projects in the "good" category had a coarser grained composition: 91 percent, versus 57 percent of projects in the "poor" category. This indicates that a coarse-grained subgrade is more likely to have good performance in a rubblized system. On the other end of the spectrum, poorly drained, fine-grained soils (clays and silts) were also present in the subgrades of "good"-performing pavements. Therefore, while a well-drained subgrade could increase the likelihood of good performance, it is possible to achieve an adequate rubblized base with a range of subgrade soil types. Subgrade drainage is further discussed in Section 7.14.

Table 15. Native Soil Properties

Project	DGI	SSV	Frost Index	Prevailing Native Soils
1 Good	14	3.8	F-3	Dense silty sand and gravel
2 Good	12	4.2	F-3	Silty sand and gravel
3 Good	14	4.0	F-3	Clay and silt; varying amounts of sand and gravel
4 Good	12	4.3	F-3	Well-drained sandy loam or loam; silty sand with traces of clay
5 Good	14	4.0	F-3	Well-drained fine sandy loam to silt loam soils that become clay-like at a depth of 2-3 ft
6 Good	12	4.3	*	Well-drained to moderately well-drained loams and silt loams
7 Good	12	4.2	*	Sands and gravel to silts and clays
8 Good	14	4.0	F-3	Sands with some silt, sandy silts and shallow bedrock
9 Good	10	4.5	F-3	Sandy loams and silt loams
10 Good	10	4.5	F-3	Sandy loams and silt loams
11 Good	4	5.3	F-2	Silty sands and sands with a trace of gravel
Average	*11.7*	*4.2*		
1 Poor	10	4.5	F-3	Silty clays with 17% moisture content
2 Poor	12	4.2	F-3	Loams, silt loams and sandy loams
3 Poor	12	4.2	F-3	Silt loam
4 Poor	10	4.5	*	Well-drained and moderately well-drained silty clay loam to sandy clay loam subsoil over sandy loam glacial till
5 Poor	12	4.2	*	Silt loam; water table 1-5 ft
6 Poor	10	4.5	F-3	*
7 Poor	18	3.3	F-4	Sandy, underlain by clay materials
8 Poor	4	5.3	F-2	Sands with a trace of silt and some gravel
Average	*10.9*	*4.4*		
p-value	0.585	0.573		
Different?	No	No		

* Information not available

7.12 Thickness of HMA Pavement

Question: Does the thickness of the HMA pavement influence the ultimate performance of the rubblized pavement system?

Answer: Possibly

As discussed above for rubblized layer thickness, the HMA pavement thickness is determined in the design phase such that the required SN is achieved. Theoretically, a pavement with this calculated design HMA thickness should meet the structural requirement and exhibit adequate performance for the intended service life. It is of interest, however, to compare "good" and "poor" project HMA thicknesses to evaluate whether that parameter affected the ultimate pavement performance.

The study projects' HMA pavement thicknesses are shown in Table 16. The design thickness for project 11-Good (USH 8, Oneida County) was 4.5 inches; an additional inch of HMA was added during construction. The average paved HMA thicknesses for pavements in the "good" and "poor" categories were similar (5.6 and 5.8 inches, respectively), and there was no statistical difference between the two performance categories. These results indicate that the final HMA thickness did not directly influence the ultimate performance of the rubblized pavements in this study.

Previous research and experience has proposed minimum HMA thickness thresholds for rubblized systems. The most recent suggested minimum thickness for Wisconsin pavements is 4 inches. [*2*] The NAPA guidelines for minimum HMA thickness (based on traffic level) are shown in Table 17.

All pavements in the study meet the suggested 4-in minimum HMA thickness for Wisconsin pavements. Several pavements were thinner than the NAPA suggested minimums. Projects 2- and 6-Good had thicknesses less than 5 inches; however, the ESAL levels for these pavements were relatively low (less than 2 million ESALs). Projects 2- and 5-Poor also had thicknesses less than 5 inches, but these projects' ESAL levels were higher (4.5 and 2.7 million ESALs, respectively). It is possible that additional HMA thickness would have benefited these pavement systems.

Table 16. HMA Pavement Thickness and ESAL Level

Project	Thickness (in)	Design ESALs	Project	Thickness (in)	Design ESALs
1 Good	7	2,979,130	1 Poor	6	4,690,980
2 Good	4	1,737,400	2 Poor	4.5	4,489,500
3 Good	5.25	15,972,400	3 Poor	6.5	5,007,800
4 Good	5	671,600	4 Poor	6	8,511,800
5 Good	6.5	4,416,500	5 Poor	4.5	2,667,237
6 Good	4.75	722,700	6 Poor	7	6,132,000
7 Good	5.5	7,679,600	7 Poor	5	2,284,900
8 Good	5	3,577,000	8 Poor	7	6,066,300
9 Good	6.5	4,547,900			
10 Good	6.5	4,547,900			
11 Good	5.5	1,314,000			
Average	5.6		*Average*	5.8	

Note: HMA thickness p-value = 0.629 (not statistically different)

Table 17. NAPA Suggested Minimum HMA Thickness

ESALs x 10^6	Minimum HMA Thickness (in) [*27*]
< 5	5
5 - 10	6
> 10	7

7.13 Time of Rubblization

Question: Does the time of year the pavement was rubblized affect ultimate pavement performance?

Answer: No

The month and year that rubblization took place is shown for each project in Table 18. The majority of pavements in the "good" performance category were rubblized in July and August, while the majority of pavements in the "poor" performance category were rubblized in June. Subgrade support is typically lowest during the spring thaw period, which in Wisconsin generally occurs during the months of March, April and May. Pavements rubblized in June were therefore likely not affected by spring thaw conditions. However, wet conditions could result in poor subgrade support at any time of the year.

Table 18. Month and Year of Project Rubblization

Project	Time of Rubblization		Project	Time of Rubblization	
1 Good	July	1998	1 Poor	Jun/Jul	2009
2 Good	August	1999	2 Poor	June	2002
3 Good	Apr/May/Jun	2003	3 Poor	August	2002
4 Good	September	2000	4 Poor	Jun/Jul/Aug	2007
5 Good	July	2002	5 Poor	Jun/Jul/Aug	2004
6 Good	June	2003	6 Poor	*	2006
7 Good	September	2005	7 Poor	June	2002
8 Good	July	2000	8 Poor	May	2006
9 Good	August	2005			
10 Good	May	2007			
11 Good	August	2003			

Summary	Good	Poor
April	1	0
May	2	1
June	2	5
July	3	3
August	3	3
September	2	0

* Information not available

7.14 Subgrade Drainage

Question: Does inclusion of a subgrade drainage system determine how the rubblized pavement system will perform?

Answer: In some cases

The inclusion of subgrade drainage systems for the analysis projects is reported in Table 19. In both the "good" and "poor" performance categories, just over half of the projects had some type of subgrade drainage system.

Projects for which a drainage system was included tended to have subgrades with silt and clay, whereas sand or gravel subgrades did not have drainage included (reference Table 15). In some projects, drainage was only specified at known problem areas (8-Good, 11-Good, and 8-Poor). These practices are consistent with what is recommended in WisDOT guidelines (Section 4).

In the case of project 4-Poor, edge drains were installed as a change order after initial rubblization attempts resulted in a questionable paving platform. The subgrade in this area was classified as well-drained (Table 15). While the drainage measures likely improved the rubblized material quality, they did not solve the ultimate problem which led to poor performance.

The above discussion shows that, in some cases, subgrade drainage is likely a necessary component for good rubblized pavement performance. However, drained systems did not guarantee good performance in the projects investigated in this study. Current FDM recommendations are adequate (i.e., drainage might help remove water when low-permeability soils are present).

Table 19. Subgrade Drainage System

Project	Drainage	Project	Drainage
1 Good	Yes	1 Poor	Yes*
2 Good	No	2 Poor	No
3 Good	Yes*	3 Poor	No
4 Good	No	4 Poor	Yes
5 Good	No	5 Poor	No
6 Good	No	6 Poor	Yes
7 Good	No	7 Poor	Yes
8 Good	Yes †	8 Poor	Yes †
9 Good	Yes		
10 Good	Yes		
11 Good	Yes †		

*Existing prior to rubblization

†At low points and/or frost heave areas

7.15 Warranted HMA Pavement

Question: Does construction of the HMA pavement under warranty influence the ultimate performance of the rubblized pavement system?

Answer: No

When a Wisconsin STN pavement is constructed under warranty, the contractor selects the materials and methods for HMA construction (e.g. mixture type, gradation, layer thicknesses, [6] etc.) and guarantees the pavement performance for five years. Slightly less than half of the in-depth study pavements were constructed under warranty (see Table 20). Four performed well, and four were in the poor-performing category; it is therefore unlikely that specification of warranted HMA pavement results in good or poor performance of rubblized systems.

[6] The total HMA pavement thickness is designed by the Department.

Table 20. Warranted and Non-Warranted Pavements

Performance Category	Non-warranted	Warranted
Good	7	4
Poor	4	4

7.16 Transverse Crack Development

Question: Is the performance of the rubblized pavement system related to the time that transverse cracks developed?

Answer: Yes

Pavement distress data (PDI and PCI) were analyzed to determine when transverse cracks developed in the HMA pavement. The age of the HMA pavement when transverse cracks were recorded in at least half of the analyzed survey segments was determined and is reported for each analysis project in Table 21.

The average age at which transverse cracks developed in at least half of the HMA pavement area was 6 and 4 years for "good" and "poor" projects, respectively. The statistical analysis did not show a difference between the two performance categories, but the p-value in this example approached the threshold ($p < 0.05$) for statistical difference. In general, transverse crack development occurred later in the "good" category pavements: 9 or 10 years in several cases. It is therefore concluded that early transverse crack development was related to future poor performance.

Table 21. Pavement Age at Time of Transverse Crack Development

Project	Age	Project	Age
1 Good	10	1 Poor	†
2 Good	3	2 Poor	8
3 Good	5	3 Poor	6
4 Good	9	4 Poor	2
5 Good	6	5 Poor	3
6 Good	5	6 Poor	3
7 Good	4	7 Poor	3
8 Good	9	8 Poor	4
9 Good	4		
10 Good	*		
11 Good	7		
Average	6	*Average*	4
p-value		0.092	
Different?		No	

* No transverse cracks at time of study

† Distress data not available

A comparison between pavement age at the time of transverse crack development and HMA overlay thickness as suggested by NAPA (see Section 7.12) is also noteworthy. For the 17 HMA pavements where transverse cracking age was available (Table 21), 7 were designed thicker than NAPA guidelines, 5 were equal to, and 5 were thinner than NAPA guidelines (see Tables 16 and 17). The average age at which transverse cracks developed in each of these categories is presented in Table 22. On average, transverse cracking occurred one year earlier in pavements that were thinner than NAPA guidelines. [*27*]

Table 22. Transverse Crack Development as Related to HMA Pavement Thickness

HMA Thickness Relative to NAPA Guidelines [*27*]	**Average Age of Transverse Crack Development (Years)**
Thicker 1-Good, 5-Good, 9-Good, 11-Good, 3-Poor, 6-Poor, 8-Poor	5.7
Equal 4-Good, 6-Good, 8-Good, 4-Poor, 7-Poor	5.6
Thinner 2-Good, 3-Good, 7-Good, 2-Poor, 5-Poor	4.6

7.17 Traffic Considerations

Question: Is the performance of the rubblized pavement system related to the roadway's predicted traffic volume?

Answer: No

To determine if roadway volume factors had any influence on rubblized pavement performance, design ESALs and truck percentages were analyzed for the in-depth study pavements. These figures are shown in Table 23. Roadways with good performance had ESAL levels ranging from under 1 million to over 15 million. The average ESAL level was about 5 million for pavements in both performance categories. No statistical difference was shown for design ESAL level.

Similarly, the truck traffic percentage did not correlate with "good" and "poor" performance. The average truck percentage was slightly higher for pavements in the poor category, but there was no statistical difference between the performance categories.

Table 23. Traffic Information

Project	Design ESALs	Percent Trucks
1 Good	2,979,130	8.3
2 Good	1,737,400	9.0
3 Good	15,972,400	9.0
4 Good	671,600	5.7
5 Good	4,416,500	14.2
6 Good	722,700	4.9
7 Good	7,679,600	4.6
8 Good	3,577,000	13.1
9 Good	4,547,900	11.6
10 Good	4,547,900	11.6
11 Good	1,314,000	10.5
Average	*4,571,000*	*9.2*
1 Poor	4,690,980	6.9
2 Poor	4,489,500	7.9
3 Poor	5,007,800	12.0
4 Poor	8,511,800	17.4
5 Poor	2,667,237	6.0
6 Poor	6,132,000	11.6
7 Poor	2,284,900	13.5
8 Poor	6,066,300	10.6
Average	*5,109,200*	*10.8*
p-value	0.719	0.307
Different?	No	No

7.18 Functional Classification

Question: Is the performance of the rubblized pavement system related to the roadway's functional classification?

Answer: Yes (rural versus urban)

Wisconsin highways are grouped into classes according to the "character of service they are intended to provide." [*28*] Roadways are first categorized by population: an area with greater than 5,000 people is designated as urban, and an area is rural if its population is fewer than 5,000. The highway's functional classification is then defined based on its intended use. These classifications are shown in Table 24 for this study's roadways.

Two of the pavements in the "poor" performance category had both rural and urban sections. For this analysis, they were considered urban projects, as most of the project lengths were within urban limits. The percentage of urban projects in the "poor" performance category was higher than in the "good" category (63 versus 36 percent, respectively). Urban projects can be more challenging from a construction standpoint than rural projects. Urban areas often have constricted work zones and complicated staging, limited construction work time periods, and utility structures to avoid. It is therefore possible that urban construction complications were a factor in premature distress formation in the poor-performing pavements.

The majority of projects were functionally classified as principal arterials (rural or urban). In this study, a roadway's classification had no relationship to ultimate performance.

Table 24. Functional Classification

Project	Rural/ Urban	Functional Classification
1 Good	R	Principal arterial
2 Good	U	Minor arterial
3 Good	U	Principal arterial - Interstate
4 Good	R	Minor arterial
5 Good	R	Principal arterial
6 Good	U	Principal arterial
7 Good	U	Principal arterial
8 Good	R	Principal arterial
9 Good	R	Principal arterial
10 Good	R	Principal arterial
11 Good	R	Principal arterial
1 Poor	U	Principal arterial
2 Poor	U	Principal arterial
3 Poor	U	Principal arterial
4 Poor	R	Principal arterial - Interstate
5 Poor	R/U	Minor arterial
6 Poor	R/U	Principal arterial
7 Poor	R	Principal arterial
8 Poor	R	Principal arterial

7.19 Construction Conditions

Question: Does adjacent traffic during rubblization and/or traffic on lower HMA layers affect the performance of the rubblized pavement system?

Answer: No

An ideal work zone is one where the roadway is completely closed to traffic, either through a detour or with a crossover on a 4-lane divided highway. This allows construction to progress undisturbed by live traffic. This is not always possible, however, and rubblization often occurs with live traffic in an adjacent lane. The construction zone conditions for the projects analyzed in this study are shown in Table 25. Ten out of the eleven projects in the "good" performance category were rubblized with adjacent traffic. This demonstrates that adequate rubblization can be achieved under the more demanding live traffic construction situation.

HMA is typically paved in several thin layers until the planned pavement thickness is completed. While it is ideal to have the full thickness paved prior to traffic loading, travel lanes are often opened before all pavement layers have been placed. The pavement opening conditions for this study's projects are shown in Table 25. For these projects, traffic flowing on the lower layer(s) of HMA did not necessarily result in poor pavement performance: over half of the pavements in the "good" performance category had traffic loading prior to paving of the full HMA thickness.

Table 25. Construction Conditions

Project	**Work zone**	**Traffic prior to surface layer?**
1 Good	A	Yes
2 Good	A	Yes
3 Good	A	Yes
4 Good	A	Yes
5 Good	A	Yes
6 Good	A	Yes
7 Good	A	Yes
8 Good	F-D	No
9 Good	A	No
10 Good	A	No
11 Good	A	Yes
1 Poor	F-C	Yes
2 Poor	A	No
3 Poor	A	Yes
4 Poor	F-C	No
5 Poor	F-C	No
6 Poor	A	No
7 Poor	A	No
8 Poor	F-C	No

A: Adjacent live traffic lane during rubblization

F-D: Full closure with detour

F-C: Full closure with crossover

7.20 Summary

In the overall project analysis, 16 factors were reviewed that could potentially affect ultimate performance of the rubblized pavement system. Statistical analysis was used where possible to determine if a factor resulted in a difference between "good" and "poor" performance categories.

The 16 factors are listed in Table 26, along with whether each factor was determined to be related to pavement performance. For the 19 projects reviewed in this study, most of the analyzed factors were not found to be directly related to pavement performance.

Table 26. Factors Related to Rubblized Pavement Performance

Factor	Related to Performance
Existing concrete pavement type	No
Year of existing concrete pavement construction	No
Age of existing concrete pavement	No
Overall condition of existing concrete pavement (PDI)	No
Severity of distresses present in existing concrete pavement	No
Ride quality of existing concrete pavement	No
Thickness of rubblized pavement layer	No
Existing base and subbase properties	No
Native soil properties	Possibly
Thickness of HMA pavement	Possibly
Time of rubblization	No
Subgrade drainage	Possibly
Warranted HMA pavement	No
Transverse crack development	Yes
Traffic considerations (ESALs, truck percentage)	No
Functional classification (rural vs. urban)	Yes
Construction conditions	No

7.21 Conclusions

The following bullet points provide conclusions based on results from the overall project analysis described in Sections 7.3 through 7.19. Recommendations for design and construction practices are provided in Section 10.

- The following factors were not directly related to ultimate pavement performance for the analyzed pavements:
    - Construction year, age, thickness, pavement type and overall condition of the existing concrete pavement
    - Base and subbase properties
    - Traffic levels and construction conditions

- Severity of existing concrete distresses were not related to performance, with the exception of joint and/or crack condition, as discussed in Section 6.21.
- The month that rubblization took place was not related to performance for the analyzed studies, but rubblizing during the spring thaw should be avoided.
- Use of an HMA pavement warranty was not found to be related to ultimate pavement performance. However, it is important that the inspection and acceptance of the rubblized pavement be given adequate attention in a warranted HMA pavement situation, as the rubblized material itself is not covered by the warranty.
- Two projects in the poor performance category might have benefited from additional HMA thickness.
- Rubblized systems were more likely to have good performance when the subgrade had a sandy composition.
- Inclusion of subgrade drainage systems likely benefited some projects but did not guarantee good performance.
- Current FDM guidelines state that difficulty with rubblization could be encountered when the DGI is greater than 12. However, several projects in the good performance category had DGI values of 14.
- Transverse cracks in the HMA developed more quickly in pavements in the "poor" performance category.
- Pavements in the "poor" performance category were more likely to be in urban settings.

8. Design Considerations

8.1 Pavement Service Life

Pavement service life is an important factor in the planning and design of a roadway's rehabilitation schedule. The initial service life is the number of years a pavement is expected to perform with routine maintenance before requiring rehabilitation (e.g., HMA overlay or mill and overlay). The current initial service life used for rubblized pavement systems in Wisconsin is 22 years. The initial service life for a traditional HMA pavement is 18 years; the rubblized pavement service life is assigned a 25 percent increase because the rubblized material is considered a drained layer. [*29*]

Trends in pavement distress data are often used to predict a pavement's condition over time and thereby estimate its initial service life. PDI data for pavements in the "good" and "poor" performance categories were plotted versus pavement age, and a series of trends were fit to forecast the pavement distress level. At the time of this study's distress analysis (2010), the pavements ranged in age from 1 to 12 years.

Three predictive fits were evaluated: linear, second-order polynomial, and exponential as developed by Von Quintus, et. al. [*2*] These curves are described in Equations 1, 2, and 3, respectively.

$$PDI = at + b \qquad \text{Equation 1}$$

$$PDI = at^2 + bt + c \qquad \text{Equation 2}$$

$$PDI = 100\left[1 - e^{-a\left(\frac{t}{t_{des}}\right)^b}\right] \qquad \text{Equation 3}$$

Where: t = pavement age (years)
t_{des} = pavement design life = 22 years
a, b, c = regression constants

PDI data and the predictive trends are plotted in Figures 6 and 7 for pavements in the "good" and "poor" performance categories, respectively. Regression constant values and goodness of fit (R^2) for each trend are also provided in Figures 6 and 7. Equation 2 (polynomial fit) did not correlate well with PDI data in the "poor" performance category and is not presented.

For this evaluation, a PDI threshold of 65 was defined as the point at which a pavement is at the end of its initial service life. This PDI value would be reached with the presence of 11 or more severe transverse cracks per tenth-mile survey segment in the HMA pavement, which was the prevalent distress noted in the "poor" performance category projects.

Based on the PDI threshold of 65 and the trends shown in Figure 6, the initial service life of rubblized pavement systems with good performance ranges from 17 to 25 years. Pavements in the "poor" performance category may only be serviceable for 13 to 15 years (Figure 7). The 17 to 25 year estimate for good-performing pavements is on par with the previous service life prediction for Wisconsin pavements of 21 years by Von Quintus, et al. [*2*] Current WisDOT design values of 22 years for HMA over rubblized pavement and 18 years for traditional HMA pavements fall within the 17 to 25 year range. [*29*]

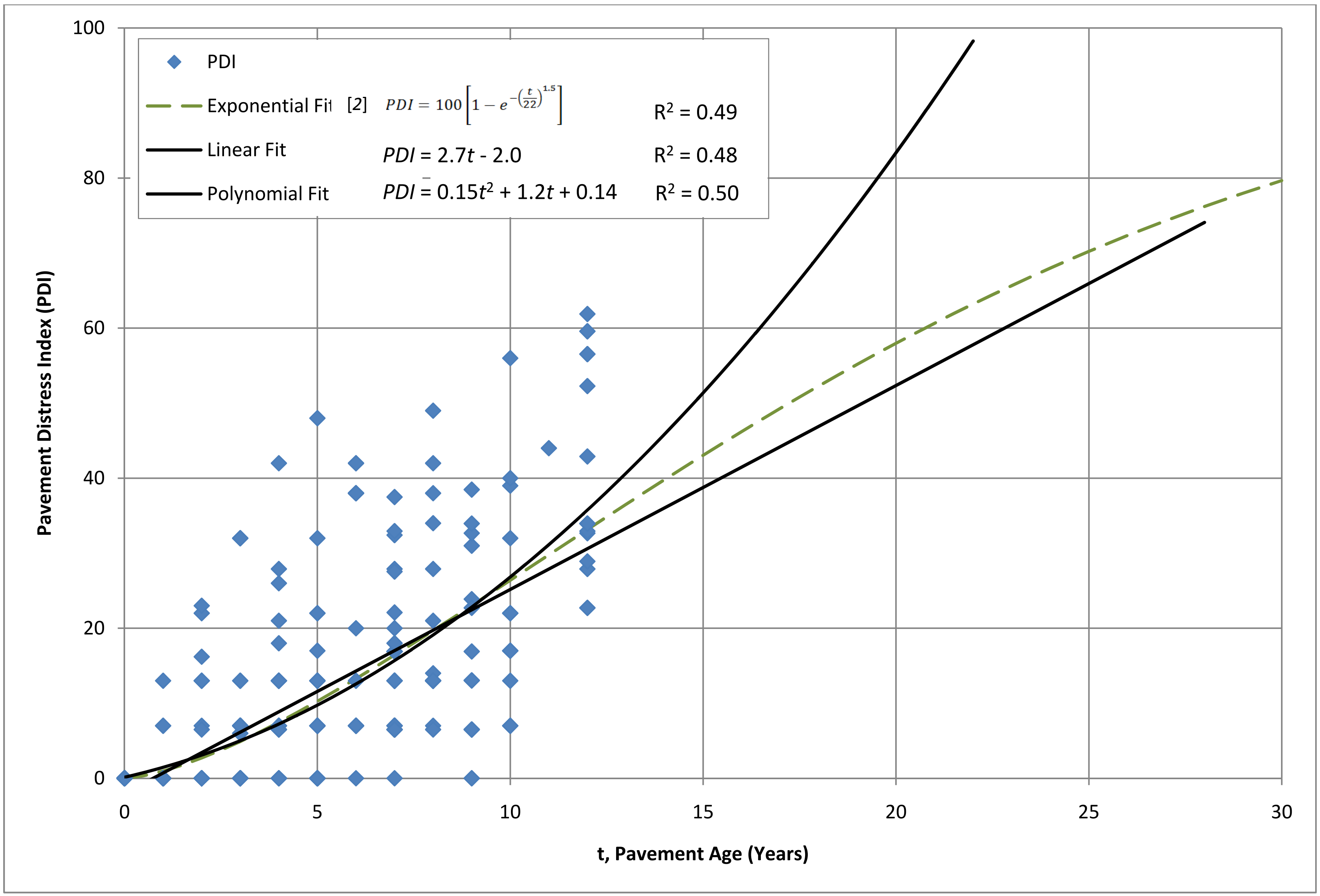

Figure 6. Pavement distress prediction, "good" performance category pavements

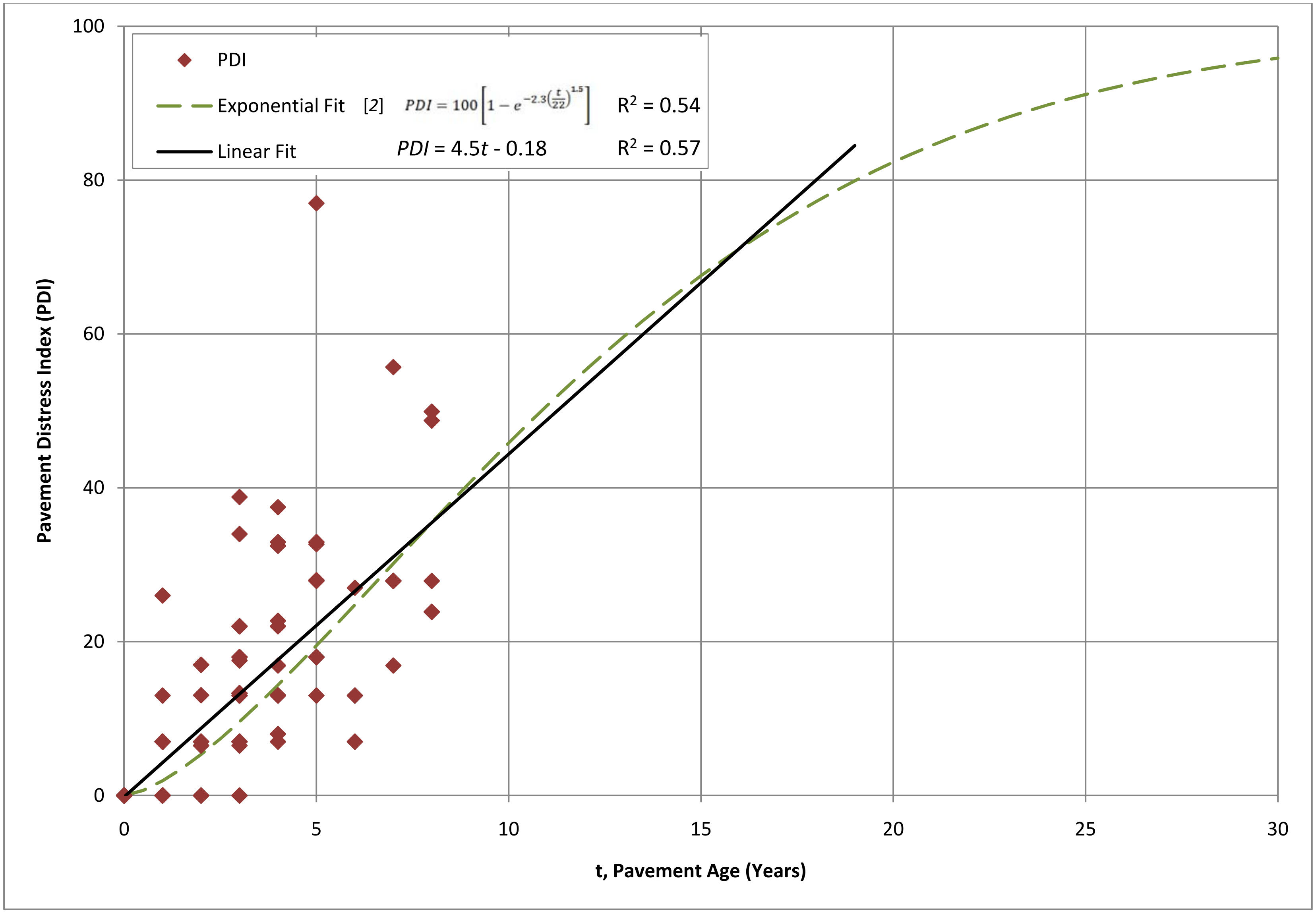

Figure 7. Pavement distress prediction, "poor" performance category pavements

8.2 Structural Layer Coefficient

WisDOT pavement design is based on AASHTO design procedures, which use the sum of the products of each pavement layer's structural layer coefficient and layer thickness to compute the SN of the entire pavement structure. The SN is computed using Equation 4 as follows:

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \qquad \text{Equation 4}$$

In Equation 4, a_1, a_2, and a_3 are structural layer coefficients for the HMA pavement, the base layer, and the subbase layer, respectively; D_1, D_2, and D_3 are the respective layer thicknesses.

The structural layer coefficient is an empirical unit. The original layer coefficients were established during the AASHO Road Test in the 1950s and have since been calibrated by state agencies to represent local materials. As discussed in Section 2.2, a nationally accepted range of values has been used for a_2 when the base layer is rubblized concrete. These values range from 0.14 to 0.29. [*5, 10*]

Pavement layer modulus is another common design input. The MEPDG uses modulus values to predict pavement distress and service life. Relationships between a granular material's structural layer coefficient and modulus have been developed. If the resilient modulus, E_2, is known for an untreated base layer, the following relationship can be used to estimate the layer coefficient:

$$\mathrm{a_2} = 0.249(\mathrm{logE_2}) - 0.977 \qquad \text{Equation 5}$$

This relationship is plotted in Figure 8. Current WisDOT design practices stipulate an a_2 value of 0.20 to 0.24 for rubblized pavements, which correlates with a modulus of approximately 55 to 80 ksi. The layer coefficient specified for crushed stone is 0.14; this correlates with E_2 equal to approximately 30 ksi.

It is difficult to determine the resilient modulus for a rubblized layer prior to design, as the rubblized layer is not yet available for testing. Therefore, the only way to determine actual design parameters for a rubblized layer is to perform testing of rubblized pavement systems after construction, as discussed by Von Quintus, et al. [*2*] It is possible to determine E_2 for a rubblized pavement layer through non-destructive methods such as FWD testing.

National results for rubblized pavement layer testing have been summarized by Von Quintus, et al. A wide range of modulus values were determined: 35 ksi to over 100 ksi. Using Equation 5, this range correlates to a_2 values of 0.15 to 0.27, which is consistent with the national range of a_2 values mentioned above. The lower end of the estimated values represents highly broken pieces (2 inches or less), while a 100 ksi modulus might be found with a lightly broken concrete slab. [*2*]

The modulus for an ideal rubblized layer, with good interlocking particles (broken pieces 6 to 12 inches in size), was determined by Von Quintus, et al. to be greater than 70 ksi (a_2 = 0.23). The design modulus recommended in that study for Wisconsin pavements was 65 ksi (a_2 = 0.22). This value was not achieved via field testing but rather by applying an iterative method relating observed and predicted pavement distress levels. [*2*]

Concern has been expressed by Wisconsin pavement designers that the current layer coefficients used for rubblized pavement design are high; this results in thin HMA pavement design, leading to premature distress. This study did not find HMA pavement thickness to be a direct cause of rubblized pavement failure, and therefore current layer coefficient values can be assumed to be valid. However, it would be worthwhile to use actual field testing of Wisconsin's rubblized pavements to further refine the rubblized layer coefficient used for pavement design.

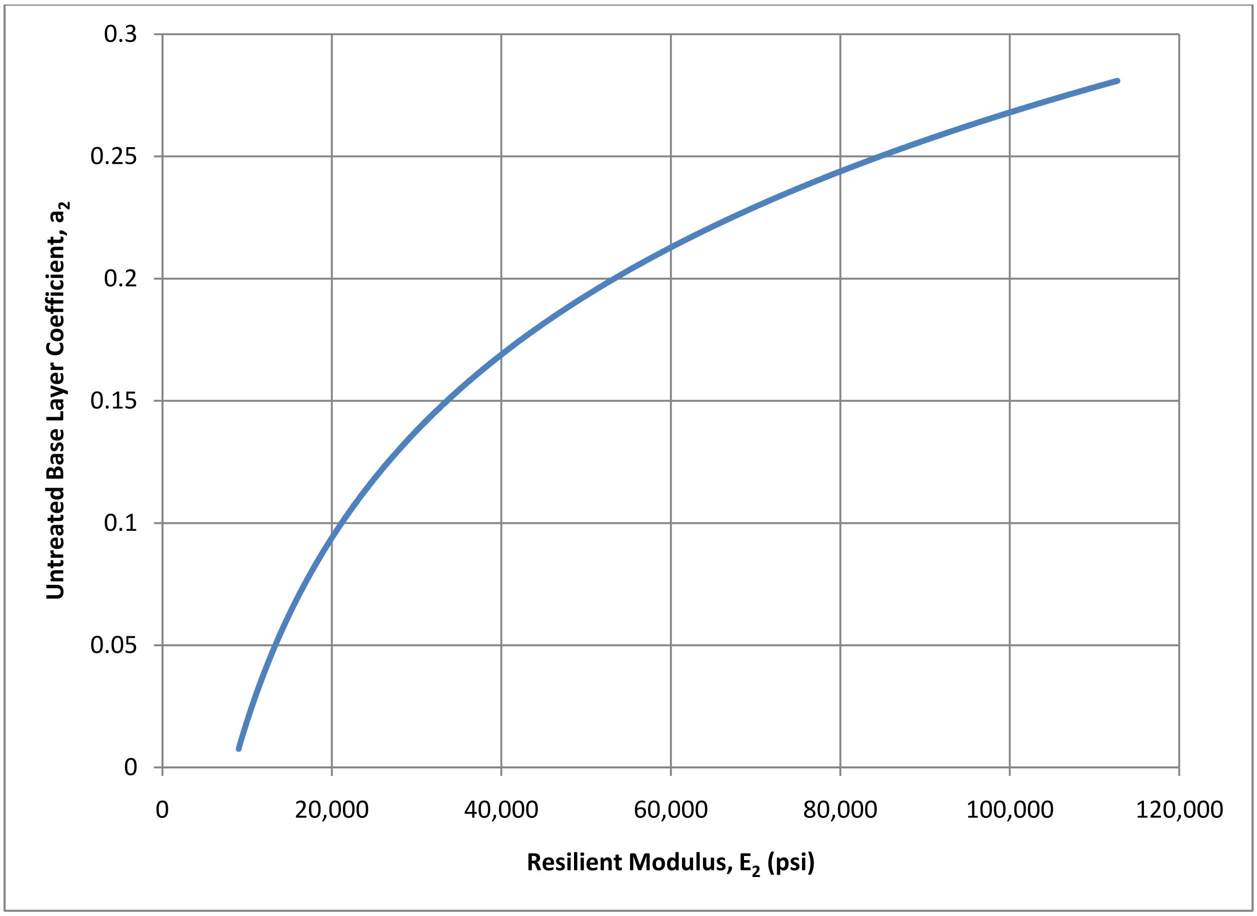

Figure 8. Relationship between structural layer coefficient and resilient modulus for untreated base.

8.3 Conclusions

Based on the information presented in the preceding sections, the following conclusions were drawn:

- Predictive distress models with a PDI threshold of 65 forecast an initial service life of 17 to 25 years for pavements in the "good" performance category. The predicted service life of pavements in the "poor" performance category was 13 to 15 years.
- Current values for structural layer coefficient of the rubblized layer (0.20 to 0.24) are valid, but these values should be updated using actual field testing of Wisconsin rubblized pavements.

9. Study Conclusions

Conclusions for each analysis performed in this study were presented at the end of Sections 6 through 8. The conclusions are repeated below for ease of reference.

Section 6 - Project Specific Analysis

- Close observation of the rubblizing process and inspection of the rubblized layer are critical to good performance. Documented excavation of test holes and test rolling are useful ways to evaluate the condition of the rubblized layer.
- If areas of weak support are noted during construction, the following modifications could result in better performance: modification of the breaking pattern, EBS and/or base patching, and spot placement of additional HMA thickness.
- If wet conditions exist (e.g., the subgrade has a high moisture content, there is a local high water table), drainage features could benefit the pavement system. If a significant rain event occurs while the rubblized layer is exposed, the material should be given time to thoroughly dry prior to paving.
- Tenting of HMA cracks, the most prevalent distress noted in poor-performing projects, was a result of non-uniform support at previous concrete pavement joint and crack locations. Reflective cracks developed in the HMA pavement, and water seeped through to the rubblized layer. The water froze and expanded during the winter months, leading to tenting of the HMA cracks.

Section 7 - Overall Performance Analysis

- The following factors were not directly related to ultimate pavement performance for the analyzed pavements:
    - The following concrete pavement parameters: construction year, age, thickness, pavement type and overall condition
    - Base, subbase, and native soil properties
    - Traffic levels and construction conditions
- Severity of existing concrete pavement distresses were not related to performance, with the exception of joint and/or crack condition, as discussed in Section 6.21.
- The month that rubblization took place was not related to performance for the analyzed studies, but rubblizing during the spring thaw should be avoided.
- Use of an HMA pavement warranty was not found to be related to ultimate pavement performance. However, it is important that the inspection and acceptance of the rubblized pavement be given adequate attention in a warranted HMA pavement situation, as the rubblized material itself is not covered by the warranty.
- Two projects in the poor performance category might have benefited from additional HMA thickness.
- Rubblized systems were more likely to have good performance when the subgrade had a more coarse grained composition.

- Inclusion of subgrade drainage systems likely benefited some projects but did not guarantee good performance.
- Current FDM guidelines state that difficulty with rubblization could be encountered when the DGI is greater than 12. However, several projects in the good performance category had DGI values of 14.
- Transverse cracks in the HMA developed more quickly in pavements in the "poor" performance category.
- Pavements in the "poor" performance category were more likely to be in urban settings.

Section 8 - Design Considerations

- Predictive distress models with a PDI threshold of 65 forecast an initial service life of 17 to 25 years for pavements in the "good" performance category. The predicted service life of pavements in the "poor" performance category was 13 to 15 years.
- Current values for structural layer coefficient of the rubblized layer (0.20 to 0.24) are valid, but these values should be updated using actual field testing of Wisconsin rubblized pavements.

10. Recommendations

Based on the results and conclusions from the projects analyzed in this study, the following ten recommendations are proposed:

1. Correct deteriorated joints and cracks prior to rubblization.

Severely deteriorated joints and cracks should be repaired prior to rubblization to increase the uniformity of the paving platform. The following joint replacement methods were successful in this study's projects:

a) *Aggregate base patching after rubblization* - Remove the rubblized concrete full-width and full-depth at the location of the deteriorated joint/crack. Fill with CABC. Ensure the base patch becomes homogenous with the surrounding rubblized material during rolling.
   - This method is preferred if there is adequate time for the base patching operation between rubblization and rolling. In addition to repairing clearly distressed joints and cracks, other problem areas noted during rubblization can be addressed.

b) *Aggregate base patching prior to rubblization* - Saw cut and remove the deteriorated concrete pavement full-depth. Determine the width of the removal based on the amount of deterioration at the joint or crack; intact concrete can be left in place. Fill with CABC. Do not rubblize the aggregate base patch. Instruct the rubblizer operator to stop breaking at base patch locations. Ensure the base patch is homogenous with the surrounding rubblized material after compaction.

- This method is acceptable if there is limited time between rubblization, rolling and HMA paving. As it requires concrete pavement saw cutting, it will likely be more costly than the previous method.

c) *Concrete base patching prior to rubblization* - Saw cut and remove the deteriorated concrete pavement full-depth. Determine the width of the removal based on the amount of deterioration at the joint or crack; intact concrete can be left in place. Place the concrete patch (nondowelled). High early strength concrete can be used. Rubblize the base patch along with the remainder of the concrete pavement.
   - This method is only recommended if lane closure time is limited. In this case, traffic can use the travel lane after concrete base patching is completed and prior to rubblization. However, this is likely the most costly repair method. In addition, the concrete base patch might rubblize differently than the surrounding concrete, resulting in a non-homogenous area. [*30*]

As discussed in the descriptions above, the most suitable repair method will depend on project-specific constraints and cost considerations. A flow chart outlining the construction sequence for the three repair method options is provided in Appendix 6.

If it is suspected that the subgrade has been compromised under deteriorated joints, also perform EBS and backfill with CABC prior to base patching. The depth of EBS will be at the engineer's judgment.

It is recommended that a field review be conducted to identify joints in need of repair. Joints with heavy deterioration, spalling, and/or evidence of pumping should be candidates for repair. The PCI rating system can be used as a guide; relevant distresses are provided in Table 27. At minimum, joints should be addressed that have been rated at the severity levels defined in Table 27.

Table 27. Joint and Crack Severity Rating Thresholds as Defined by PCI [*26*]

Distress Type	**Severity Rating**	**Description**
Distressed Joints/Cracks	Medium or High	Distress greater than 6 inches wide in wheel path*
Joint Spalling	Medium or High	Depth of spall is greater than 1 inch and most or all spalled pieces are missing
Pumping†	Any	Ejection of slab foundation material through joints or cracks
Cracking	High	Crack width greater than 2 inches

* Joints that meet this severity rating threshold outside of the wheel paths should also be repaired
† If pumping is noted, EBS should also be performed

2. Ensure all joints/cracks are indistinguishable after rubblization.

Joints and cracks remaining in the underlying rubblized layer will eventually reflect through to the HMA surface and could lead to the failure described above. If joints and cracks are still evident after rubblization, they should either be re-rubblized or repaired with aggregate base patching as described in recommendation number one.

3. Test roll the rubblized layer and address weak areas as necessary.

Test rolling should be specified at an initial frequency of 100 ft every 1000 ft. Each lane should be test rolled. If high deflection is noted, the test frequency should be increased to 100 ft every 500 ft, or as often as the engineer deems necessary. The more area test rolled, the greater the chance of finding and remedying problem areas.

Locations of high deflection under test rolling should be marked, repaired with EBS, backfilled with rubblized material and/or CABC, and rolled according to specification.

A flow chart outlining these guidelines for test rolling and repair is provided in Appendix 7.

It is also recommended that undistributed quantities of test rolling, EBS and CABC be included in the plans for the purpose of addressing problem areas discovered during rubblization.

4. Correct known areas of weak subgrade support prior to rubblization.

If weak areas of subgrade are known to exist, repair of these areas (e.g., concrete removal and EBS) should be included in the project plans.

If possible, use FWD testing to assess the condition of the subgrade. As reported by Von Quintus, et al., the foundation layers underlying the concrete pavement should have an in-place modulus of at least 10 ksi. [*2*] FWD testing can help identify areas with lower modulus values.

Network-level FWD testing is an idea that has been proposed in Wisconsin. If routine monitoring of STN roadways using the FWD is implemented, it would be an excellent way to preemptively identify trouble areas in candidate rubblization projects. This information could potentially be used during the life cycle cost analysis and pavement type selection.

5. Carefully monitor the rubblization process.

Long-term performance of rubblized pavements is critically dependent upon close monitoring and evaluation of the rubblization process. The particle sizes should be assessed using test holes, as outlined in the standard specification. Communication should be maintained with the rubblizer operator, as he can help identify areas of weak support. Project staff should carefully inspect the rubblized surface for water seepage, distinguishable cracks or joints, and other issues that would need additional attention. The flow chart provided in Appendix 7 outlines these guidelines.

Rubblization in urban areas should have particular attention paid, as these projects typically have more complicated construction constraints than rural projects. Rubblization should still be monitored closely in projects with warranted HMA pavement. Although less Department oversight is required during warranted paving, the rubblized material itself is not warranted and should be inspected as if no pavement warranty were involved.

6. Ensure that the foundation is dry prior to HMA paving.

Projects were more likely to have early failure when the subgrade and/or rubblized layer was wet. Current FDM guidelines suggest the use of a drainage system when fine-grained, low-permeability soils are present. Based on this study's results, it is recommended that FDM guidelines also strongly encourage the use of a drainage system when subgrade soils have a high moisture content (i.e., above the soil's plastic limit).

FDM guidelines state that a more detailed soils investigation may be necessary if the water table is less than four feet from the top of the existing subgrade. It is recommended that if this is a seasonal condition, wait to rubblize until the water table has subsided. If the water table is permanently high, proceed with the detailed soils investigation and consider drainage options. Rubblization may not be a feasible reconstruction option.

It is recommended that HMA pavement be placed as soon as possible to protect the rubblized layer from rain events. If the rubblized pavement is exposed to heavy rain, the project engineer should consider postponing paving until the paving platform is thoroughly dried.

7. Promptly seal all cracks that occur in the HMA pavement.

If cracks do occur in the HMA pavement, it is critical to prevent water penetration by sealing them as quickly as possible. This will help prevent water from pooling, freezing and expanding, which could lead to tenting of the crack.

8. Review the initial service life used for rubblized pavement design.

In current WisDOT pavement design, newly constructed HMA pavements over a non-drained base are assigned an initial service life of 18 years, and rubblized pavements are assigned an initial service life of 22 years. [*29*] This 25 percent increase in service life is designated because the rubblized material has been defined as a drained layer. However, results from this study have put into question the drainability of the rubblized layer. Tests of rubblized pavements in Michigan have also shown mixed results on drainage of rubblized concrete. [*4*] The service life predicted for this study's pavements in the "good" performance category was 17 to 25 years. A larger population of rubblized pavements should be reviewed, however, if changes to the design service life are to be made.

9. Revise the current DGI guideline.

The FDM currently states that "based on WisDOT research and experience, the rubblization construction process experiences difficulties with soil classified with a DGI greater than 12." Several pavements in the "good" performance category had a DGI of 14. Changing the DGI guideline to 14 is recommended.

10. Further research.

It is recommended that further research of Wisconsin rubblized pavement systems be performed as described below.

A test section should be constructed utilizing the joint repair and test rolling guidelines described above. This study would refine the threshold for deteriorated joint repair and the frequency of test rolling. The impact on cost and construction time should also be investigated. Because tenting of the transverse joints typically appeared within the first few winter seasons, results from this investigation would be available quickly.

A large-scale study should be undertaken to investigate the modulus and layer coefficient of the rubblized pavement layer. Actual in-place measurements of the rubblized modulus should be catalogued for Wisconsin's rubblized pavement systems. FWD testing can be employed to evaluate this parameter. The modulus value(s) obtained could either be used directly in future mechanistic-empirical designs, or used to determine a statewide average structural layer coefficient for input with current design practices.

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25. Wisconsin Department of Transportation. "Pavement Distress Survey Manual." Unpublished.

26. Wisconsin Department of Transportation "Pavement Distress Survey Procedures." Division of Transportation Investment Management. Pavement Data Unit. Apr. 2010.

27. National Asphalt Pavement Association. "Rubblization: Design and Construction Guidelines on Rubblizing and Overlaying PCC Pavements with Hot-Mix Asphalt." Information Series 132. Jan. 2006.

28. Wisconsin Department of Transportation Functional Classification Criteria. Apr. 2003. Available online: http://www.dot.wisconsin.gov/projects/planresources/functional.htm

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Appendix 1 - Construction Diary Notes

USH 12, Walworth County

Paving crew initially used tack coat over rubblized layer, but were asked to stop on first day of paving. Test hole of rubblized layer on July 1, 1998 (first day of HMA paving). "Pavement breaking size meets specifications." Rubblization passes over gas lines was observed closely.

July 7, 1998 - Project staff "expressed concerns about the pressure variation at the joints during rubblization and how that will impact the ride specifications later."

July 8, 1998 - "Serious flexing was noted on the first lift in the inside E.B. lane after it was opened to traffic from STA 10+576 to 10+767. [Paving contractor] placed second lift to firm the area."

July 9, 1998 - Moist silt discovered after borings taken 7-10 ft deep. Rubblization pattern modified to create bigger pieces in area of soft subgrade STA 10+100 to 10+680.

July 20, 1998 - WisDOT staff conducted FWD tests before rubblization and after paving (area not documented).

HMA leveling courses were paved in some areas.

Timeline:

- Install edge drains: June 24-July 1, 1998
- Rubblize: June 29 - July 30, 1998
- Pave first layer: July 1 - August 18, 1998
- Paving second layer: July 20 - August 20, 1998
- Pave surface layer: August 24 - September 2, 1998

STH 67, Walworth County

8/3/99 - "Antigo Construction began rubblizing in the Village of Williams Bay. Rolling and compaction operations were observed and found to be adequate. The existing base has not shown any failure so far… A representative from Antigo Const. was on site to observe the process. A test hole was opened to determine the size of the concrete particles. Some of the particles were larger than the spec. The rubblizing machine was slowed to decrease the particle size."

HMA was paved immediately following the rubblization operation. Traffic was then allowed on the first HMA layer. HMA paving took place from 8/3/99 to 9/10/99, but it is unclear from the construction diary when paving was complete in the rubblized section of the project.

Timeline:

- Rubblize and pave first layer: August 3-5, 1999
- Paving of final layers: Exact dates unknown

I-894, Milwaukee County

April 2003

- 15-16 - Milling existing asphalt and partial depth concrete began NB
- 18 - Milling existing asphalt and partial depth concrete began SB
- 20 - Rubblized ~100 ft of pavement NB near Howard Ave to test. Looked "fairly good"
- 21 - Rubblizing began outside lanes NB. Test hole: particles met spec, but project manager thought "breakage was greater than what we needed," so rubblizing energy was turned down. Proof roll ordered, many project staff and rubblization crew watched. Encountered soft areas near transverse and longitudinal joints. Other areas were solid. Soft areas were marked for removal to 2-ft depth and replacement with breaker run and CABC (EBS). Rubblizing energy was reduced.
- 22 - EBS areas were repaired. Intervals of solid and weak base support were encountered as rubblization continued. Additional EBS areas were marked for repair. Comment that "pavement is not really broken into small pieces anymore. It is more like a crack and seat operation." Paving lower layer began NB, milling continued SB
- 23 - Milling, rubblizing and paving lower layer continued NB and SB
- 24 - DOT TSS staff observed rubblization. Agreed with decision to reduce rubblization frequency, proof roll to determine support quality, and repair soft areas. Repair was changed from EBS to concrete base patching for remainder of project. Rubblization and proof rolling continued SB. Water coming through cracks after rubblization. Milling and paving lower layer continued SB.
- 25 - Rubblizing and proof rolling continued SB. No base patch areas. Milling and paving lower layer continued SB.
- 30 - Paving contractor raised concerns about quality of rubblized base and its effect on ultimate ride quality. WisDOT staff noted that soft areas are being repaired and that HMA densities have been fine, indicating solid base. It was decided to continue rubblizing with the lower frequency (resulting in larger cracking pattern), continue proof rolling, concrete base patch soft areas, lightly crack these base patches to make homogenous base, add ¼-in extra HMA thickness, and profile mill prior to paving surface layer.

May 2003

- 6 - Milling and rubblizing median lane began NB. Several soft areas marked; these were concrete base patched after lower layer of HMA paved.
- 7 - Milling median lane finished NB. Rubblizing continued NB. Several soft areas marked. Spots were typically near highly deteriorated joints.
- 9 - Milling began EB
- 13 - Rubblizing median lane continued NB. Water noted coming through rubblized material. Paving SB.
- 15 - Rubblizing began outside lanes EB. Paving followed.

- 16 - Rubblizing complete outside lanes EB. Several base patch areas defined during proof rolling.
- 19 - In response to paving contractor's worry about good paving platform, the project manager said that the bad joints are being located during proof rolling and fixed with base patching. The bad joints "are fairly clear and defined. They either yield or they don't." WisDOT team members agreed that there won't be failures at joint locations. Paving EB.
- 21 - Lower layer paving median lanes EB.
- 28 - Milled existing asphalt and concrete pavement WB.
- 29 - Rubblizing outside lanes WB. Some base patch areas noted. WisDOT TSS and CO staff watched rubblizing and were "impressed with the thoroughness of the processes."
- 30 - Rubblizing outside lanes WB.

June 2003

- 3 - WisDOT and FHWA meeting with Marshall Thompson (University of Illinois). Described project's rubblization as "coarse rubblization or a fine crack and seat process." Thought this method would work fine. Said that the good HMA densities indicated the foundation was good (after base patching in required areas). Thought HMA thickness (5.25 in) was on the thin side.
- 4 - Rubblizing outside EB lanes. Paving WB.
- 5 - Rubblizing outside EB lanes. Poor stretch near 51^{st} St. that required base patching. Paving lower layer EB. Profile milling SB.
- 9 - FWD test results from prior week in WB lane 1 between 51^{st} and 60^{th} streets. Lower layer HMA had 4.03 SN. Adding future upper layer (1.75 in) = 4.74 SN, plus foundation = 5.44 SN. No structural issues. Paving lower layer EB.
- 10 - Paving surface layer SB.
- 11 - Profile milling NB and portions of SB.
- 12 - Paving surface layer SB. Rubblized WB outer lanes - many base patch areas.
- 13 - Paving surface layer NB.
- 18 - Rubblizing WB lanes 2 and 3 from Loomis Rd to 51^{st} St. Concrete in better shape than other portions of project.
- 19 - Rubblizing WB lanes 2 and 3. 18-ft base patch required from STA 149+50 to 150+75. Paving lower layer WB.
- 20 - Finish paving lower layer WB.
- 21 and 24 - Paving surface layer NB.
- 28 - Finished paving surface layer NB. Paved surface layer SB.
- 30 - Paved surface layer SB.

July 2003

- 1 and 14 - Paved surface layer SB.
- 1, 2, 10, 15, 16, 23-25 - Paved surface layer EB.
- 9, 12, 18, 19, 21, 26, 28-30 - Paved surface layer WB.

STH 42, Door County

Pavement was rubblized "too much" in the beginning (southern end of project). The first 100 ft were over-broken, so the rubblizing intensity was reduced. Soft spots noted at the areas listed in the table below. Contractor placed wedges in these areas.

STH 42 Soft Spot Locations

Station	**Located near:**
6+10	2000 ft south of Walker Rd
158+50	CTH HH intersection
471+25	150 ft north of Wayside Rd

Timeline:

- Rubblize mainline: September 5-15, 2000
- Pave first layer: September 5-15, 2000
- Pave second layer: September 18-28, 2000
- Pave third layer: May 2-18, 2001

USH 41, Oconto County

Very few details provided in construction diary. It is unclear when rubblization took place. Paving began on 7/10/2002 and was completed on 8/2/2002. No problems noted.

STH 23, Sheboygan County

Several trouble areas were encountered during the rubblization phase. The following are notes from the construction diary:

- 5/30/03 - "Due to rain, subgrade seemed soft but not unstable [under Rangeline Road overpass]."
- 6/9/03 - "There was some settling in the [WB] driving lanes [west of Woodland Rd]. Antigo also changed frequency of shoulder machine due to soft subgrade. At the beginning of the day there appeared to be some subgrade problems [near STH 32]. [The paving contractor] did not feel comfortable paving over this area. He recommended removing pavement and soft subgrade at least 1.5 feet down. I agreed. Antigo also said due to soft subgrade they would change the rubblizing frequency until the subgrade got harder."

In addition, the areas listed in the table below were addressed during rubblizing or paving operations.

Trouble Areas Encountered on STH 23

Location (STA)		**Trouble**	**Solution**
153+30 WB	1500 ft west of Woodland Rd	Settling, soft spot, water rising through cracks in shoulder	Modified rubblizing frequency
1686+25 to 1686+75 WB	500 ft east of STH 32	Soft subgrade	Remove pavement and soft subgrade 1.5 ft down*
152+25 to 153+45 EB	1500 ft west of Woodland Rd	Soft spot	Removed soft spot, rubblized concrete as a base, covered with 1 in CABC
152+60 to 153+15 EB	1500 ft west of Woodland Rd	Pavement dip developed under traffic	Surface was repaved
1685+10 EB	250 ft east of STH 32	Pavement dip developed under traffic	Surface was repaved
1685+10 to 1685+55 EB 1705+14 to 1706+91 EB 1712+78 to 1713+19 EB 1717+74 to 1720+66 EB 163+82 to 167+88 EB	250 ft east of STH 32 2200 ft east of STH 32 2200 ft west of Rangeline Rd 1700 ft west of Rangeline Rd 100 ft west of Woodland Rd	Binder tore along shoulder joint	Milled and filled 5-in HMA
148+92 to 149+26 WB 150+72 to 153+40 WB 155+18 to 155+97 WB 156+86 to 157+49 WB	1600 ft west of Woodland Rd 1400 ft west of Woodland Rd 1000 ft west of Woodland Rd 800 ft west of Woodland Rd	Unknown	Excavated and filled with rubblized concrete and CABC
1704+67 to 1705+11 WB shoulder 1705+70 to 1707+66 WB shoulder	2200 ft east of STH 32 2300 ft east of STH 32	Soft areas	Excavated and backfilled with rubblized concrete and CABC

Paving was completed in stages, with traffic allowed on the lower layers.

Timeline:

- Rubblize mainline: June 9-16, 2003
- Pave lower layers: June 11-13 and June 25-27, 2003
- Pave surface layer: June 30-July 2, 2003

USH 12/14 (Madison Beltline), Dane County

Work was completed at night. Single lane closures were in place with traffic in adjacent lane. Paving directly followed rubblization. Lanes were open to traffic between night paving operations. Traffic flowed on first HMA layer for four days prior to paving of second layer (weekend + one day of rain).

No mention of rubblization details: breaking pattern, base and subgrade quality, etc. No problems noted.

Timeline:

- Rubblize and pave first layer: September 12-22, 2005
- Pave second layer: September 26-27, 2005
- Pave third layer: September 29-October 6, 2005

STH 11, Green County

July 13, 2000 - Two test holes dug, and size requirements were met. Paving contractor not satisfied with surface of rubblized layer and opted to make two passes with vibrating smooth drum roller prior to paving. Antigo Construction eventually re-rolled entire project with ribbed roller per paving contractor's request.

July 14, 2000 - Soft area noted near "Vet clinic." Two test holes dug at centerline and in lane - size of rubblized pieces "good."

July 17, 2000 - Additional soft areas noted at STA 585+00 to 600+00. Modified breaking pattern to more of a crack and seat in these areas.

July 21, 2000 - Proof rolled area noted above. Decided to place 8-in base course layer at STA 589+00 to 595+60.

July 28, 2000 - Underdrain placed in soft area noted above.

August 8, 2000 - Aggregate and density problems noted in HMA mix. "Densities… variable due to unevenness of rubblized pavement underneath."

Timeline:

- Rubblize: July 13-29, 2000
- Pave first layer: August 8-23, 2000
- Pave second layer: August 10-29, 2000
- Pave surface layer: September 5-13, 2000

USH 53 NB, Chippewa County

Work completed under traffic in adjacent lane. No traffic on first and second layers - all layers paved prior to traffic. No mention of rubblization details: breaking pattern, base and subgrade quality, etc. No problems noted.

Timeline:

- Passing lane
  - Milling asphalt: July 21-22, 2005
  - Install drainage: July 25-27, 2005
  - Rubblize: August 8-10, 2005
  - Pave first layer: August 10-11, 2005
  - Pave second layer: August 12-13, 2005
  - Pave third layer: August 15, 2005
- Driving lane
  - Milling asphalt: August 17-18, 2005
  - Install drainage: August 19, 2005
  - Rubblize: August 20-22, 2005
  - Pave first layer: August 22-23, 2005
  - Pave second layer: August 24, 2005
  - Pave third layer: August 25-26, 2005

USH 53 SB, Chippewa County

Work completed under traffic in adjacent lane. No traffic on first and second layers - all layers paved prior to traffic. No mention of rubblization details: breaking pattern, base and subgrade quality, etc.

One problem noted: 18-ft section at STA 1187+41 over a culvert pipe that wasn't rubblized. Discussed problem with Antigo and paving contractor, since the HMA pavement was under warranty. Wasn't clear from diary how issue was resolved.

Timeline:

- Milling asphalt: May 8-10, 2007
- Install drainage: May 14-17, 2007
- Rubblize: May 14-17, 2007
- Pave first layer: May 18-23, 2007
- Pave second layer: May 23, 2007
- Pave third layer: June 4, 2007

USH 8, Oneida County

No problems noted during rubblizing. After rubblizing and paving lower layer, contractor raised concerns about the rubblized pavement settling. Planned thickness was 4.5 inches (2.5-in lower layer plus 2-in surface layer). Contractor proposed paving an additional 1 inch of HMA and placing in two layers. WisDOT agreed, and total HMA thickness paved was 5.5 inches.

Timeline:

- Install drainage correction at frost heave locations: August 4-22, 2003
- Rubblize: August 19-26, 2003
- Pave first layer: August 19-26, 2003
- Pave second layer: September 2-4, 2003
- Pave third layer: September 5-11, 2003

CTH E, Milwaukee County

Test area on first day of rubblization. Rubblization modified to a "heavy cracking to break the slab without turning the surface to gravel… The slab was verified to be broken enough that it no longer reacted as a single unit.

Several areas were test rolled and areas that needed undercutting were marked:

- 212+00 to 167+00
- 151+00 to 130+47
- 153+16 to 167+00

Additional failure areas:

- Yielding soft spot at STA 155+85
- Pavement settlement noted between STA 164+29 to 164+63. Area was removed and replaced with aggregate base patching
- HMA rutting and shoving at STA 174+00 and west of Menomonee River Bridge (STA 152+00)
- Aggregate base repair performed in eastern portion of project (103rd St to 91st St)

Timeline:

- Rubblize: June 4-5 (WB), July 29-30, August 27 (EB) 2009
- Pave first layer: June 6-12 (WB), August 1-5, August 29-31 (EB), 2009
- Pave second layer: June 10-12 (WB), August 4-5, August 29-31 (EB), 2009
- Pave third layer: September 15-18, 2009

STH 20, Racine County

No trouble noted during rubblization process. No mention of breaking pattern, base and subgrade quality, etc. However, after the lower HMA layer was placed, noticeable settlement was observed. An additional one inch of HMA was placed in the center two travel lanes for the purpose of providing a leveling layer prior to placement of the surface layer. An additional 1,400 tons of E-10 mix was change ordered.

Two contract modifications for pay deduct for deficient asphalt pavement (pavement did not meet density requirements). Five percent deduct for 1,500 tons; two percent deduct for 750 tons. The deficient pavement remained in place. Locations unknown.

Timeline:

- Rubblize:
  - June 17-21, 2002 (inside lanes)
  - June 24-28, 2002 (outside lanes)
- Paving:
  - June 24-28, 2002 (inside lanes)
  - June 26-July 2, 2002 (outside lanes)

USH 41, Brown County

Problem noted early in paving - blowup in NB passing lane. Contractor said it was due to subgrade movement, but project engineer thought it was due to dump trucks rutting the aggregate surface.

Timeline:

- Rubblize:
  - August 6-13, 2002 (NB lanes)
  - August 13-15, 2002 (SB lanes)
- Pave first layer:
  - August 7-13, 2002 (NB lanes)
  - August 13-15, 2002 (SB lanes)
- Pave second layer
  - August 19-20, 2002 (NB lanes)
  - August 15-19, 2002 (SB lanes)
- Pave surface layer
  - August 27, 2002 (NB lanes)
  - August 28-29, 2002 (SB lanes)

I-43, Rock County

Original staging plan called for construction of temporary shoulders to accommodate traffic while construction in adjacent lane was completed. Prior to construction, a "short stretch" was test rolled, and no problems with base were noted. However, soft spots were encountered during milling of existing shoulder. Temporary shoulders subsequently failed when paved over soft spots. Some soft spots were excavated and repaired. Failure suspected to be caused by trapped water and saturated subgrade. Plans modified to construct crossovers so shoulders would not have to support traffic. Pipe underdrain installation also added to the plans.

Rubblization difficult. Rubblizer operator suspected poor subgrade and subsequently used a modified breaking pattern. Difficult to achieve 2-in rubble at surface, and surface fracturing was inconsistent. Agreed that the modified breaking pattern was best - "break as much as possible without tenting the joints." Also decided to remove loose asphalt patching material and joint sealant.

Many meetings held to discuss quality of rubblized pavement after completion of rubblization in NB lanes. Personnel from WAPA, Antigo, paving contractor, WisDOT, and University of Illinois evaluated conditions. Determined that rubblized material had adequate structure, surface was acceptable for paving, and HMA warranty should remain in place. Instructed to proceed with underdrain installation and rubblization in SB lanes. WisDOT staff inspected SB rubblization pattern; found it acceptable.

Several rain delays during rubblization and paving. No further problems noted during mainline paving.

Timeline:

- I-43 northbound
    - Rubblize: June 7-18 & July 3-9, 2007
    - Install drainage: June 15-19, 2007
    - Pave first layer: July 5-11, 2007
    - Pave second layer: July 18-19, 2007
    - Pave third layer: July 24-25, 2007
- I-43 southbound
    - Install drainage: August 14-21, 2007
    - Rubblize: August 20-24, 2007
    - Pave first layer: September 5-13, 2007
    - Pave second layer: September 14-18, 2007
    - Pave third layer: September 21-22, 2007

STH 113, Dane County

Poor quality material found in frontage road (not mainline) at west end of frontage road (north/west area of project). This area later had EBS.

Marsh area (near Kennedy Rd. and River Rd.) was excavated deeper and farther north than anticipated; geogrid stabilizing mat and French drains were added south of River Rd. Standing water and drainage problems were also noted in this area. (The marsh area was primarily an HMA new construction - addition of two northbound lanes - not rubblize.) Pavement failures were noted and corrected in this area - not documented if in new construction or rubblized lanes.

Test hole dug south of CTH I/STH 19 interchange. "The rubblizing appears to be effectively breaking the bottom half of the pavement slab as per standard spec 335.3.2… There are evident depressed areas that will require a leveling course."

Antigo noted a few soft spots. Two near STA 191 and 198 (near STH 19 interchange) was dug out 3 to 4 ft down and filled with broken concrete and base material. Water noted at this interchange, which caused lower HMA layer to "flex and break." Another noted 200 ft south of River Rd.

Rain events while rubblizing north/west portion of project (STH 19). Wet conditions noted.

Timeline:

- Rubblize: May 4, June 21-25, July 29-August 2, August 24, 2004
- Pave first layer: June 29-30, July 6, August 11, August 25, 2004
- Pave second layer: July 1-5, July 7, July 1,2 2004
- Pave third layer: July 7-9, August 9, August 23, September 13, 2004

USH 53, Chippewa County

Paving diary not available for review.

USH 53, Douglas County

No problems noted during rubblizing. No information on paving.

Rubblizing timeline:

- NB inside lane: June 4-6, 2002
- SB inside lane: June 7-12, 2002
- NB outside lane: June 12-17, 2002
- SB outside lane: June 17-20, 2002

USH 51 SB, Lincoln County

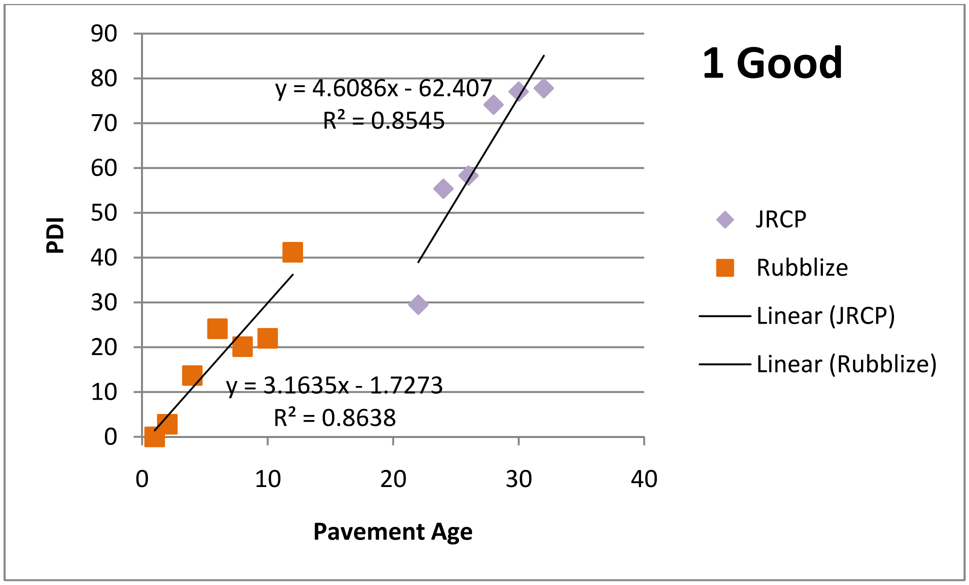
Wet, weak subgrade was discovered during rubblization at several locations where frost heave correction was planned. Two areas were recorded in the diary (STA 1604+00 to 1605+00 and STA

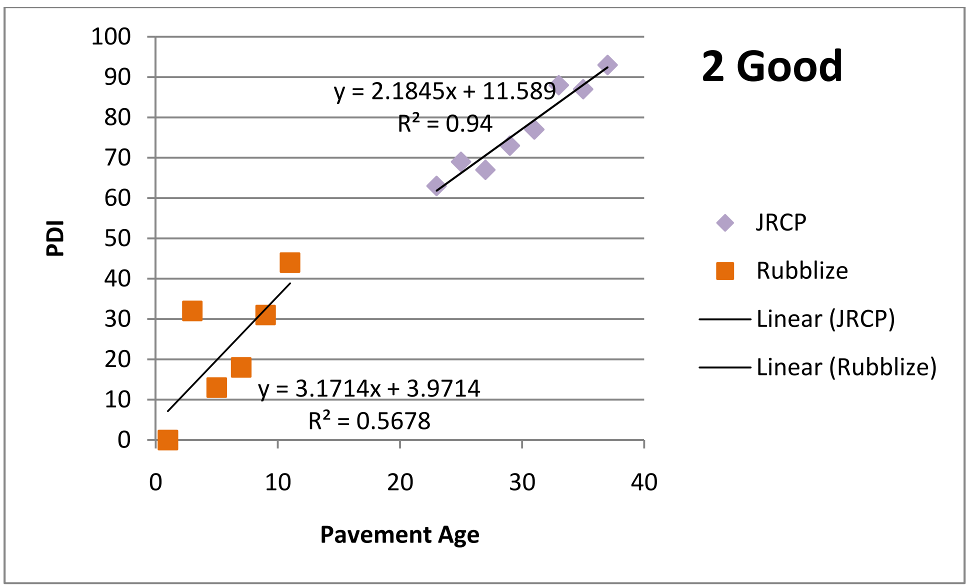
1364+43 to 1364+73), and other areas were mentioned but not specifically documented. In the soft subgrade areas, the subgrade was removed to a depth of 3 to 4 ft (EBS) and replaced with breaker run and CABC. It was noted that some of the EBS areas later became wet, and attempts were made to dry them out. However, heavy rain occurred, and some wet/soft areas were paved over without drying out.

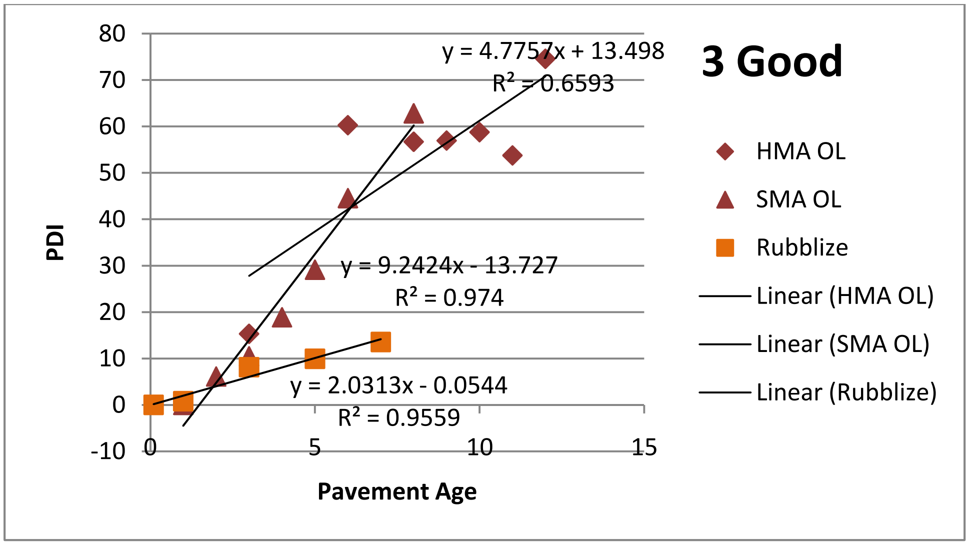
Timeline:

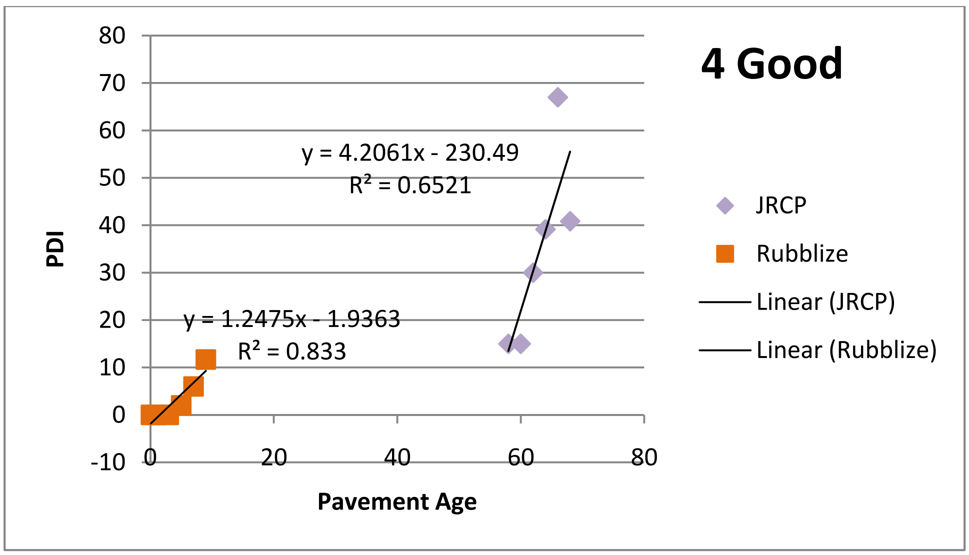
- Crossover construction: September to November, 2005
- Install drainage: April 20-25, 2006
- Rubblize: April 27-May 19, 2006
- Pave first layer: May 2-22, 2006
- Pave second layer: May 23-June 5, 2006
- Pave third layer: June 6-23, 2006

Appendix 2 - Plots of Historic PDI Data






3 Good
PDI
Pavement Age
y = 4.7757x + 13.498
R² = 0.6593
y = 9.2424x - 13.727
R² = 0.974
y = 2.0313x - 0.0544
R² = 0.9559
HMA OL
SMA OL
Rubblize
Linear (HMA OL)
Linear (SMA OL)
Linear (Rubblize)


4 Good
PDI
Pavement Age
y = 4.2061x - 230.49
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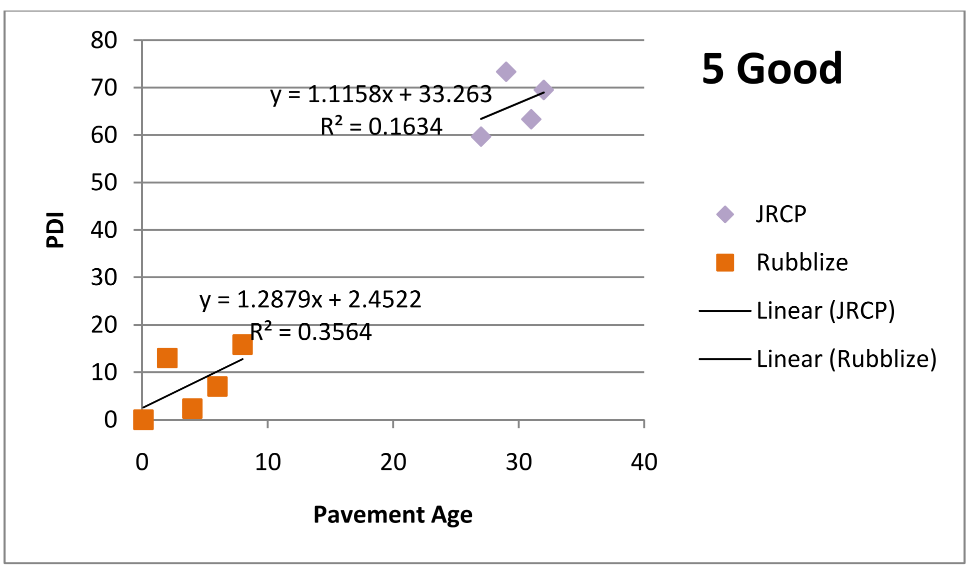
y = 1.2475x - 1.9363
R² = 0.833
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)


5 Good
PDI
Pavement Age
y = 1.1158x + 33.263
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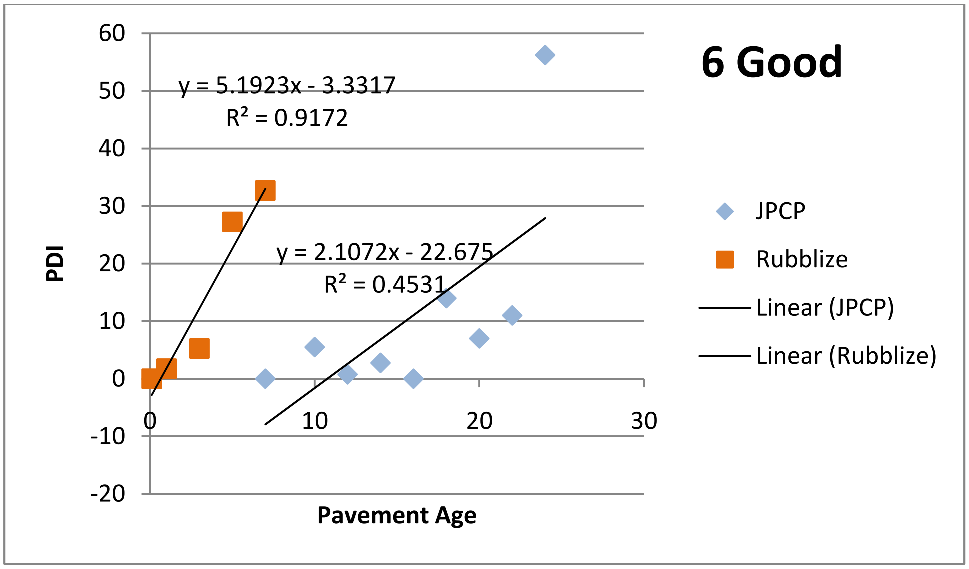
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)


6 Good
PDI
Pavement Age
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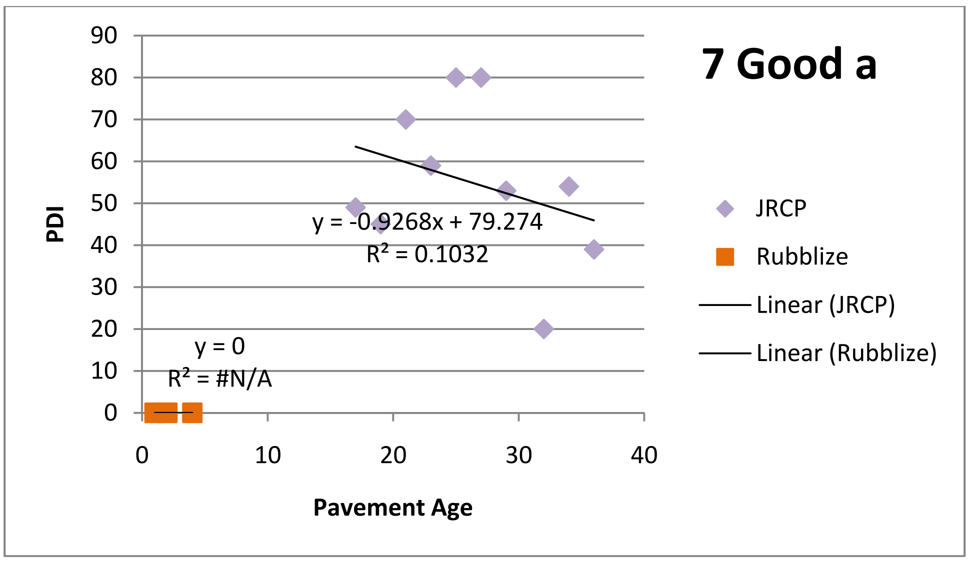
JPCP
Rubblize
Linear (JPCP)
Linear (Rubblize)


7 Good a
PDI
Pavement Age
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JRCP
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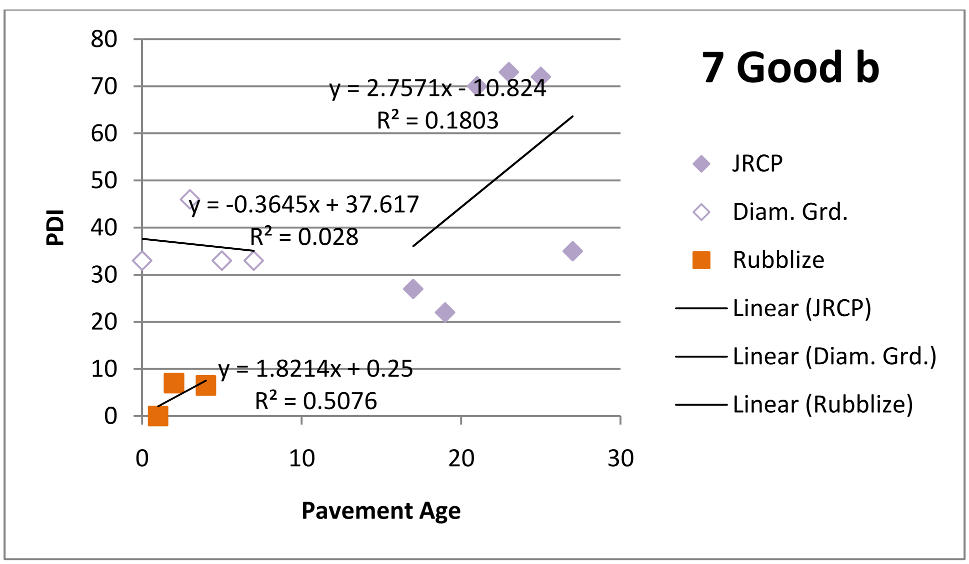
Linear (JRCP)
Linear (Rubblize)


7 Good b
PDI
Pavement Age
y = 2.7571x - 10.824
R² = 0.1803
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R² = 0.028
y = 1.8214x + 0.25
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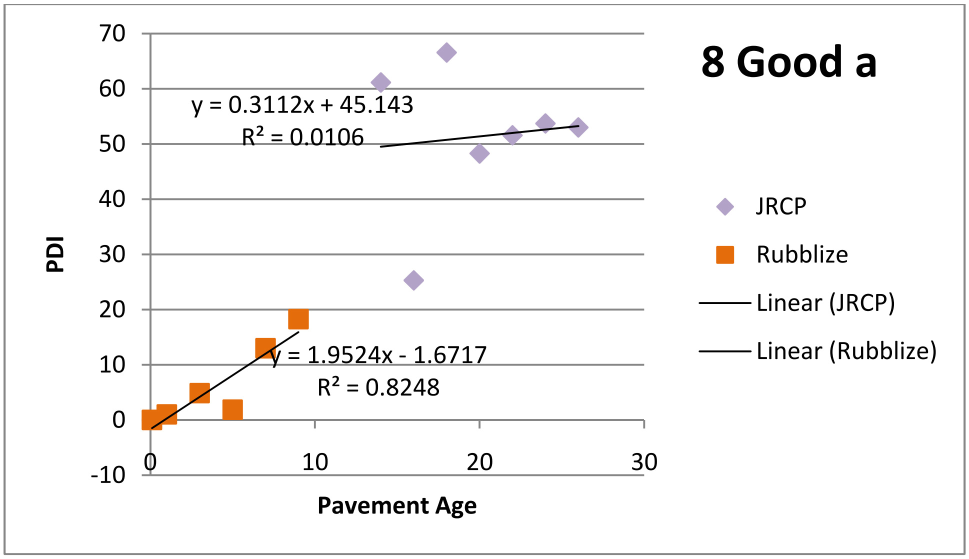
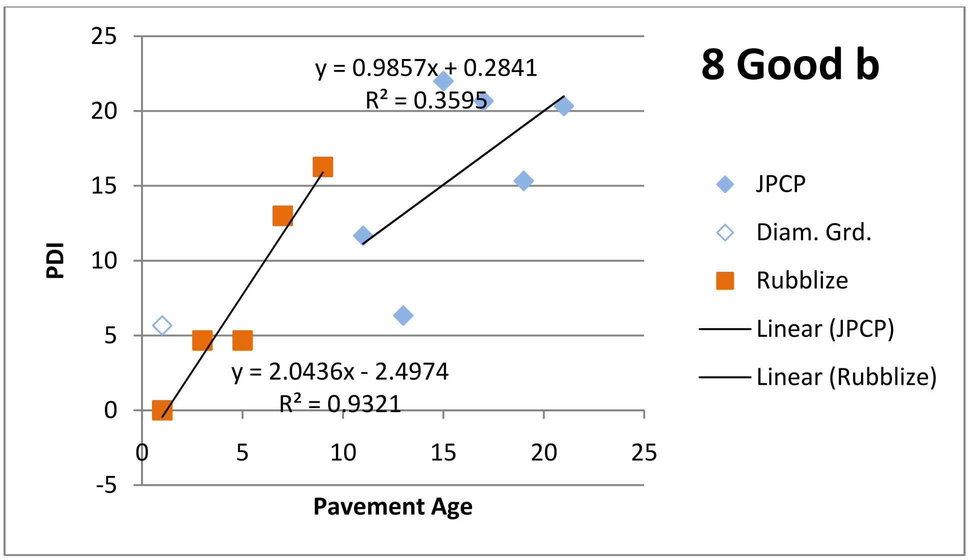
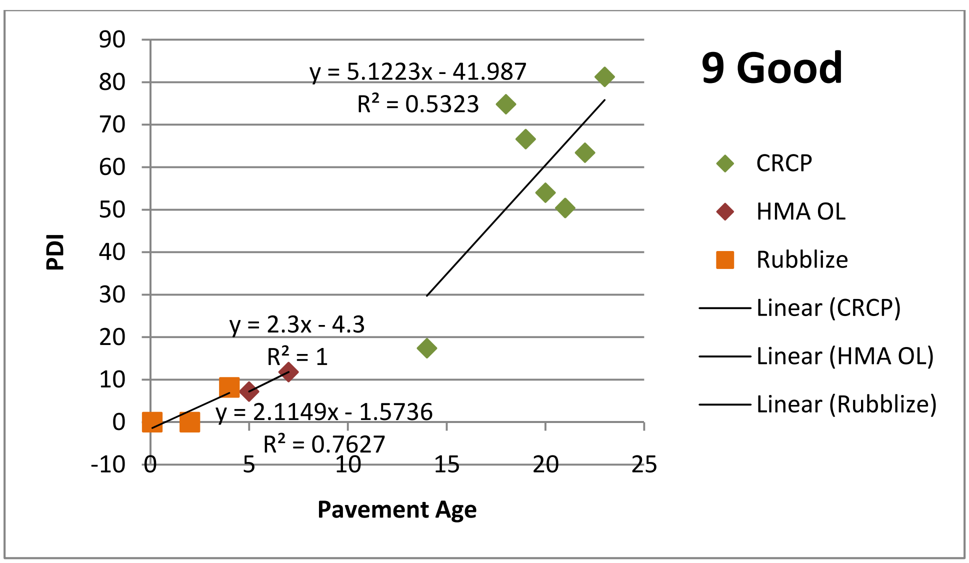
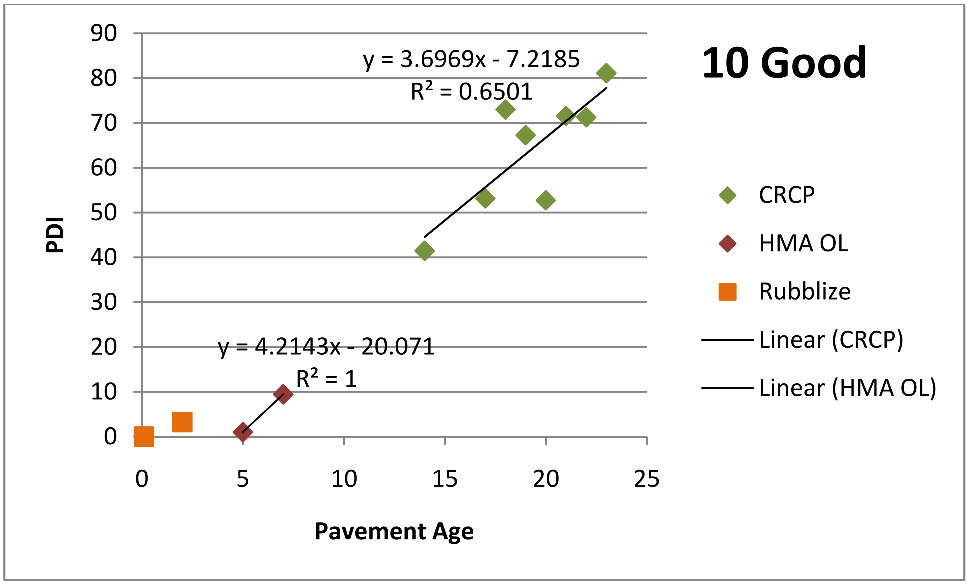
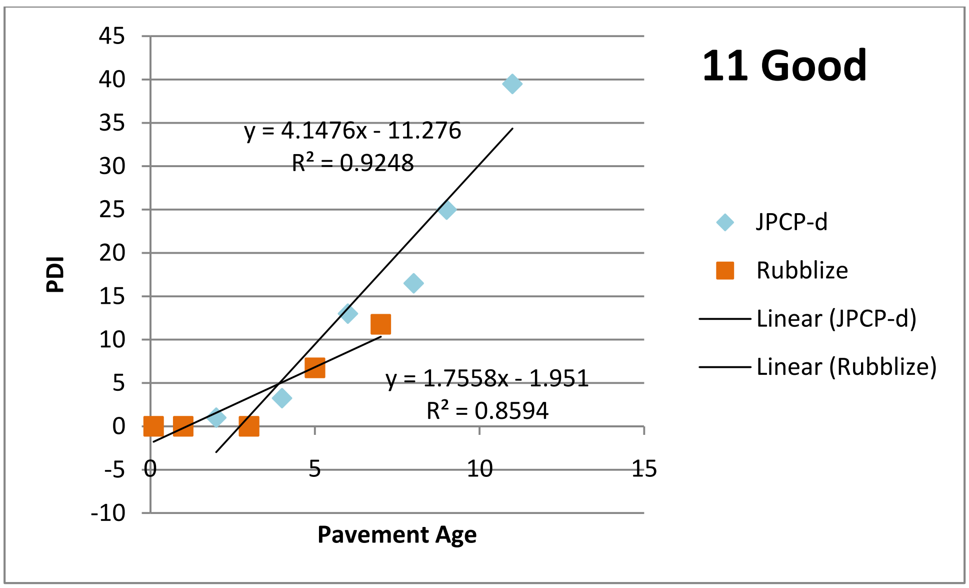
JRCP
Diam. Grd.
Rubblize
Linear (JRCP)
Linear (Diam. Grd.)
Linear (Rubblize)


8 Good a
PDI
Pavement Age
y = 0.3112x + 45.143
R² = 0.0106
y = 1.9524x - 1.6717
R² = 0.8248
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)
70
60
50
40
30
20
10
0
-10
0
10
20
30

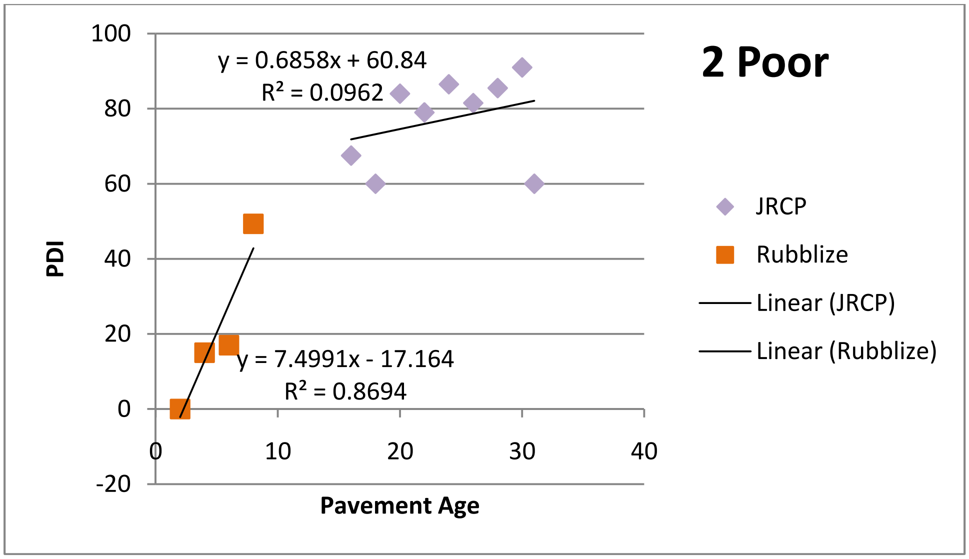

8 Good b
PDI
Pavement Age
y = 0.9857x + 0.2841
R² = 0.3595
y = 2.0436x - 2.4974
R² = 0.9321
JPCP
Diam. Grd.
Rubblize
Linear (JPCP)
Linear (Rubblize)
25
20
15
10
5
0
-5
0
5
10
15
20
25

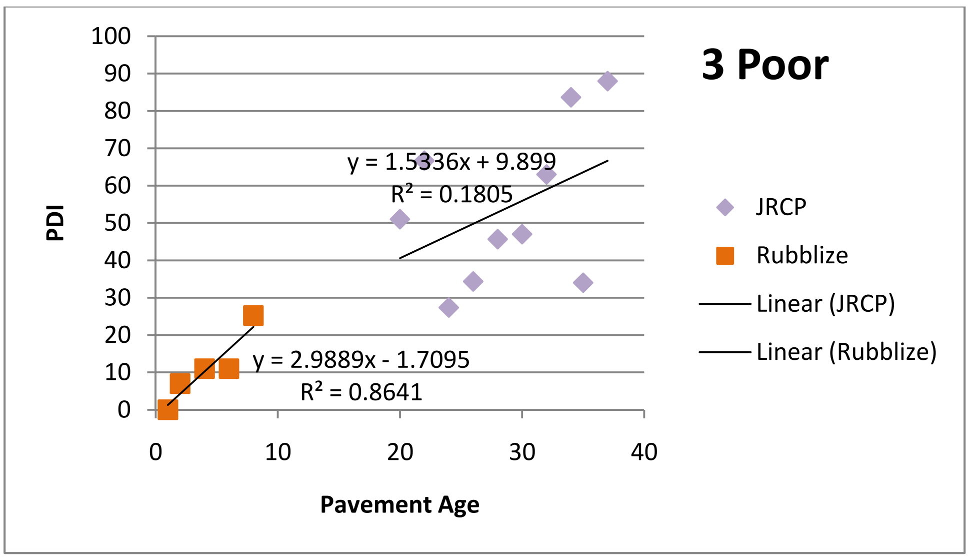

9 Good
PDI
Pavement Age
y = 5.1223x - 41.987
R² = 0.5323
y = 2.3x - 4.3
R² = 1
y = 2.1149x - 1.5736
R² = 0.7627
CRCP
HMA OL
Rubblize
Linear (CRCP)
Linear (HMA OL)
Linear (Rubblize)
90
80
70
60
50
40
30
20
10
0
-10
0
5
10
15
20
25

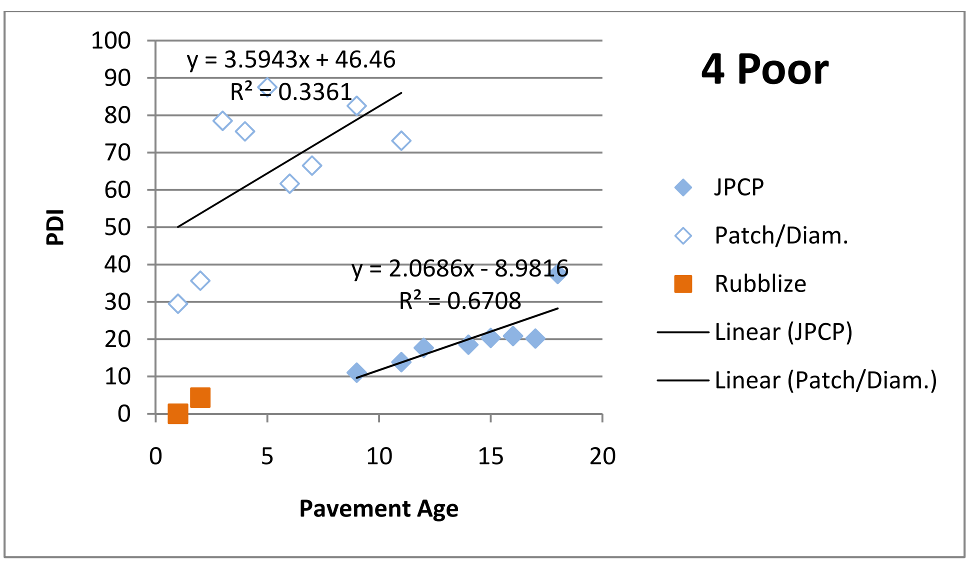


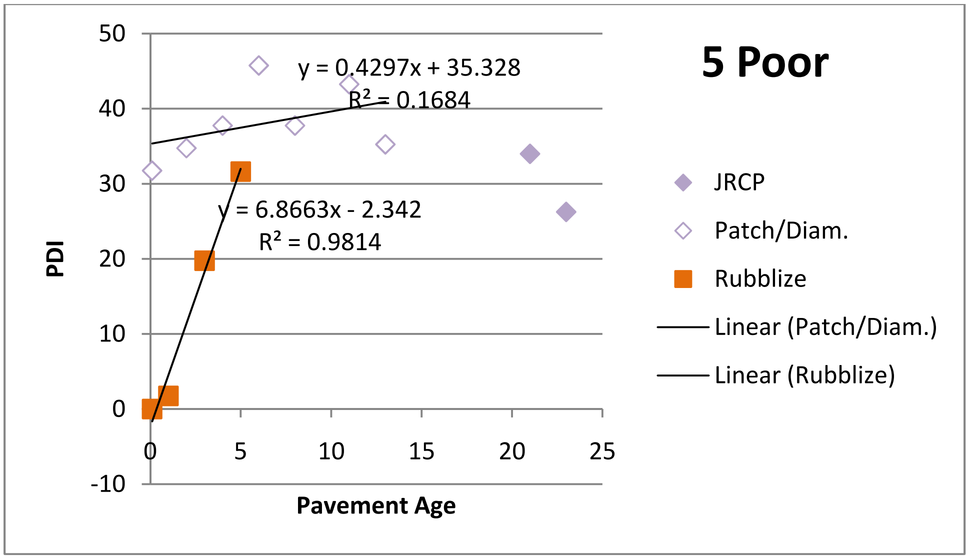


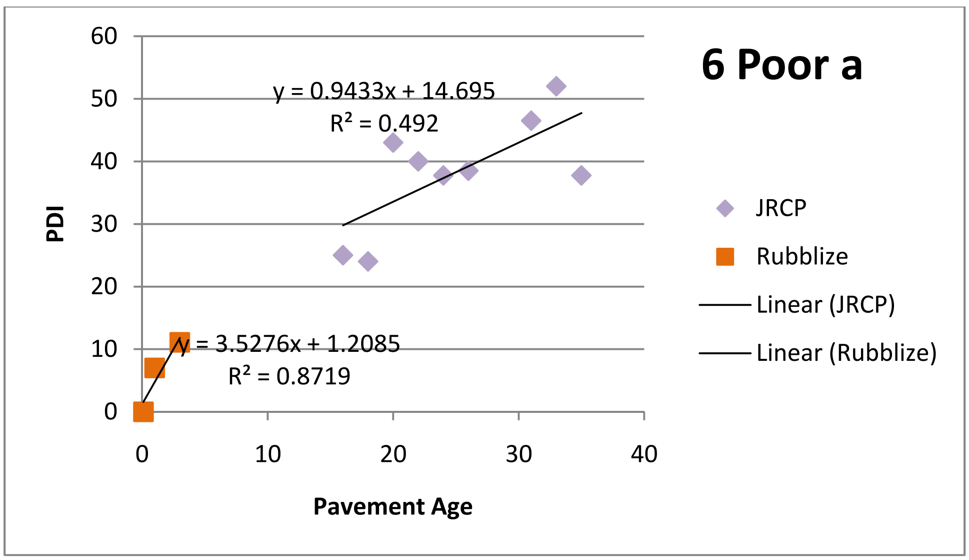
Project 1 Poor - Historic PDI data not available

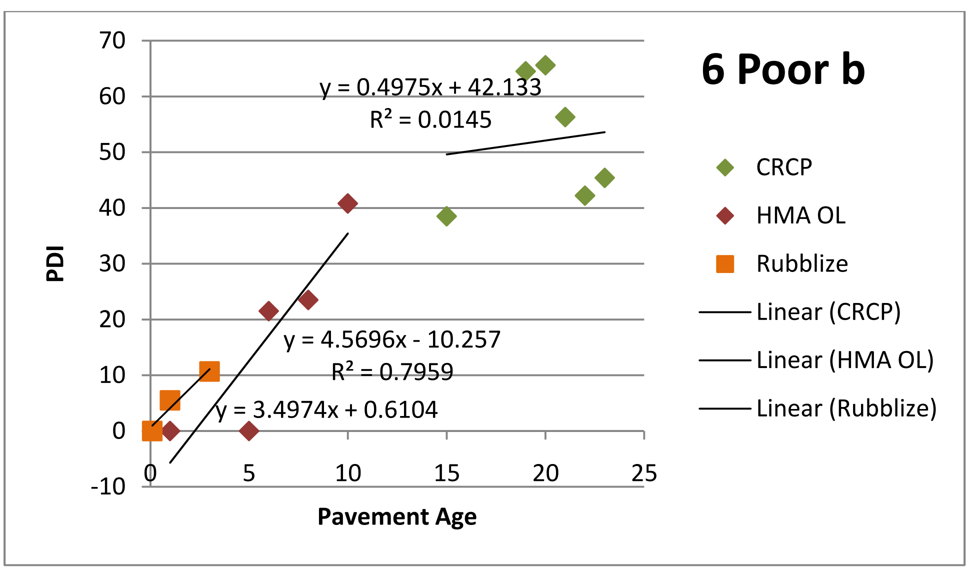

2 Poor
y = 0.6858x + 60.84
R² = 0.0962
y = 7.4991x - 17.164
R² = 0.8694
PDI
Pavement Age
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

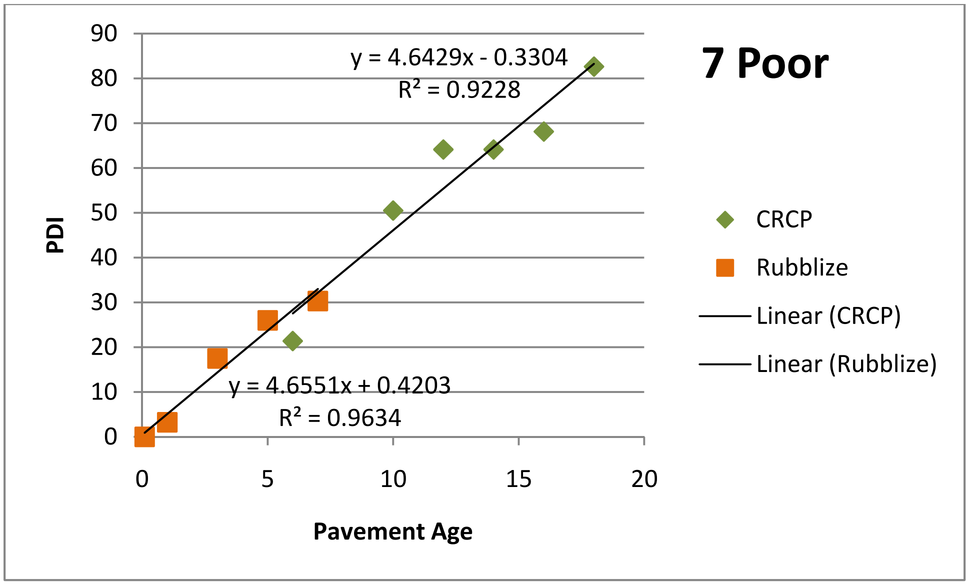

3 Poor
y = 1.5336x + 9.899
R² = 0.1805
y = 2.9889x - 1.7095
R² = 0.8641
PDI
Pavement Age
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

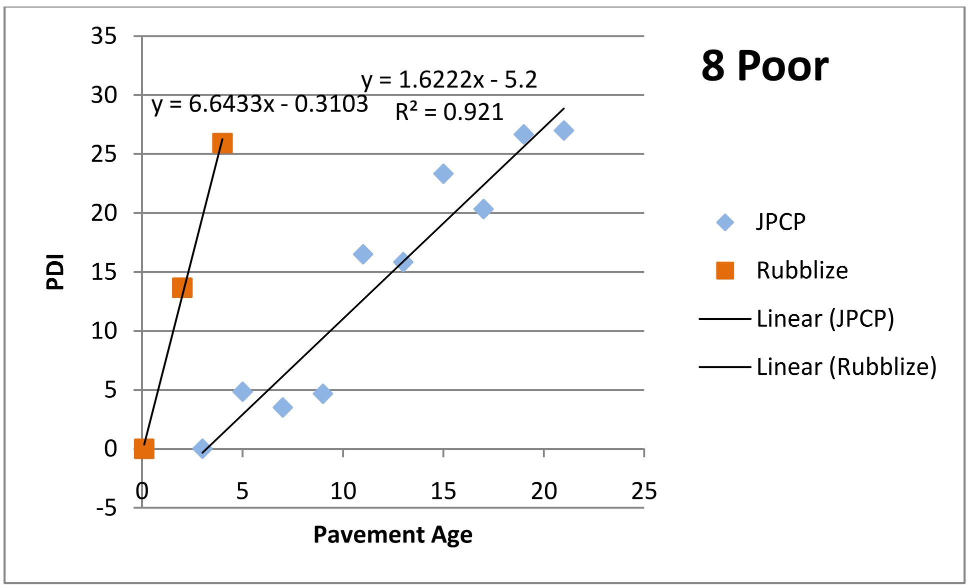

4 Poor
y = 3.5943x + 46.46
R² = 0.3361
y = 2.0686x - 8.9816
R² = 0.6708
PDI
Pavement Age
JPCP
Patch/Diam.
Rubblize
Linear (JPCP)
Linear (Patch/Diam.)


5 Poor
PDI
Pavement Age
y = 0.4297x + 35.328
R² = 0.1684
y = 6.8663x - 2.342
R² = 0.9814
JRCP
Patch/Diam.
Rubblize
Linear (Patch/Diam.)
Linear (Rubblize)

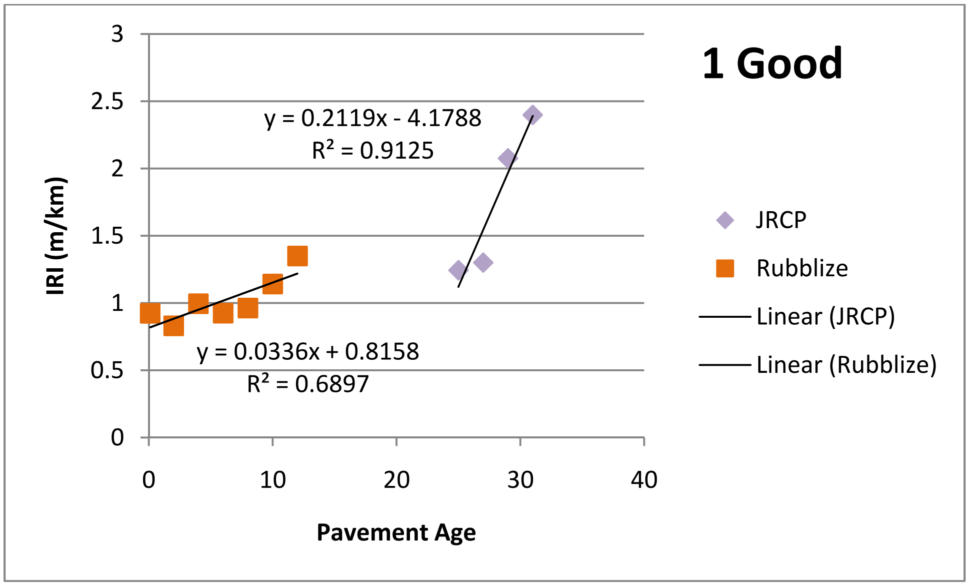

6 Poor a
PDI
Pavement Age
y = 0.9433x + 14.695
R² = 0.492
y = 3.5276x + 1.2085
R² = 0.8719
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

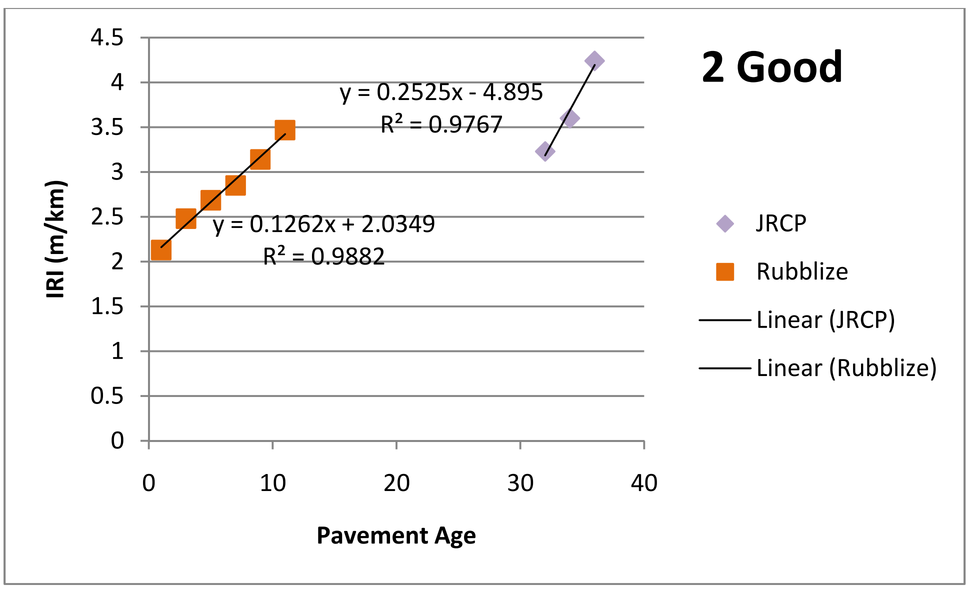

6 Poor b
PDI
Pavement Age
y = 0.4975x + 42.133
R² = 0.0145
y = 4.5696x - 10.257
R² = 0.7959
y = 3.4974x + 0.6104
CRCP
HMA OL
Rubblize
Linear (CRCP)
Linear (HMA OL)
Linear (Rubblize)

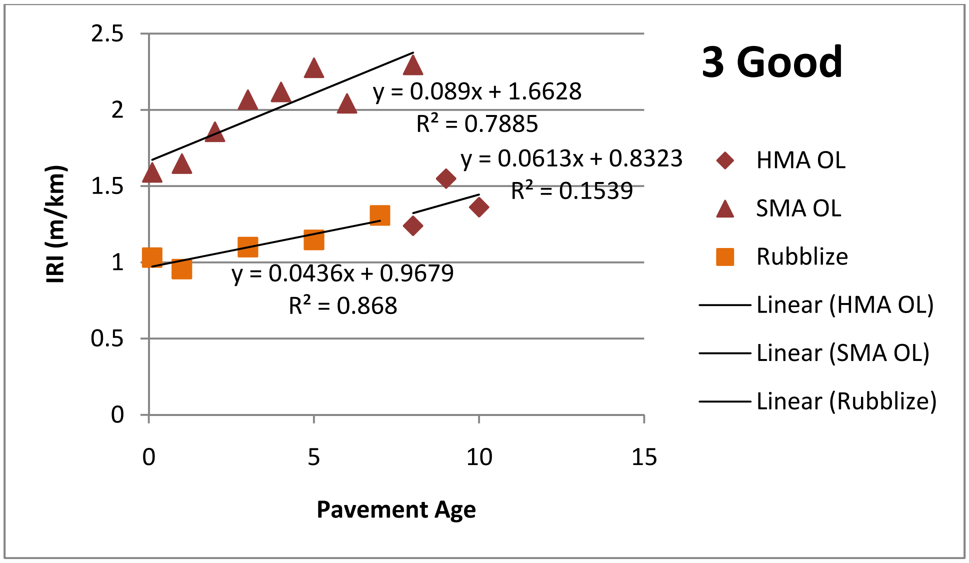

7 Poor
y = 4.6429x - 0.3304
R² = 0.9228
y = 4.6551x + 0.4203
R² = 0.9634
PDI
Pavement Age
CRCP
Rubblize
Linear (CRCP)
Linear (Rubblize)

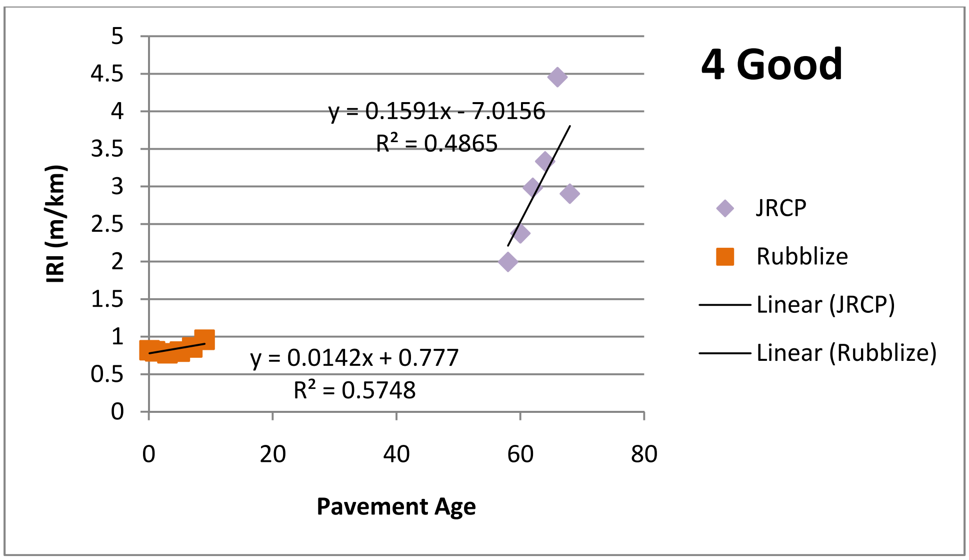

8 Poor
y = 6.6433x - 0.3103
y = 1.6222x - 5.2
R² = 0.921
PDI
Pavement Age
JPCP
Rubblize
Linear (JPCP)
Linear (Rubblize)

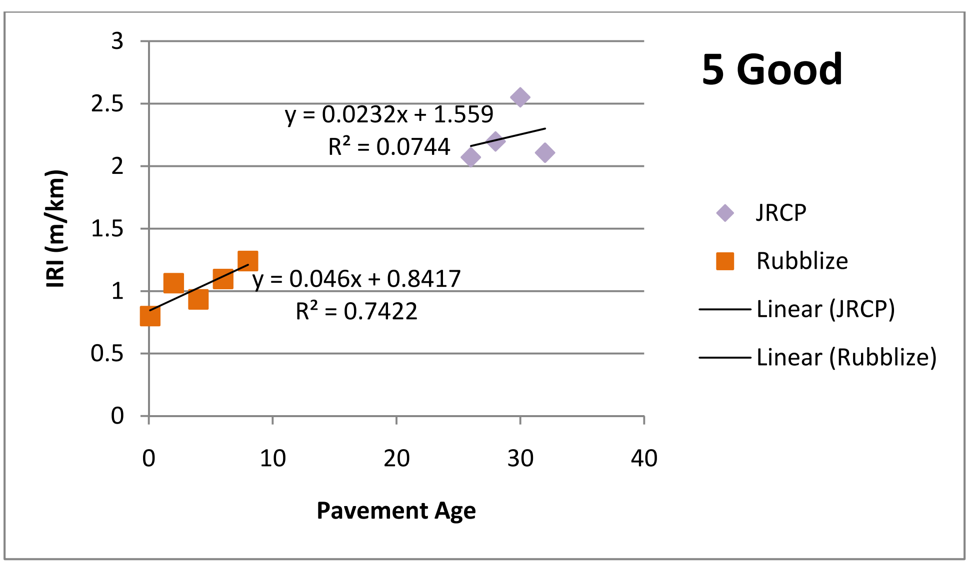
Appendix 3 - Plots of Historic IRI Data

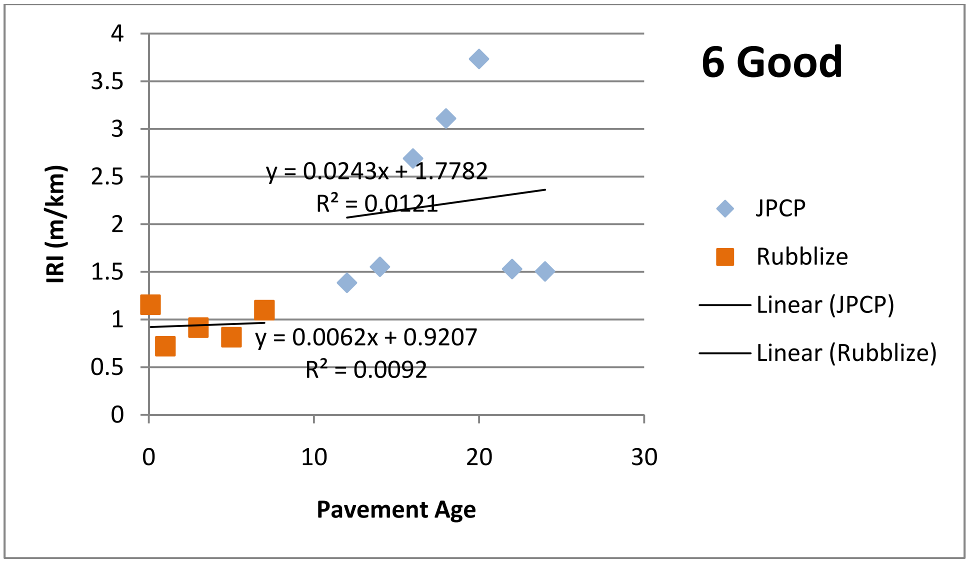


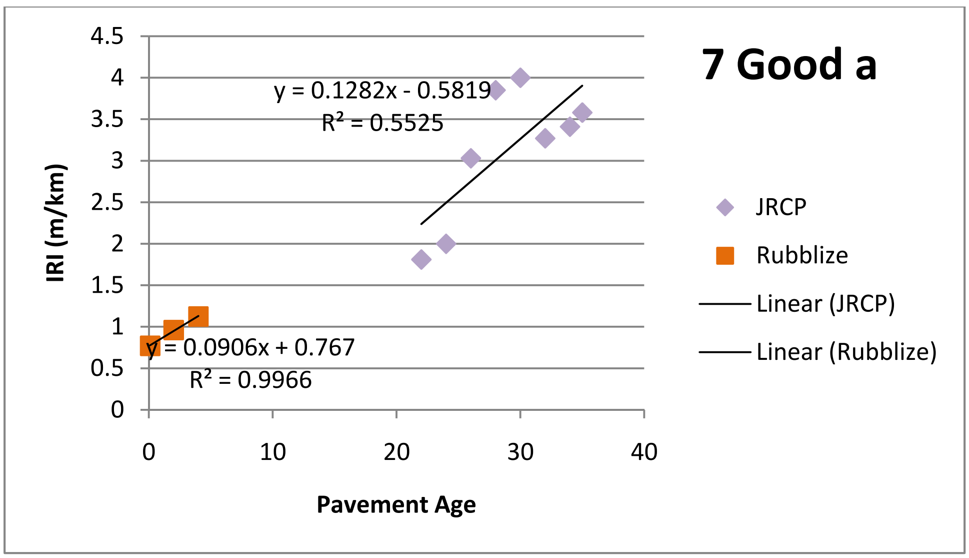


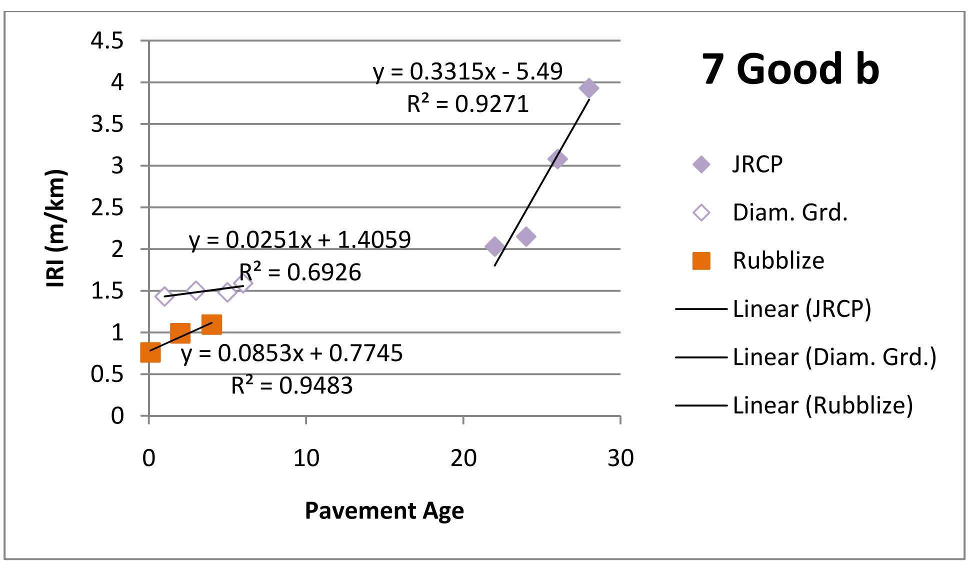

3 Good
IRI (m/km)
Pavement Age
y = 0.089x + 1.6628
R² = 0.7885
y = 0.0613x + 0.8323
R² = 0.1539
y = 0.0436x + 0.9679
R² = 0.868
HMA OL
SMA OL
Rubblize
Linear (HMA OL)
Linear (SMA OL)
Linear (Rubblize)

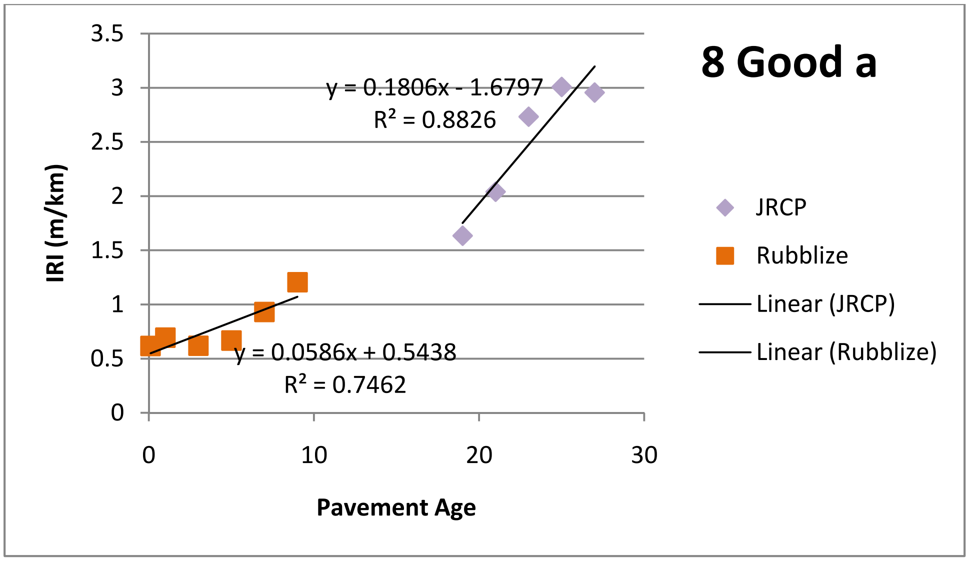

4 Good
IRI (m/km)
Pavement Age
y = 0.1591x - 7.0156
R² = 0.4865
y = 0.0142x + 0.777
R² = 0.5748
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

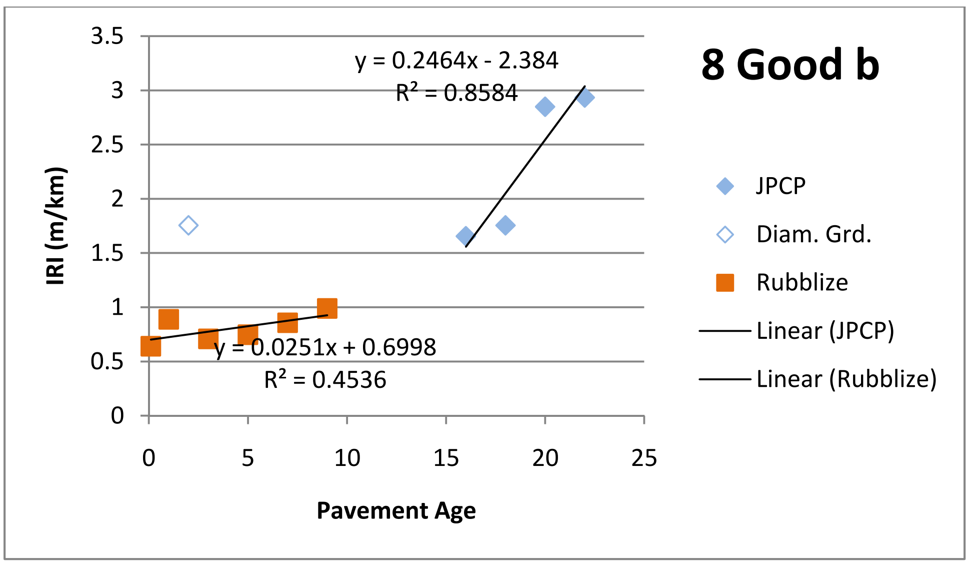

5 Good
IRI (m/km)
Pavement Age
y = 0.0232x + 1.559
R² = 0.0744
y = 0.046x + 0.8417
R² = 0.7422
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

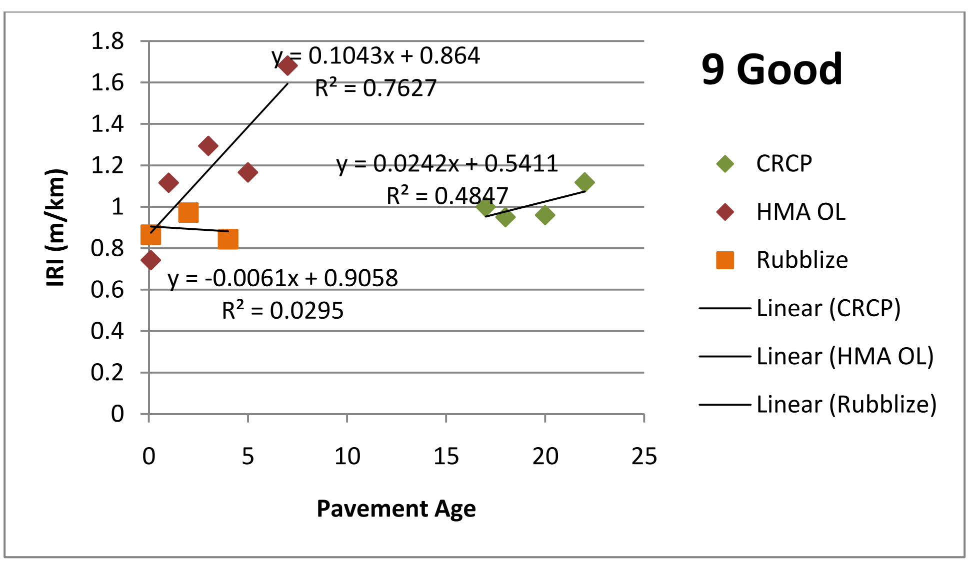

6 Good
IRI (m/km)
Pavement Age
y = 0.0243x + 1.7782
R² = 0.0121
y = 0.0062x + 0.9207
R² = 0.0092
JPCP
Rubblize
Linear (JPCP)
Linear (Rubblize)

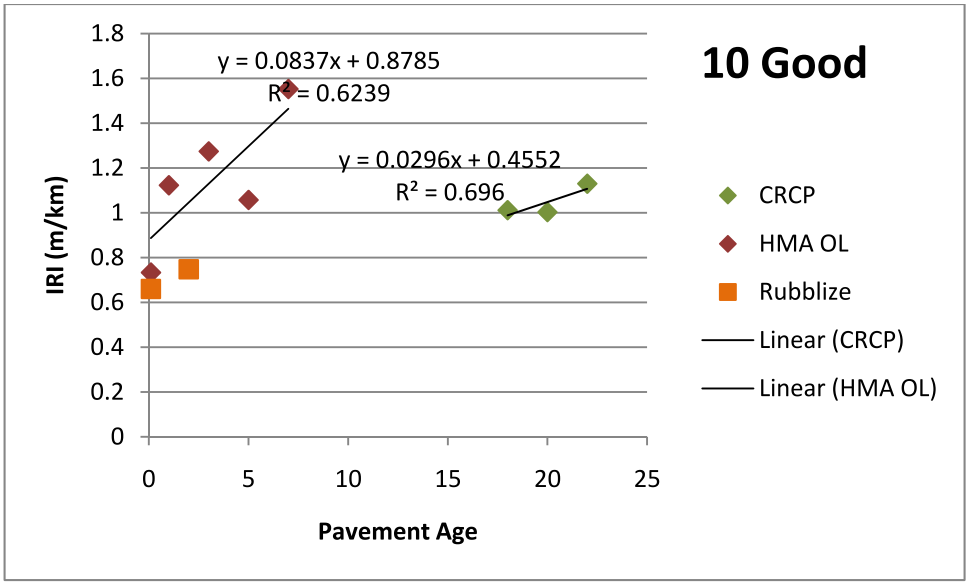

7 Good a
IRI (m/km)
Pavement Age
y = 0.1282x - 0.5819
R² = 0.5525
y = 0.0906x + 0.767
R² = 0.9966
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

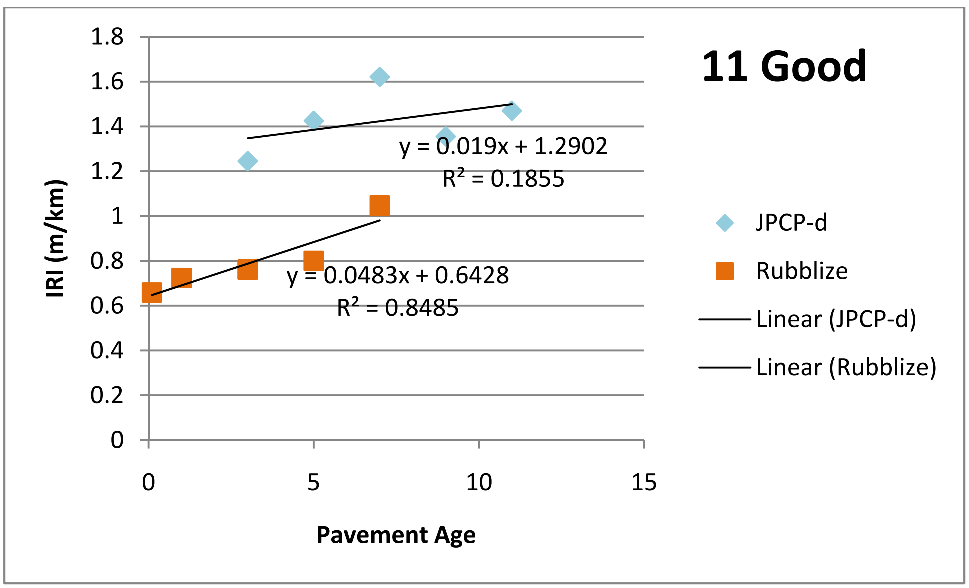

7 Good b
IRI (m/km)
Pavement Age
y = 0.3315x - 5.49
R² = 0.9271
y = 0.0251x + 1.4059
R² = 0.6926
y = 0.0853x + 0.7745
R² = 0.9483
JRCP
Diam. Grd.
Rubblize
Linear (JRCP)
Linear (Diam. Grd.)
Linear (Rubblize)


8 Good a
3.5
3
2.5
2
1.5
1
0.5
0
IRI (m/km)
0
10
20
30
Pavement Age
y = 0.1806x - 1.6797
R² = 0.8826
y = 0.0586x + 0.5438
R² = 0.7462
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

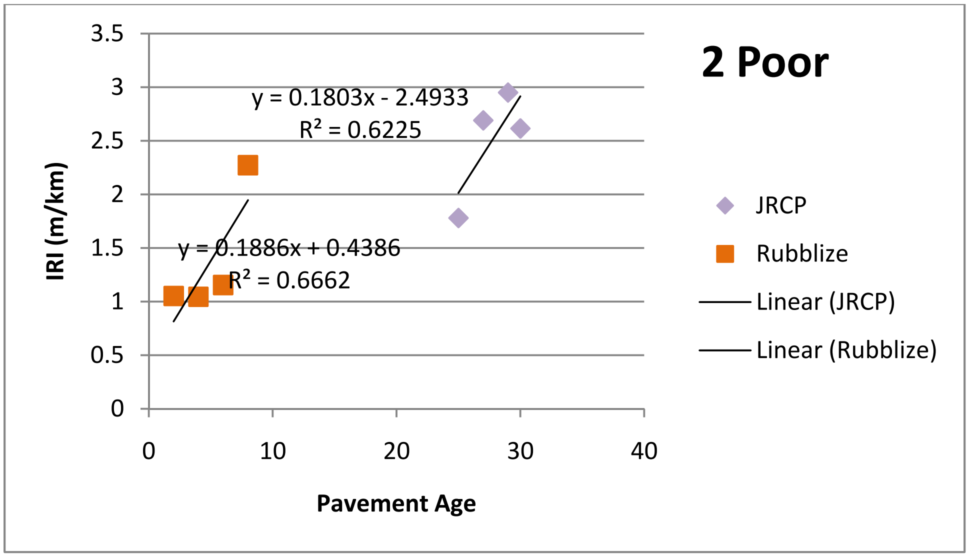

8 Good b
3.5
3
2.5
2
1.5
1
0.5
0
IRI (m/km)
0
5
10
15
20
25
Pavement Age
y = 0.2464x - 2.384
R² = 0.8584
y = 0.0251x + 0.6998
R² = 0.4536
JPCP
Diam. Grd.
Rubblize
Linear (JPCP)
Linear (Rubblize)

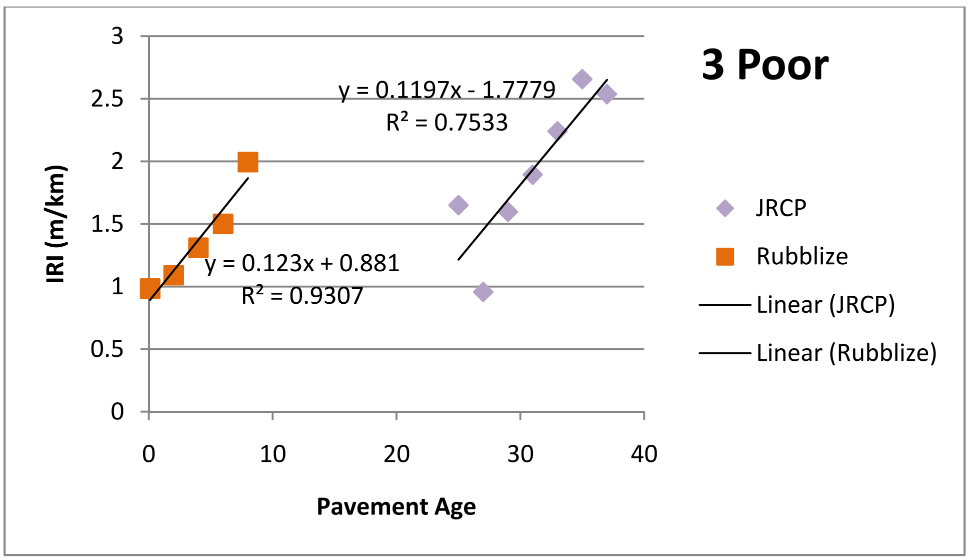

9 Good
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
IRI (m/km)
0
5
10
15
20
25
Pavement Age
y = 0.1043x + 0.864
R² = 0.7627
y = 0.0242x + 0.5411
R² = 0.4847
y = -0.0061x + 0.9058
R² = 0.0295
CRCP
HMA OL
Rubblize
Linear (CRCP)
Linear (HMA OL)
Linear (Rubblize)

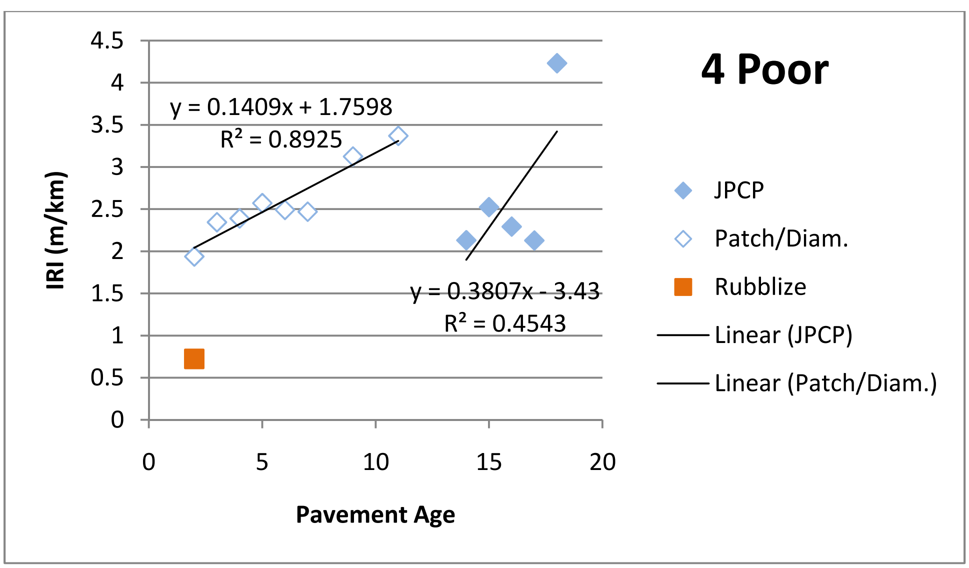




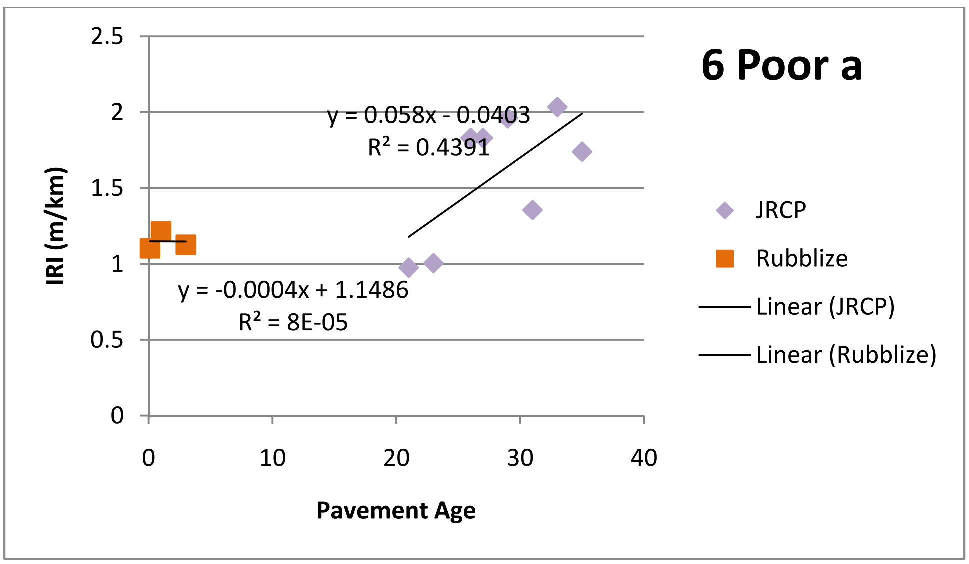
Project 1 Poor - Historic IRI data not available

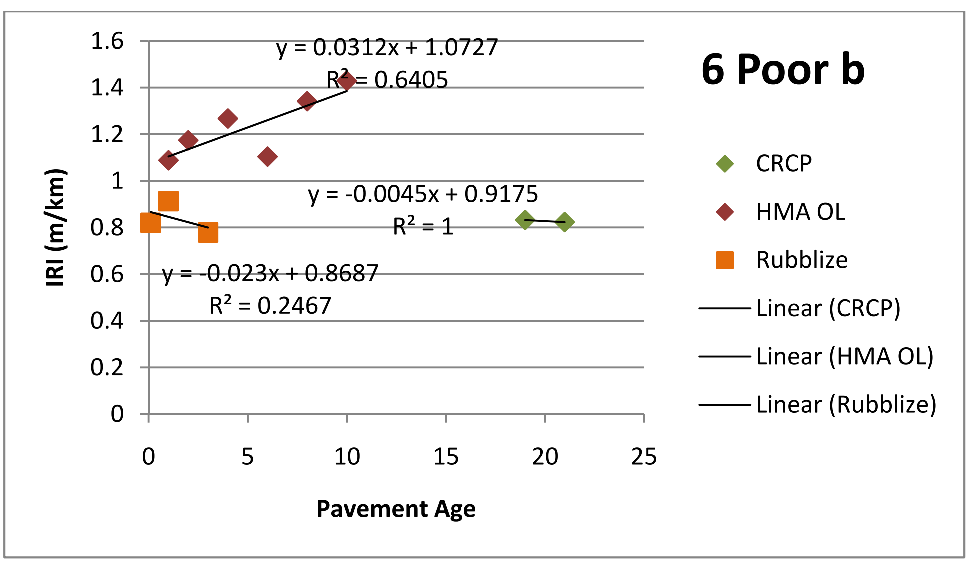

2 Poor
IRI (m/km)
Pavement Age
y = 0.1803x - 2.4933
R² = 0.6225
y = 0.1886x + 0.4386
R² = 0.6662
3.5
3
2.5
2
1.5
1
0.5
0
0
10
20
30
40
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

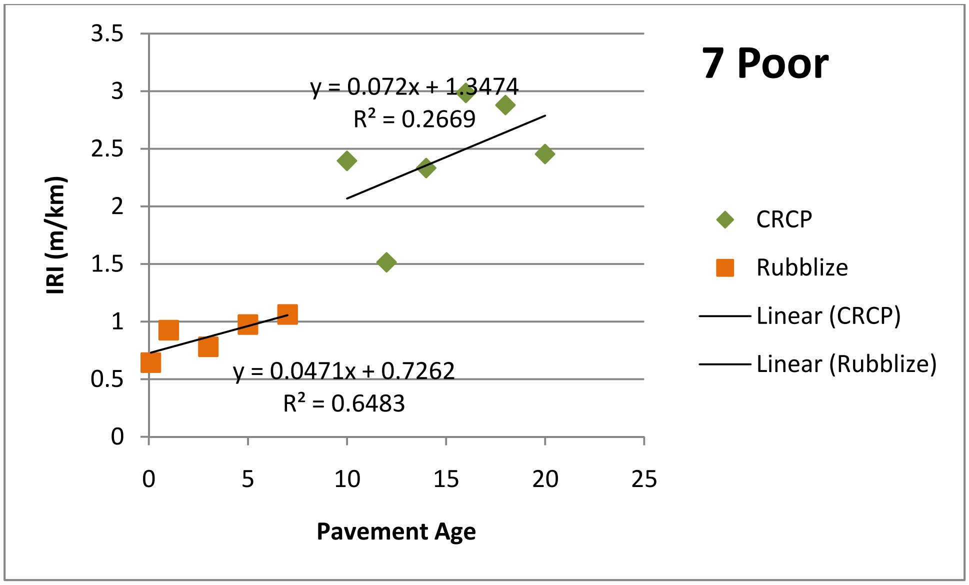

3 Poor
IRI (m/km)
Pavement Age
y = 0.1197x - 1.7779
R² = 0.7533
y = 0.123x + 0.881
R² = 0.9307
3
2.5
2
1.5
1
0.5
0
0
10
20
30
40
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)

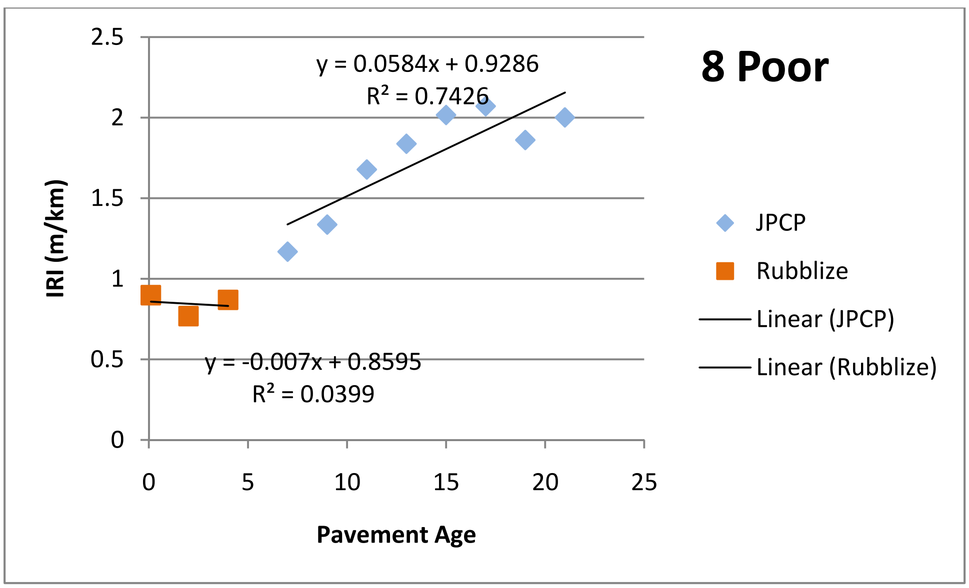

4 Poor
IRI (m/km)
Pavement Age
y = 0.1409x + 1.7598
R² = 0.8925
y = 0.3807x - 3.43
R² = 0.4543
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
5
10
15
20
JPCP
Patch/Diam.
Rubblize
Linear (JPCP)
Linear (Patch/Diam.)


5 Poor
y = 0.0871x + 1.203
R² = 0.6846
y = 0.1252x + 0.6829
R² = 0.9167
IRI (m/km)
Pavement Age
Patch/Diam.
Rubblize
Linear (Patch/Diam.)
Linear (Rubblize)


6 Poor a
y = 0.058x - 0.0403
R² = 0.4391
y = -0.0004x + 1.1486
R² = 8E-05
IRI (m/km)
Pavement Age
JRCP
Rubblize
Linear (JRCP)
Linear (Rubblize)


6 Poor b
y = 0.0312x + 1.0727
R² = 0.6405
y = -0.0045x + 0.9175
R² = 1
y = -0.023x + 0.8687
R² = 0.2467
IRI (m/km)
Pavement Age
CRCP
HMA OL
Rubblize
Linear (CRCP)
Linear (HMA OL)
Linear (Rubblize)


7 Poor
IRI (m/km)
3.5
3
2.5
2
1.5
1
0.5
0
0
5
10
15
20
25
Pavement Age
y = 0.072x + 1.3474
R² = 0.2669
y = 0.0471x + 0.7262
R² = 0.6483
CRCP
Rubblize
Linear (CRCP)
Linear (Rubblize)


8 Poor
IRI (m/km)
2.5
2
1.5
1
0.5
0
0
5
10
15
20
25
Pavement Age
y = 0.0584x + 0.9286
R² = 0.7426
y = -0.007x + 0.8595
R² = 0.0399
JPCP
Rubblize
Linear (JPCP)
Linear (Rubblize)

Appendix 4 - Final PDI Values for Concrete Pavements Prior to Rubblization

Project	PDI value of each surveyed segment																Average (all values)
1-Good	75	74	75	78	74	77	73	85	78	74	79	78	79	75	86	75	
2-Good	93																
3-Good	Data not available																
4-Good	67	9	47	40	20	22	3	3	68	71	48	76	47	54			
5-Good	68	71															
6-Good	48	76	47	54													
7-GoodEB	39																
7-GoodWB	33																
8-Gooda	39	45	73	45	53	67	49										
8-Goodb	10	4	3														
9-Good	94	94	68	69													
10-Good	72	96	93	70	70	93	85	74									**Good**
11-Good	19	53	53	33													**59**
1-Poor	Data not available																
2-Poor	61	59	69														
3-Poor	94	82															
4-Poor	76	72	77	81	72	61											
5-Poor	46	33	30	32													
6-Poora	35	36	34	46													
6-Poorb	26	50	49	51	46	46	26	57	57	46							
7-Poor	82	85	69	95	75	73	91	91									**Poor**
8-Poor	63	34	29	10	19												**56**

Appendix 5 - Final IRI Values for Concrete Pavements Prior to Rubblization

Project	**IRI value of each surveyed segment, m/km**																**Average (all values)**
1-Good	1.90	2.36	2.49	2.7	2.43	2.01	2.63	3.14	1.94	2.25	2.2	2.27	2.46	2.14	3.09	2.36	
2-Good	4.38	4.10															
3-Good	Data not available																
4-Good	3.38	3.13	3.05	2.92	2.87	3.52	2.04	3.06	3.03	2.84	2.71	2.70	3.22	2.18			
5-Good	1.96	2.07	2.29														
6-Good	4.01	3.75	3.88	3.3													
7-GoodEB	3.58																
7-GoodWB	1.59																
8-Gooda	2.79	3.58	3.09	2.68	2.41	3.01	2.72	3.48	3.15	2.77	2.77	3.03					
8-Goodb	2.07	1.42	1.88	2.03	1.34	1.79											
9-Good	1.18	1.15	1.13	1.02	1.11												
10-Good	1.13	1.13	1.15	1.04	1.05	1.25	1.16										**Good**
11-Good	1.80	1.64	1.28	1.23	1.8	1.58	1.29	1.14									**2.34**
1-Poor	Data not available																
2-Poor	2.57	2.76	2.32	2.81													
3-Poor	2.94	2.13	2.54														
4-Poor	3.30	3.17	3.55	3.41	3.06	3.19											
5-Poor	2.57	2.13	1.91	2.45	2.38	2.29											
6-Poora	1.72	1.88	1.58	1.78													
6-Poorb	0.86	0.88	1.01	0.80	0.70	0.70	0.81	0.70	1.05	0.72							
7-Poor	2.45	2.21	2.45	2.78	2.32	2.05	2.84	2.54									**Poor**
8-Poor	1.63	2.18	2.23	2.1	1.93	1.94											**2.10**

Appendix 6 - Joint and Crack Repair Options and Construction Sequence

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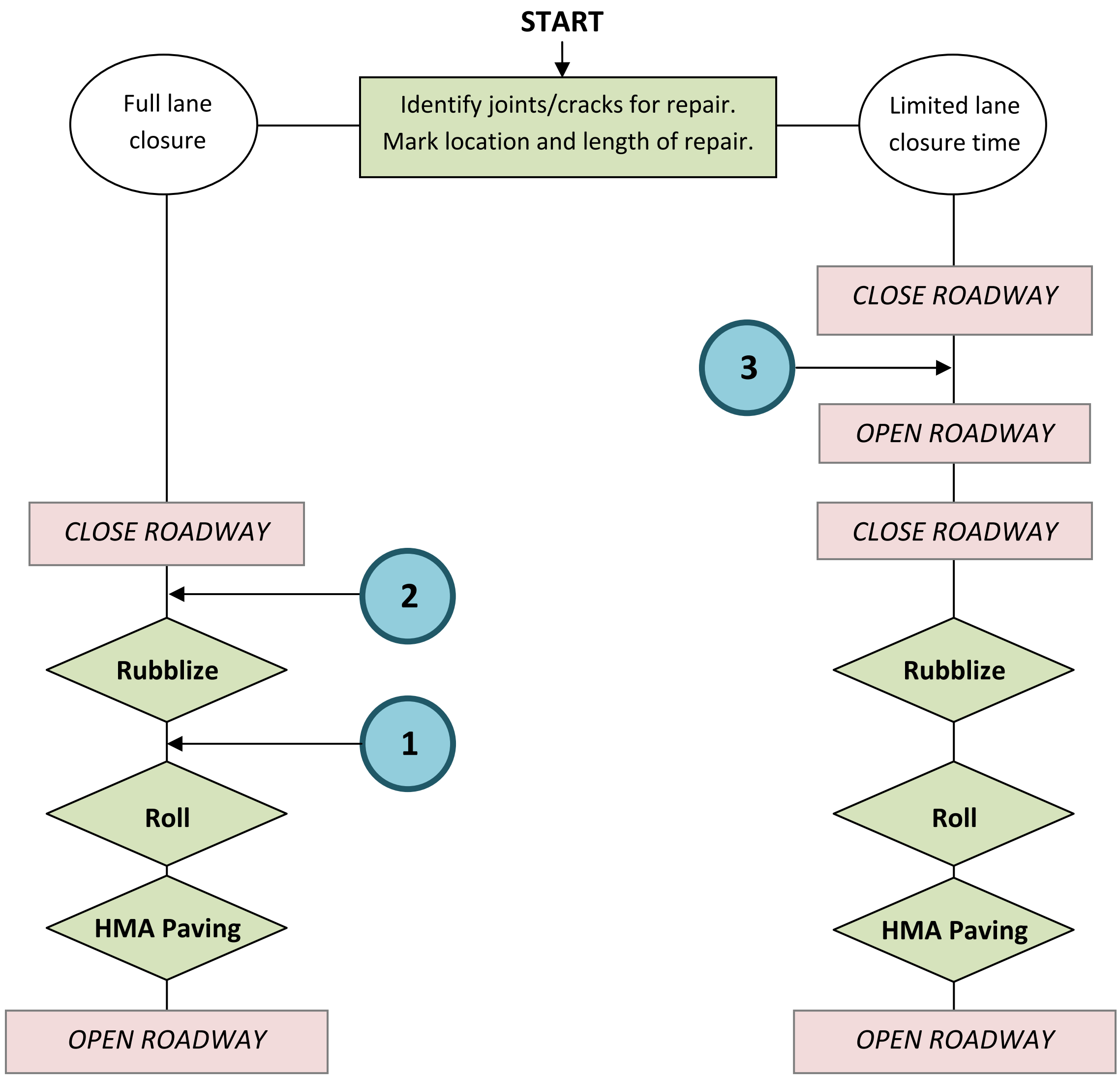

Repair options:

1. **Aggregate base patching after rubblization** - Ensure joints/cracks for base patching are marked such that they can be located after rubblization. Remove rubblized concrete full-width, full-depth, and at a length to adequately remove all deteriorated concrete. Base patch with CABC. Complete repair at locations identified prior to construction, along with additional locations noted during rubblization.

2. **Aggregate base patching prior to rubblization** - Saw cut concrete on both sides of joint/crack. Determine width of removal based on amount of deterioration; intact concrete can be left in place. Remove deteriorated concrete full-depth. Base patch with CABC. Complete repair at locations identified prior to construction. Do not rubblize repaired locations.

3. **Concrete base patching prior to rubblization** - Saw cut concrete on both sides of joint/crack. Determine width of removal based on amount of deterioration; intact concrete can be left in place. Remove deteriorated concrete full-depth. Place the concrete patch (nondowelled). Complete repair at locations identified prior to construction. Rubblize repaired areas along with the remainder of the concrete pavement.

Appendix 7 - Sample Flow Chart for Rubblizing Operation

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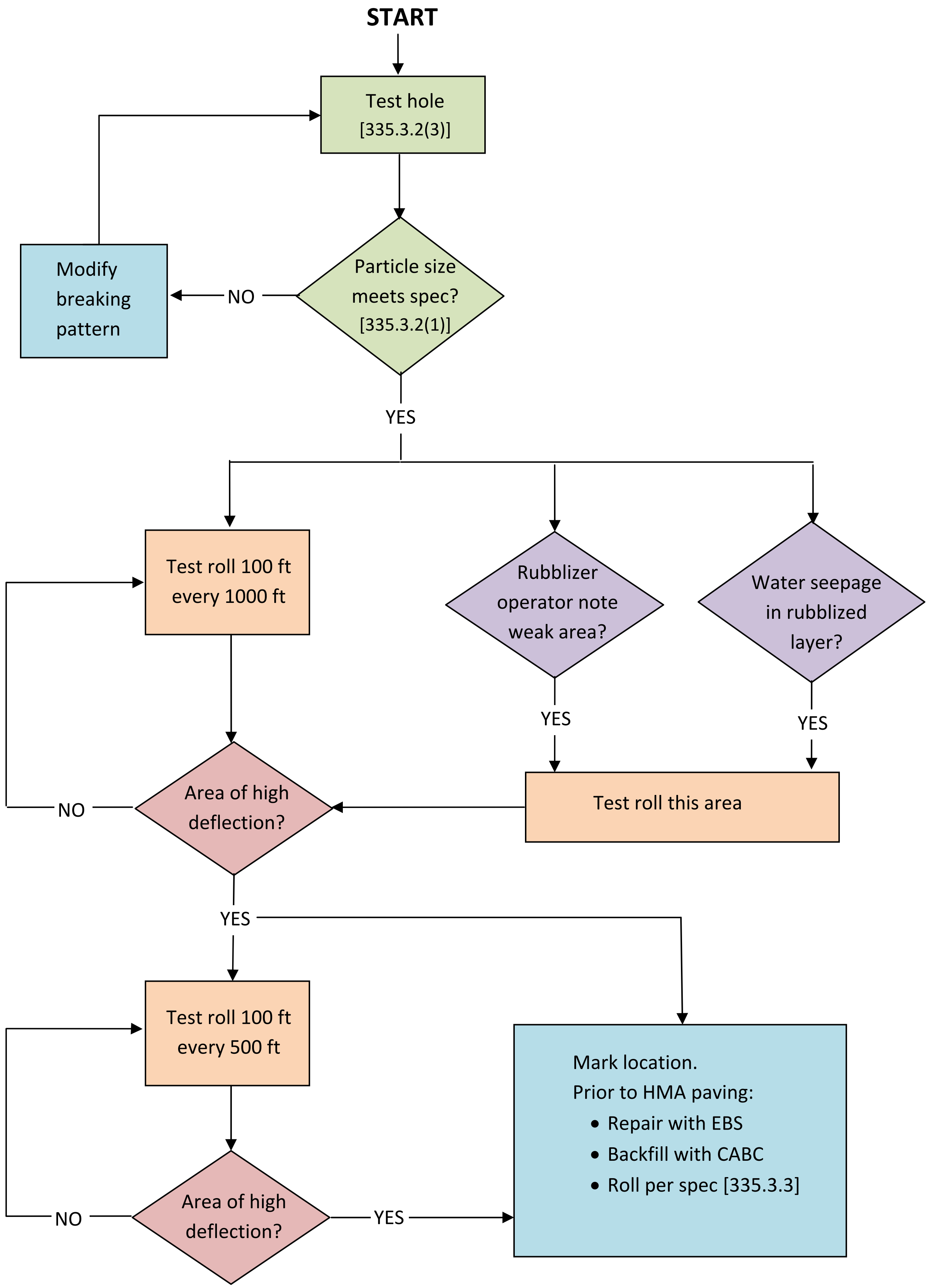
START
Test hole
[335.3.2(3)]
Particle size
meets spec?
[335.3.2(1)]
NO
Modify
breaking
pattern
YES
Test roll 100 ft
every 1000 ft
Rubblizer
operator note
weak area?
Water seepage
in rubblized
layer?
YES
YES
Test roll this area
NO
Area of high
deflection?
YES
Test roll 100 ft
every 500 ft
Mark location.
Prior to HMA paving:
• Repair with EBS
• Backfill with CABC
• Roll per spec [335.3.3]
NO
Area of high
deflection?
YES