Reflective Crack Mitigation Guide for Flexible Pavements

Final Report
September 2015

ASPHALT MATERIALS & PAVEMENTS PROGRAM

IOWA STATE UNIVERSITY
Institute for Transportation

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Reflective cracks form in pavements when hot-mix asphalt (HMA) overlays are placed over jointed and/or severely cracked rigid and flexible pavements. In the first part of the research, survival analysis was conducted to identify the most appropriate rehabilitation method for composite pavements and to evaluate the influence of different factors on reflective crack development. Four rehabilitation methods, including mill and fill, overlay, heater scarification (SCR), and rubblization, were analyzed using three performance indicators: reflective cracking, international roughness index (IRI), and pavement condition index (PCI). It was found that rubblization can significantly retard reflective cracking development compared to the other three methods. No significant difference for PCI was seen among the four rehabilitation methods. Heater scarification showed the lowest survival probability for both reflective cracking and IRI, while an overlay resulted in the poorest overall pavement condition based on PCI. In addition, traffic level was found not to be a significant factor for reflective cracking development. An increase in overlay thickness can significantly delay the propagation of reflective cracking for all four treatments. Soil types in rubblization pavement sites were assessed, and no close relationship was found between rubblized pavement performance and subgrade soil condition.

In the second part of the research, the study objective was to evaluate the modulus and performance of four reflective cracking treatments: full rubblization, modified rubblization, crack and seat, and rock interlayer. A total of 16 pavement sites were tested by the surface wave method (SWM), and in the first four sites both falling weight deflectometer (FWD) and SWM were conducted for a preliminary analysis. The SWM gave close concrete layer moduli compared to the FWD moduli on a conventional composite pavement. However, the SWM provided higher moduli for the rubblized concrete layer. After the preliminary analysis, another 12 pavement sites were tested by the SWM. The results showed that the crack and seat method provided the highest moduli, followed by the modified rubblization method. The full rubblization and the rock interlayer methods gave similar, but lower, moduli. Pavement performance surveys were also conducted during the field study. In general, none of the pavement sites had rutting problems. The conventional composite pavement site had the largest amount of reflective cracking. A moderate amount of reflective cracking was observed for the two pavement sites with full rubblization. Pavements with the rock interlayer and modified rubblization treatments had much less reflective cracking. It is recommended that use of the modified rubblization and rock interlayer treatments for reflective cracking mitigation are best.</td></tr>
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REFLECTIVE CRACK MITIGATION GUIDE FOR FLEXIBLE PAVEMENTS

Final Report
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EXECUTIVE SUMMARY

Reflective cracks form in pavements when hot-mix asphalt (HMA) overlays are placed over jointed and/or severely cracked rigid and flexible pavements. They are the result of horizontal and vertical movements at the joints and cracks in the underlying pavements.

In the first part of the research, survival analysis was conducted to identify the most appropriate pavement rehabilitation method for composite pavements and to evaluate the influence of different factors on reflective crack development in composite pavement. Four composite pavement rehabilitation methods, including mill and fill, overlay, heater scarification (SCR), and rubblization, were analyzed using three pavement performance indicators: reflective cracking, international roughness index (IRI), and pavement condition index (PCI). It was found that rubblization can significantly retard reflective cracking development compared to the other three methods. No significant difference for PCI was seen in the survival analysis for the four rehabilitation methods. Heater scarification showed the lowest survival probability for both reflective cracking and IRI, while overlay resulted in the poorest overall pavement condition based on PCI. In addition, traffic level was found to not be a significant factor for reflective cracking development. An increase in overlay thickness can significantly delay the propagation of reflective cracking for all four treatments. Soil types at rubblization pavement sites were assessed, and no close relationship was found between rubblized pavement performance and subgrade soil condition.

In the second part of this research, the study objective was to evaluate the modulus and performance of four reflective cracking treatments, which included full rubblization, modified rubblization, crack and seat, and rock interlayer. A total of 16 pavement sites were tested using the surface wave method (SWM), and in the first four sites both falling weight deflectometer (FWD) and SWM were conducted for a preliminary analysis. The SWM gave close concrete layer moduli compared to the FWD moduli on a conventional composite pavement. However, the SWM provided higher moduli for the rubblized concrete layer. After the preliminary analysis was completed, another 12 pavement sites were tested using the SWM. The results show that the crack and seat method provided the highest moduli, followed by the modified rubblization. The full rubblization and the rock interlayer gave similar but lower moduli. Pavement performance surveys were also conducted during the field study. In general, none of the pavement sites had rutting problems. The conventional composite pavement site had the largest amount of reflective cracking. A moderate amount of reflective cracking was observed for the two pavement sites with full rubblization. Pavements with the rock interlayer and modified rubblization treatments had much less reflective cracking. It is recommended that use of the modified rubblization and rock interlayer treatments for reflective cracking mitigation are best.

In the final part of this research, an analysis of cold in-place recycling (CIR) data for 100 cold in-place recycling projects was completed. The presented CIR performance data showed an overall improvement in pavement performance post-rehabilitation. This information can be used as guidance for assisting with making future decisions for pavement rehabilitation at the network level. However, appropriate CIR pavement selection is still required for obtaining good performance. The overall pavement smoothness as measured by IRI was improved after CIR

rehabilitation. The sections were categorized by CIR thickness, and the data showed that the thicker layers remained smoother longer. A model was developed to capture this phenomenon. The overall model is preliminary due to the lack of data at the lower thicknesses and the low number of projects observed 11 years post-rehabilitation, but a residual plot shows that the model captures the overall average of the data fairly well.

CHAPTER 1 INTRODUCTION

1.1 Problem Statement

Reflective cracking of asphalt mixtures is a common distress that results in a loss of pavement ride quality and service life. Several strategies exist to mitigate reflective cracking depending on the pavement structure, including the use of crack relief layers in the form of membranes and specialty asphalt mixtures (e.g., Strata), crack and seat, rubblization, cold in-place recycling (CIR) of existing asphalt overlays, and full-depth reclamation (FDR). Depending on the pavement structure, pavement condition, and traffic level, varying strategies exist that improve the performance of the pavement economically.

Despite the availability of numerous crack mitigation strategies, many of these strategies do not contain construction criteria that assist in ensuring the strategies' intended design life. One such example is the use of rubblization. There have been many instances where a pavement has been rubblized, yet upon later investigation the pavement was found to be only rubblized in the top three to four inches and not the full depth of the concrete. There has also been substantial variation in the fracture particle size of rubblized pavements.

The Iowa Department of Transportation (DOT) currently does not have a guideline or specification for reflective cracking control and mitigation in conventional composite pavement. A standard technical guide is needed for Iowa to provide detailed guidance on choosing the optimal reflective cracking mitigation strategy for a project. The guide should provide pavement designers with a crack control selection method that is, in part, based upon a reliability-based analysis and lifecycle cost analyses. It also needs to specifically address rubblization and crack and seat mitigation techniques by giving recommendations for construction specifications and structural capacity based on the most advanced research available. In addition, newly developed rock interlayers have been commonly used in Iowa's county roads, and the performance data are readily available to the research team, including the original material properties and designs. The study also needs to verify the practicability of the rock interlayer in Iowa.

1.2 Objectives

The first objective was to use Iowa's Pavement Management Information System (PMIS) for reflective cracking mitigation strategy selection at the network level. This involved collecting and analyzing pavement structure, traffic, and field performance data in Iowa composite pavements through a survival analysis. The second objective was to perform project-level pavement site investigations. This included pavement condition surveys, pavement structural moduli testing by falling weight deflectometer (FWD), and surface wave method (SWM) testing.

1.3 Report Organization

The report consists of five chapters, including this introduction as the first. The second chapter provides a literature review, which consists of the causes and mechanisms of reflective cracking, common types of reflective cracking mitigation strategies, and the decision tree for appropriate strategy selection. The third chapter compares the survival time of four different composite pavement rehabilitation methods/reflective cracking mitigation methods and evaluates the influence of different factors on reflective cracking development in composite pavement by parametric survival analysis. The fourth chapter evaluates the performance of the four different reflective cracking treatments by in situ modulus and pavement condition evaluation. The fifth chapter provides an economic analysis of the strategies for mitigating reflective cracking. There were not sufficient data for the cold in-place recycling for an economic analysis. The sixth chapter reviews the performance of cold in-place recycling projects. Finally, the seventh chapter outlines the findings, conclusions, and recommendations.

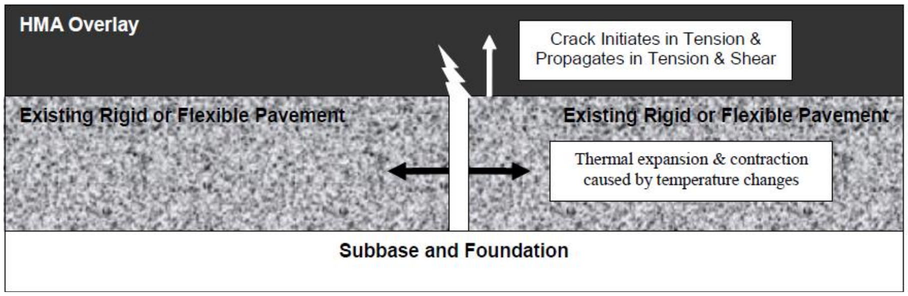
The pavement data extracted from the PMIS and Iowa Pavement Management Program (IPMP) databases and used for survival analysis in this study are listed in Appendix A. FWD back-calculated pavement layer moduli used in the project-level testing are presented in Appendix B. Appendix C contains the SWM dispersion curve data collected in the study as well as the back-calculated SWM moduli. Finally, selected pictures from the field visual distress surveys are provided in Appendix D.

CHAPTER 2 LITERATURE REVIEW

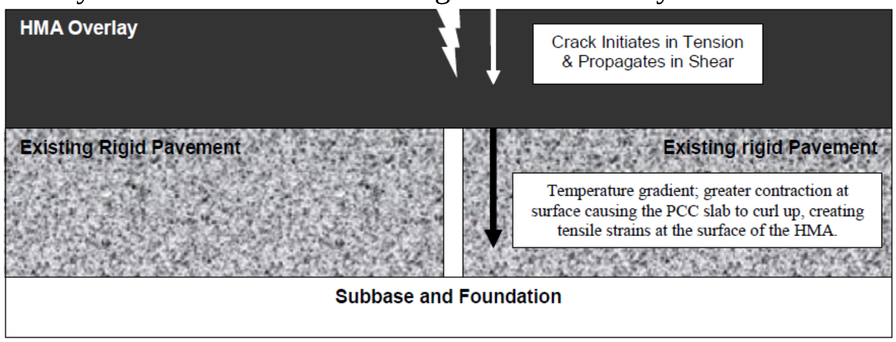
2.1 Causes and Mechanisms of Reflective Cracking

Reflective cracking is one of the most common types of distresses that occur early in the service life of composite pavements. When hot-mix asphalt (HMA) overlays are placed over jointed or severely cracked Portland cement concrete (PCC) or HMA pavements, they rapidly propagate through the HMA overlay thickness and reflect to the surface causing reflective cracks. Although reflective cracks do not generally reduce the structural capacity of a pavement, subsequent ingress of moisture and the effects of the natural environment and traffic can results in premature distress and even failure of the pavement.

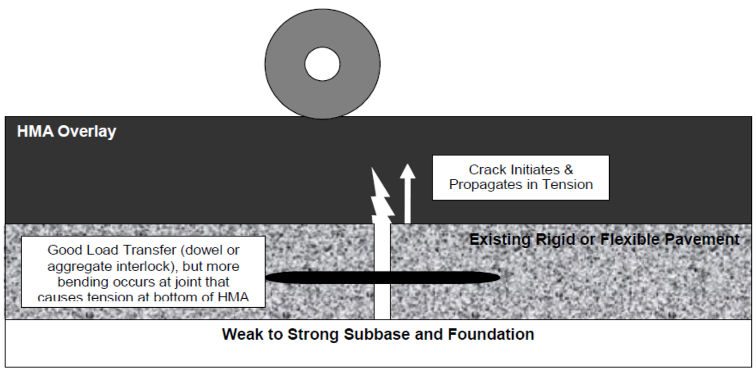
Reflective cracks propagate through the HMA overlay surface due to the movement at the crack (joint in case of existing concrete pavements) producing tensile stresses which are caused by (a) discontinuities in the underlying layers, (b) differential temperature conditions, and (c) longitudinal cracks in the old surface (Roberts et al. 1996). Schematic diagrams of thermally-induced and traffic-induced reflective cracking mechanisms are shown in Figure 1 (Von Quintus et al. 2009).



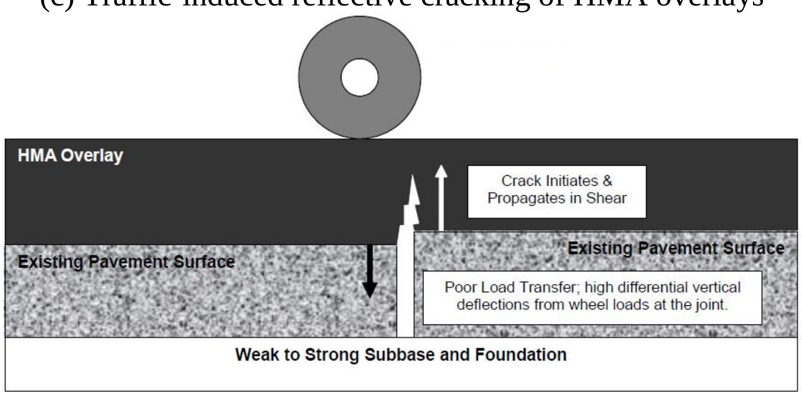
(a) Thermally-induced reflective cracking of HMA overlays: horizontal movements



(b) Thermally-induced reflective cracking of HMA overlays: curling of PCC slab



(c) Traffic-induced reflective cracking of HMA overlays

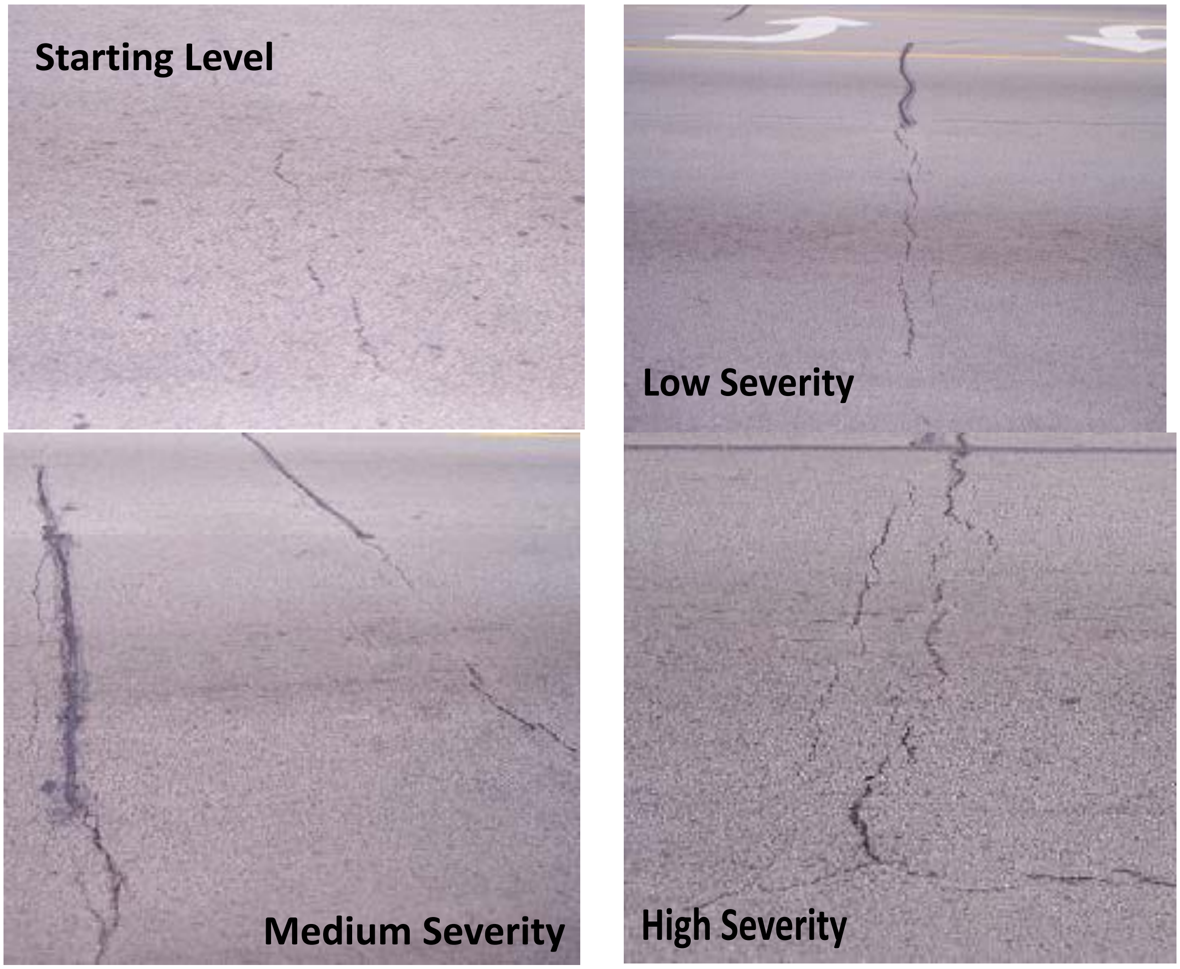


(d) Traffic-induced reflective cracking of HMA overlays

Von Quintus et al. 2009

Figure 1. Mechanisms of reflective cracking of HMA overlays

The combined effect of traffic and environmental loadings is considered to cause reflective cracks which can initiate either at the top or bottom of the HMA overlays. The rate of propagation of the reflective cracks is dependent on a number of factors including the thickness of the overlay, properties of the HMA overlay, type of reinforcement (if used), and the subgrade condition (Von Quintus et al. 2009). Reflective cracks observed in HMA overlays at different levels of severity are shown in Figure 2 (Al-Qadi et al. 2009).



Al-Qadi et al. 2009

Figure 2. Reflective cracking severity levels

Von Quintus et al. (2009) summarized the most commonly attributed factors that cause movements at joints and cracks in the existing pavement (termed as trigger factors for reflective cracking) as follows:

- Low temperatures (temperature drop)
- Wheel loads
- Freeze-thaw cycles
- Aging of HMA near surface (air voids level)

- Shrinkage of PCC, HMA, and cement-treated base (CTB)

2.2 Reflective Cracking Mitigation Strategies

The following are the various pre-overlay techniques used by different states to mitigate reflective cracking in existing HMA and PCC pavements (Von Quintus et al. 2009, Bandaru 2010):

- **Modification/Treatment of existing pavement surface**
  - Existing PCC surface
    - Crack and seat or break-and-seat
    - Rubblization
  - Existing HMA surface
    - Mill and replace wearing surface
    - Heater scarification (SCR)
    - Hot in-place recycling (HIPR)
    - Cold in-place recycling
    - Full-depth reclamation

- **Pre-overlay repairs of existing pavement surface**
  - Undersealing PCC slabs
  - HMA inlay
  - HMA patches
  - Use of leveling courses

- **Stress/Strain relieving interlayer**
  - Stress absorption membrane interlayer (SAMI)
  - Geosynthetic fabrics
  - Soft asphalt interlayer
  - Rubber modified asphalt interlayer
  - Strata reflective crack relief system
  - Interlayer stress absorbing composite (ISAC)
  - Bond breaker

- **HMA mixture modification**
  - Polymer-modified asphalt
  - Rubberized asphalt
  - Stone matrix asphalt
  - Sulfur asphalt
  - Carbon black

- **HMA overlay reinforcement**
  - Steel-reinforcing nettings
  - Geotextiles
  - Geogrids

    - Geocomposites
    - Geomembranes

- **Crack control**
    - Sawing and sealing joints in HMA overlays
    - Chip seal (HMA surface treatment)

Bennert (2010) recently completed a national survey on the reflective cracking experience of different states in the US. A total of 26 state highway agencies (SHAs), which reported that they overlay PCC pavements with HMA, participated in this survey and Iowa was one of the participants. Based on the survey results, the answers to the following questions were analyzed: relationship between the aggregate base type and years until reflective cracking observed, relationship between joint spacing and time until reflective cracking observed, common PCC treatment used by SHA prior to HMA overlay, etc. A majority of the SHAs (22 or 85 percent) reported that reflective cracking was observed within the first four years of the placement of the HMA overlay while seven SHAs reported observing reflective cracking within the first two years.

An overall conclusion drawn by Bennert (2010) based on the results of the national survey was that “there currently exists a large gap in the current practice of evaluating the potential for reflective cracking of asphalt overlays when placed on composite/rigid pavements.” Similarly, Loria-Salazar (2008) conducted a comprehensive literature review on reflective cracking mechanisms and mitigation techniques that is summarized in Table 1.

Table 1. Summary of a recent review on reflective cracking treatments

Treatment	Description	Performance
Cold in-place recycling	Remove and mill the upper layers of the existing pavement with specialized recycling equipment then mix with virgin materials to produce a strong flexible base course	Promising performance for roads with up to 13,000 ADT and 200,000annual equivalent single axle loads
Glassgrid	Geosynthetic material consisting of connected parallel sets of intersecting ribs with openings of sufficient size	Benefits in retarding or preventing reflective cracking are not clear. Field performance has varied from excellent to very poor. Concerns when used on rough surfaces
Fabric interlayer	Geosynthetic comprised solely of textiles. A paving fabric interlayer provides the generally acknowledged functions of stress-absorbing interlayer and a waterproofing membrane. The stress-related performance has been easily verified by the observed reductions of cracking in pavement overlays	Effective when used for load-related fatigue distress. It did not perform well when used to delay or retard thermal cracking. Optimum performance highly associated with proper construction procedures. The key factor is proper reinforced with fabrics have shown better performance than unreinforced overlays under same conditions
Asphalt rubber	Asphalt rubber chip seal overlaid with conventional dense graded HMA or gap graded HMA	Reduce or delay reflective cracking for a period of five years
Stress absorbing membrane	A thin layer placed between an underlying pavement and an HMA overlay for the purpose of dissipating movements and stresses at a crack in the underlying pavement before they create stresses in the overlay. SAMIs consist of a spray application at the stress relieving material, followed by placing and seating aggregate chips	Successful in reducing the rate of reflective cracking.
Crumb rubber overlay	Produced by adding ground tire rubber to HMA using the wet process	Ranged from successful to devastating failures depending on percent of crumb rubber in mix

Source: Loria-Salazar 2008

Among the various reflective cracking mitigation techniques documented in the literature, the following are the primary techniques used in Iowa: rubblization, crack and seat, CIR, FDR, crack relief or stress/strain relieving interlayer (e.g., Strata), and others (engineering fabrics, saw-and-seal, polymer-modified mixes, etc.). Apart from these techniques, milling and filling HMA overlay, sawing and sealing the joints in HMA overlays have also been employed on some projects. And, experimental studies of fabric applications in Iowa have not been conclusive. A brief summary of each of these techniques is provided below.

The following are some of the major research studies carried out in Iowa to study the effectiveness of different reflective cracking strategies:

- Cold In-Place Recycling
  - HR-1020: Transverse Cracking Study of Asphalt Pavement (1981)
  - HR-303: Field Evaluation of Cold In-Place Recycling of Asphalt Concrete (1993)
  - HR-392: Review of Cold In-Place Recycled AC Projects (1998)
  - TR-502: Evaluation of Long-Term Field Performance of CIPR Roads (2007)

- Paving Fabrics and Geosynthetics
  - HR-158: Prevention of Ref. Crack. in H Overlays with Structufors, Petromat, and Cerex (1963)
  - MLR-83: Performance of Reinforcement Fabric Used Under AC Overlays (1983)
  - HR-535: Glasgrid Fabric to Control Reflective Cracking (1990)
  - HR-360: Field Evaluation of Eng. Fabrics for AC Resurfacing – Audubon County (2001)

- Rubblization and Crack and Seat
  - HR-158: Prevention of Ref. Crack. in AC Overlays with Structufors, Petromat, and Cerex (1963)
  - HR-279: Cracking and Seating to Retard Reflective Cracking – Fremont County (1993)
  - HR-527: Crack and Seat PCC Pavement Prior to Resurfacing US 59 – Shelby County (1993)
  - HR-315: Iowa Development of Rubblized Concrete Pavement Base – Mills County (1995)
  - TR-473: Rehabilitation of PCC Pavements Utilizing Rubblization and Crack and Seat (2005)
  - TR-550: Performance Evaluation of Rubblized Pavements in Iowa (2008)

Crack and Seat

Crack and seat is a fractured slab technique that uses a drop hammer to break the existing concrete pavement slabs into smaller pieces (typically 12–48 in.) thereby reducing the effective slab length and minimizing its movement from thermal stresses. This strategy is gaining popularity in Iowa since its original use in 1986 on jointed plain concrete pavement (JPCP) from county roads to Interstate highways.

Four major steps are involved in implementing crack and seat techniques (see Figure 3): cracking the concrete slab (using a drop hammer or guillotine or modified pile driver or whip hammer), seating the cracked slab, applying special treatments, and placing the HMA overlay.

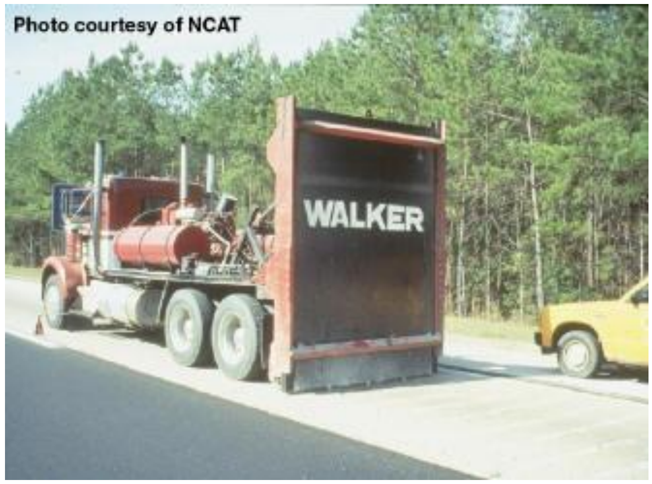


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Figure 3. Crack and seat

The cracking of the existing pavement reduces the slab movement due to thermal action, thus minimizing or controlling the reflective cracking in the HMA overlay. The resulting pieces should be large enough to retain aggregate interlock between aggregates, and yet small enough to minimize the unreinforced PCC slab joint movement (PCS/Law 1991).

It has been reported that crack and seat fractured slab technique, when used properly, has the potential to significantly delay the reflective cracking, but not completely eliminate them in the HMA overlay (Thompson 1999). They have also been reported to be effective in eliminating blowups in JPCPs (Drake 1988). Although smaller cracked PCC pieces mean larger potential reduction in reflective cracking, they also lead to larger reduction in the concrete pavement structural strength (Eckrose and Poston 1982).

A previous study conducted in Iowa (IHRB Project TR-473) identified crack and seat as a viable strategy for Iowa pavements that minimizes reflective cracking (Ceylan et al. 2005). Still, several challenges exist in the design and construction phases of a project when selecting this strategy. Sharpe et al. (1987) identified the following main concerns of the Kentucky Department of Highways when implementing this strategy:

- Selecting acceptable breaking equipment
- Validating the extent of breaking or cracking
- Determining acceptable seating/rolling patterns
- Establishing minimum asphalt overlay thicknesses

The breaking equipment used and the cracking pattern choose has an effect on the structural capacity of the pavement. With the use of crack and seat technique, the structural capacity of the pavement is generally reduced. Since the structural capacity affects the thickness of the HMA layer, proper construction criterion is necessary to achieve the intended design.

Rubblization

Rubblization is defined as "breaking the existing pavement into pieces and overlaying with HMA." It destroys the slab action of the rigid pavements. The sizes of the broken pieces usually range from sand size to 3 in. at the surface and from 12 to 15 in. on the bottom part of the rubblized layer (Von Quintus et al. 2007). The results from a comprehensive investigation conducted by Pavement Consultancy Services (PCS) (PCS/Law 1991), the National Asphalt Pavement Association (NAPA) study (NAPA 1994), and a nationwide survey conducted by the Florida DOT (Ksaibati et al. 1999) all indicate that rubblization is the most effective procedure for addressing reflection cracking. It has been concluded that the rubblized PCC behaves like "a high-strength granular base," with strength between 1.5 to 3 times greater than a high-quality, dense-graded, crushed-stone base in load-distributing characteristics (PCS/Law 1991).

In general, two types of equipment are used in the rubblization process (see Figure 4): resonant pavement breaker (RPB) and multiple-head breaker (MHB).

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Figure 4. Rubblization: Resonant pavement breaker (left) and multiple-head breaker (right)

The RPB uses vibrating hammers to break the concrete slab and destroy the bond between the concrete and the steel. The other common rubblizing equipment is the self-contained and self-propelled MHB used by Antigo Construction, which is capable of rubblizing the pavement over a minimum width of 13 ft per pass.

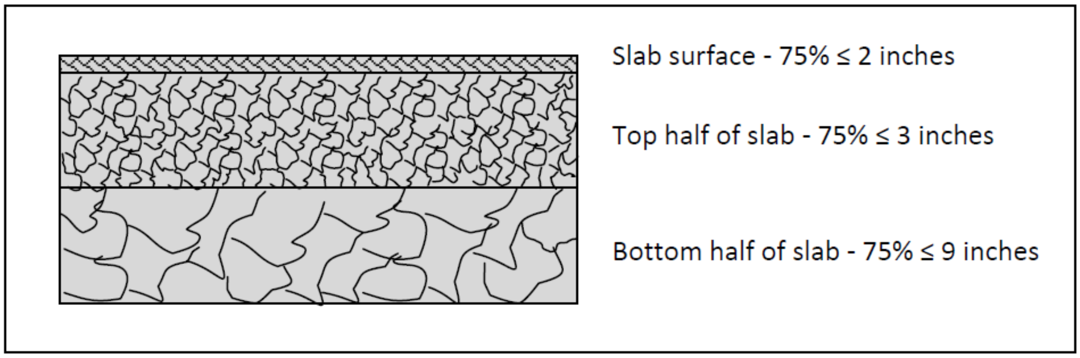
During rubblization the PCC is converted to small, interconnected pieces that serve as an aggregate base course. IHRB Project TR-473 concluded that rubblization can be a viable, rapid, and cost-effective rehabilitation method for deteriorated PCC pavements. Several state highway agencies (Illinois, Michigan, Wisconsin, etc.) have also completed studies on the performance of rubblized pavements and have concluded similar results (Von Quintus et al. 2009). To address the various construction challenges when implementing this strategy, in February 2004 the Federal Aviation Administration (FAA) adopted and published FAA Engineering Brief (EB) No. 66, *Rubblized Portland Cement Concrete Base Course*. The document includes guidance and criteria for rubblizing PCC pavements.

The Iowa DOT recognized the potential of rubblization in rehabilitating old concrete pavements and conducted a research project to rehabilitate and evaluate a severely deteriorated concrete roadway using a rubblization process as early as 1995. A 3.0 km (1.9 mi.) section of L-63 in Mills county was selected and divided into 16 sections. In 1985, HMA overlay construction was done in 13 sections after rubblizing the existing pavement with a RPB and in three sections without rubblization. This research concluded that the rubblization process prevents reflective cracking and that edge drains improved the structural rating of the rubblized roadway. In addition, it was noted that a 5 in. (125 mm) thick HMA overlay on a rubblized base provided an excellent roadway regardless of soil and drainage conditions; whereas a 3 in. (75 mm) thick HMA overlay on a rubblized base can provide a good roadway if the soil structure below the rubblized base is stable and well drained.

After the completion of this research (Tymkowicz and DeVrie 1995), the use of rubblization has steadily increased for Iowa state highways and county roadways. However, there were some changes in the rubblization practices adopted in Iowa due to poor subgrade, lack of crushed aggregate base, and the use of thin concrete pavements (Jansen 2006). The modified rubblization method was proposed and adapted in the rehabilitation project of W-14 in Winneshiek County by Antigo in 2003.

Ceylan et al. (2008) recently evaluated the performance of rubblized pavements in Iowa using field surveys (falling weight deflectometer, visual distress surveys, DCP, and coring) and concluded that Iowa’s rubblized pavement sections are performing well. The predominant distresses exhibited on HMA-overlaid rubblized PCC sections are non-load associated distresses, such as low-temperature cracking and/or longitudinal cracking. Similarly, based on long-term field monitoring results of different mitigation strategies applied to Iowa pavements, Kim et al. (2008) reported that the rubblization technique was the most effective method in retarding reflection cracking whereas the test sections with a crack relief layer exhibited the highest amount of reflection cracking. However, it is important to note that the rubblized sections had much thicker HMA overlay than the other test sections. Several state highway agencies (Illinois, Michigan, Wisconsin, etc.) have also completed studies on the performance of rubblized pavements and have reported success with the use of this technique (Von Quintus et al. 2009).

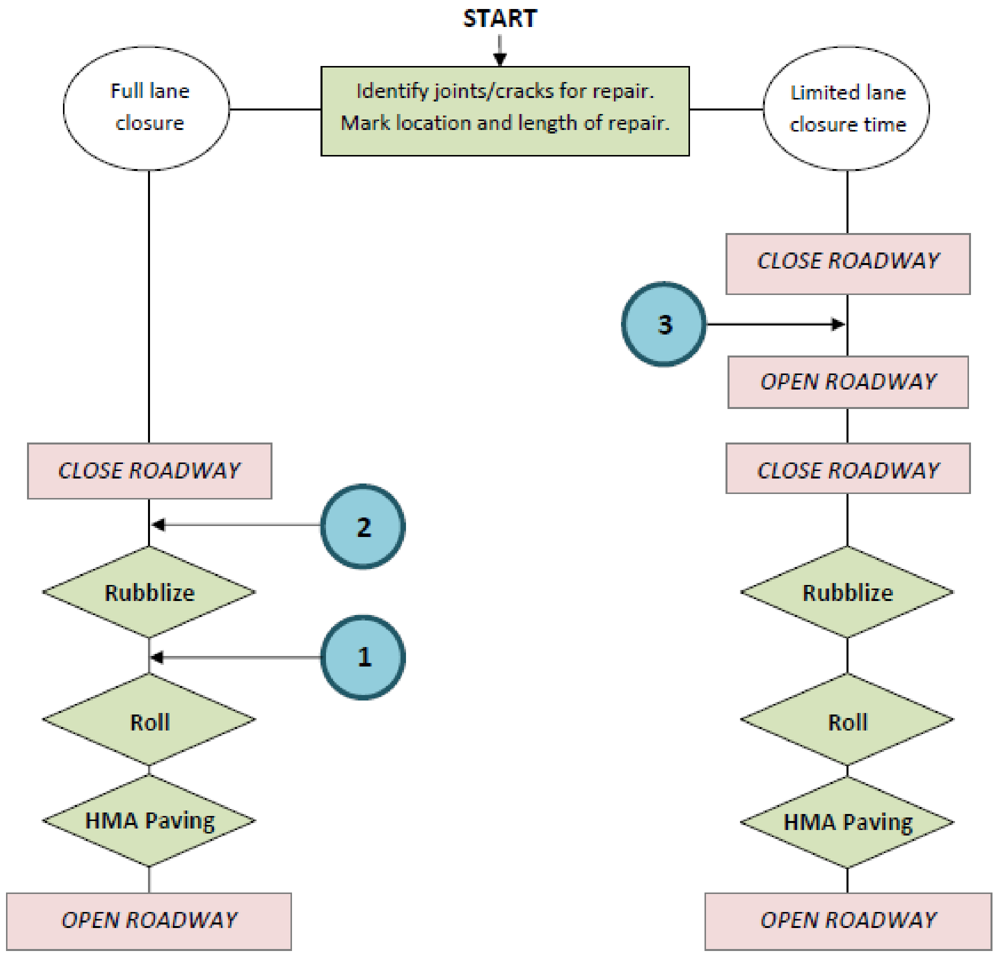
To address the various construction challenges when implementing this strategy, in February 2004 the FAA adopted and published FAA Engineering Brief (EB) No. 66, *Rubblized Portland Cement Concrete Base Course*. The document includes guidance and criterion for rubblizing PCC pavements. Similarly, the Wisconsin Department of Transportation (WisDOT) Standard Specifications give guidance to the contractors with respect to size requirements for rubblized pieces in slab surface, top half of slab, and bottom half of slab as shown in Figure 5.



WisDOT

Figure 5. Rubblized particle size requirements as per WisDOT standard specifications

Recently, Battaglia and Paye (2011) investigated premature distress formation in Wisconsin rubblized pavements by analyzing design parameters, soil properties, historic distress levels, and several additional factors for 19 good- and poor-performing pavements. It was recommended that major cracks and distressed joints in the existing PCC pavement be repaired before rubblizing/HMA overlay to prevent reflection cracking. According to Battaglia and Paye (2011), joints with heavy deterioration, spalling, and/or evidence of pumping following the pavement condition index (PCI) rating system guidelines are candidates for repair. Recommended PCC joint repair and test rolling guidelines were also proposed by Battaglia and Paye (2011), as shown in Figure 6.



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.

Battaglia and Paye 2011

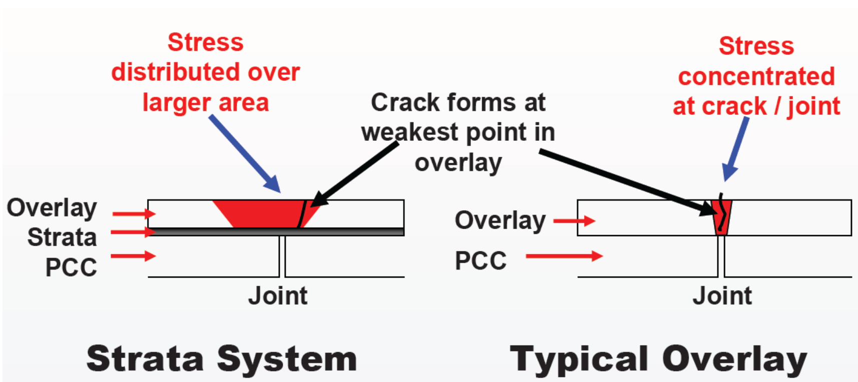
Figure 6. PCC joint and crack repair options and construction sequence for rubblization projects

Reflective Crack Relief Interlayer

A reflective cracking relief interlayer is a low stiffness pavement layer that relieves the stresses and strains built up in an underlying pavement layers by dissipating energy during vertical and horizontal deformations. Typically these layers are less than two inches and do not increase the structural value of the pavement, but they are designed to reduce reflective cracking. Various interlayer techniques have been developed and successfully used under the right application.

These include a stress absorption membrane interlayer, a rubber modified asphalt interlayer, a soft asphalt interlayer, geosynthetics (paving fabrics), and Strata.

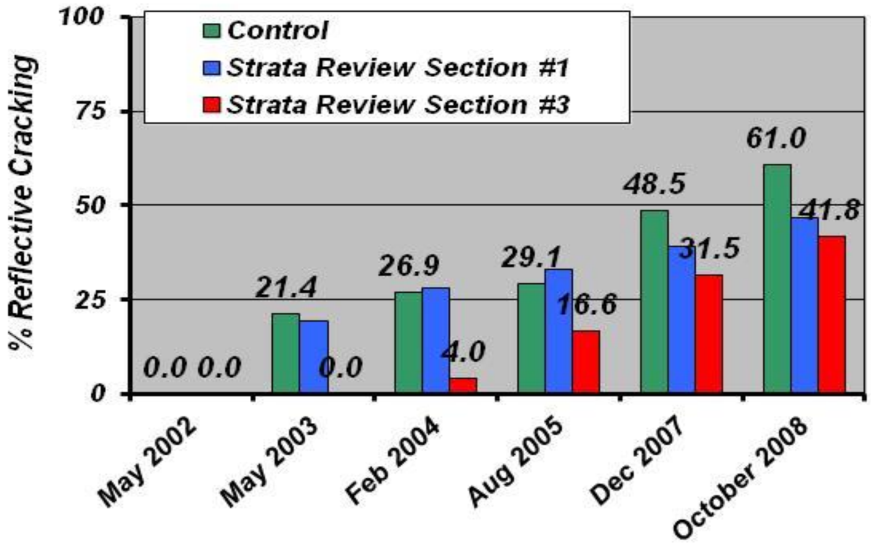
Strata is a reflective crack relief system promoted by SEM Materials, Inc. (now Road Science LLC, a division of ArrMaz Custom Chemicals) that protects the existing pavement structure from water damage and delays reflective cracks. According to Road Science LLC, the Strata system has several advantages: it significantly delays reflective cracking longer than paving fabrics and HMA overlays; it provides an impermeable interlayer to protect pavement structure from moisture damage; it provides a highly fatigue resistant material; it uses readily available aggregates and it lengthens pavement service life; it provides ease of mixing, placement, and compaction through the use of conventional HMA paving equipment and standard construction methods; and it provides savings in construction time and facilitating easy maintenance of pavement (Von Quintus et al. 2009). See Figure 7.



Von Quintus et al. 2009

Figure 7. Strata system

The Strata system was applied on an Iowa highway project in northeast Iowa on IA 9 near Decorah (Winnesheik County) in 2001 and was studied by Wagoner et al. (2006) using field observations, laboratory testing, and finite element analysis. The IA 9 project consisted of three sections (a control section and sections 1 and 3 with a nominal overlay thickness of approximately 6.3 in.) in a two-lane pavement with an average of 3,800 vehicles per day and 18 percent truck traffic. The Strata system was placed above the leveling course in sections 1 and 3 and annual surveys were conducted to monitor the development of reflective cracks. The study concluded that the Strata layer was beneficial in retarding reflective cracking. Figure 8 illustrates the reflective cracking performance of the Strata sections 1 and 3 as well as the control section.



Wagoner et al. 2006

Figure 8. Reflective cracking performance of Strata sections 1 and 3 and the control section

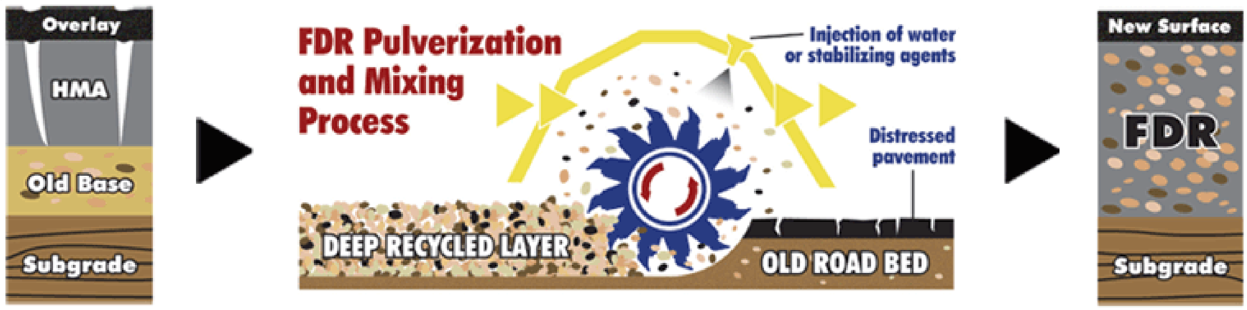
Cold In-Place Recycling/Full-Depth Reclamation

FDR and CIR are viable strategies to remove cracks in HMA pavements. CIR involves cold milling the existing HMA surface; mixing the cold milled materials with emulsified asphalt or other modifiers to improve the properties of original HMA mix; and screeding, spreading, and compacting the recycled mixture in one continuous operation (see Figure 9).

FHWA

Figure 9. Cold in-place recycling

NCHRP Synthesis 421: Recycling and Reclamation of Asphalt Pavements Using In-Place Methods defines FDR as a process that pulverizes an existing asphalt pavement along with one or more inches of the underlying base or subgrade; the pulverized material is mixed with or without additional binders, additives, or water, and then placed, graded, and compacted to provide an improved base layer for placement of surface layers (see Figure 10).



American Road Reclaimers

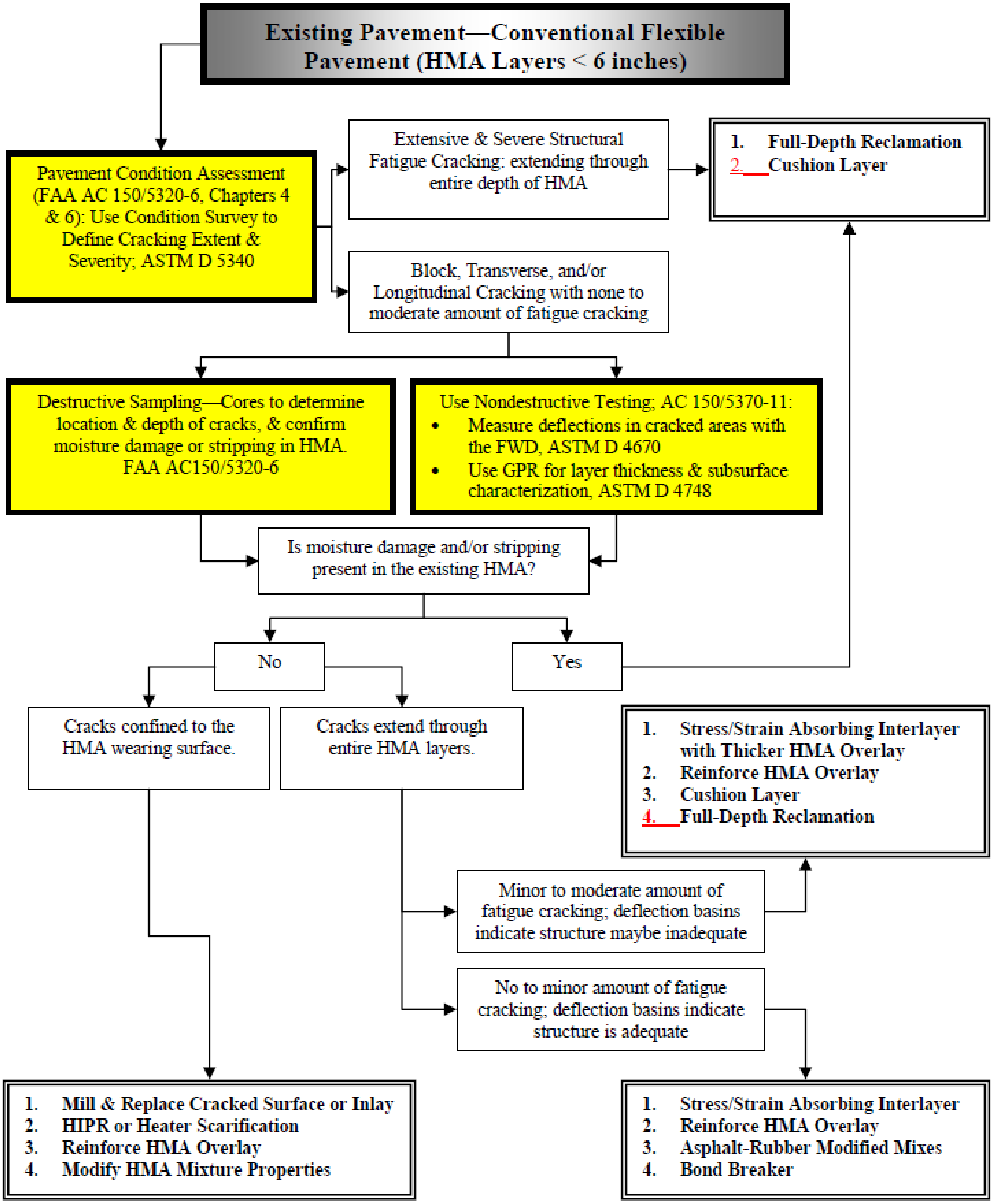
Figure 10. Full-depth reclamation

FDR works well when the pavement layer has a minimal total thickness (i.e., six inches) while CIR works well when only the top three to four inches need to be repaired. Although FDR has potential cost-saving, engineering, as well as other sustainability benefits and is considered a viable rehabilitation alternative, information reported in the literature is scanty with respect to the material properties of FDR to facilitate the structural design of pavements incorporating FDR stabilized base materials. In fact, there is some controversy on how to characterize the FDR layer stabilized with asphalt emulsions (Thompson et al. 2009).

Schram (2011) recently reported on Iowa's experience with CIR and FDR techniques. Over a five-year total, there have been 53 CIR projects (foam and emulsion) in Iowa costing $118 million and totaling 1,800 lane-miles. On the other hand, FDR (using fly ash stabilization) over a five-year total amounts to only three projects costing $8.6 million and totaling 100 lane-miles. The IHRB Project TR-502, *Evaluation of Long-Term Field Performance of Cold In-Place Recycled Roads: Field and Laboratory Testing*, studied the performance of CIR in Iowa projects extensively. The study concluded that a CIR layer effectively acts as a stress relieving layer to mitigate reflective cracking.

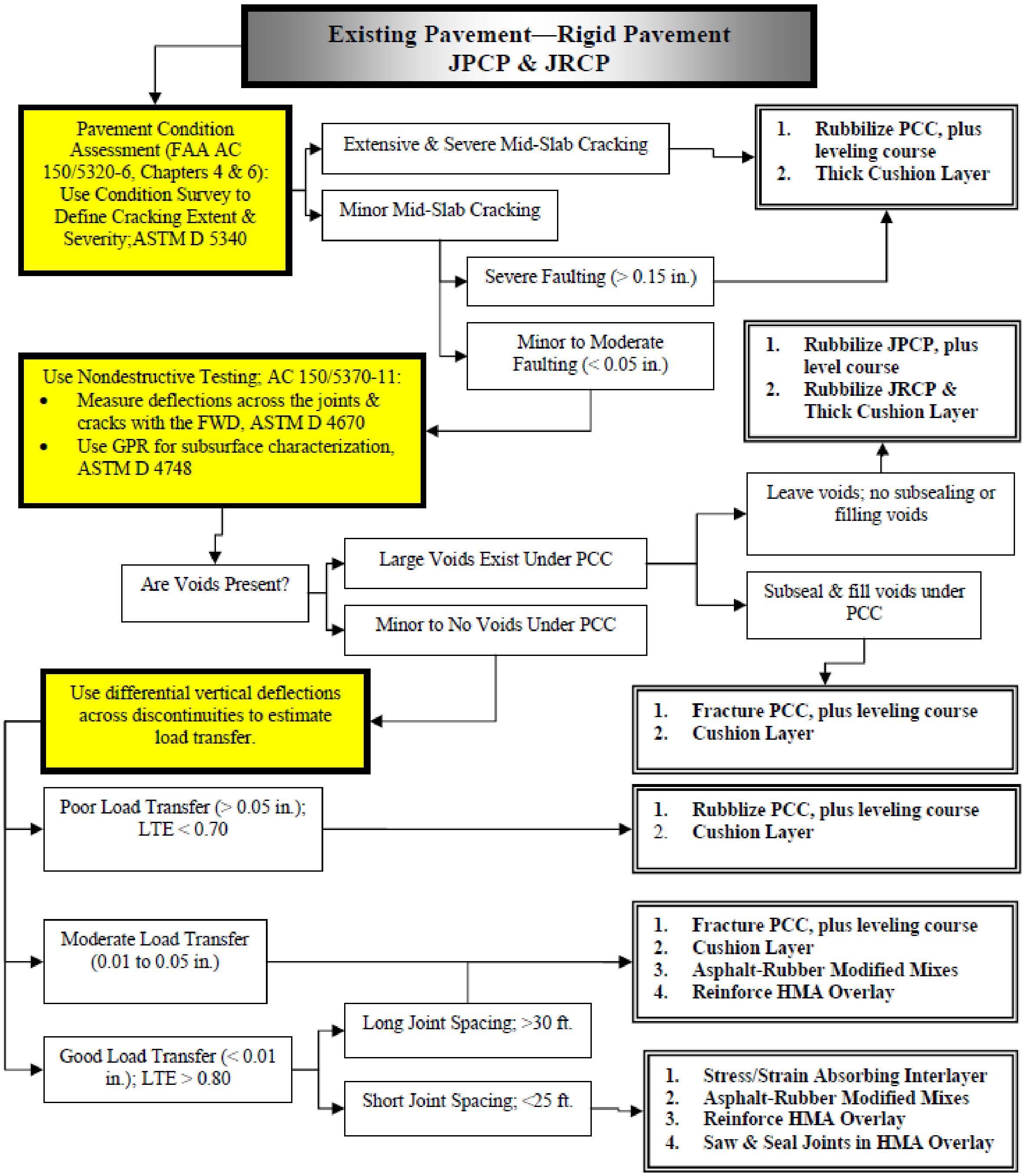
Although all these techniques have been successfully used with recommendations for further investigation and expanded use in Iowa, they still continue to be used modestly due to lack of proper technical guidance. While limited performance data is available for many of the existing and newer methods and products (including the proprietary ones), the performance data available for other reflective cracking mitigation techniques have not been examined or documented from the perspective of providing technical guidance on the appropriate use of various pre-overlay techniques for different situations. This report details additional CIR performance data to summarize the current performance of CIR in Iowa in Chapter 6. Additional technical guidance on CIR techniques is needed to provide practical guidance to owners, industry, and practitioners regarding proper project selection, design, and quality control of reflective crack mitigation techniques forms the basis of this proposed research.

Von Quintus et al. (2009) reviewed products and processes that have been used to mitigate reflective cracks in rigid and flexible airport pavements. Decision tress providing guidance to select the appropriate mitigation treatment method for the site and in place pavement condition was developed (see Figure 11 and Figure 12). Similar decision trees would be greatly beneficial to Iowa design engineers when selecting a reflective cracking mitigation strategy for a particular project.



Von Quintus et al. 2009

Figure 11. Decision tree for providing guidance reflective cracking mitigation in HMA overlays of existing conventional flexible airport pavements



Von Quintus et al. 2009

Figure 12. Decision tree for providing guidance reflective cracking mitigation in HMA overlays of existing conventional rigid airport pavements

CHAPTER 3 NETWORK-LEVEL REFLECTIVE CRACKING MITIGATION STRATEGIES

3.1 Chapter Objective

The main objective of this chapter is to identify the most appropriate pavement mitigation strategy by using the Iowa PMIS. This involved collecting pavement structure, traffic, and field performance data in Iowa's composite pavements. Four widely used rehabilitation strategies for composite pavements were chosen for evaluation from the PMIS database. These include HMA overlay, HMA mill and fill, SCR, and PCC rubblization. Reliability/Survivial analysis was applied for the data analyis to compare the survival time of the four treatment methods and to evaluate the influence factors for the reflective cracking development using JMP (SAS Institute 2012).

3.2 Background

Four widely used rehabilitation strategies for composite pavements evaluated in this chapter are as follows:

- HMA overlay
- HMA mill and fill
- Heater scarification
- PCC rubblization.

The HMA overlays are simply the process of installing a new layer of HMA directly over an existing pavement structure. They generally provide good performance over flexible pavements, but their performance for composite pavements may depend on the extent of reflective cracking. Surface recycling has been reported by the FHWA to be successful in removing pre-existing reflective cracks prior to an HMA overlay (FHWA 2002). Mill and fill and SCR are generally used in Iowa as two common ways to remove cracks from old HMA overlays. In the SCR method, the pulverized pavement materials are used along with recycling agents in the re-paving process, while in the mill and fill process, the contractors typically use new asphalt concrete mix for repaving after milling. Therefore, the SCR treatment can be considered to result in "reclaimed asphalt pavement (RAP)." Rubblization is defined as "breaking the existing concrete pavement into smaller fragments and overlaying it with HMA." The extent of rubblization depends on the thickness and size of the broken concrete slab, and the intent of rubblization is to produce a structurally sound base which prevents reflective cracking by eliminating the existing pavement distresses and joints.

A suitable data source to monitor the pavement performance and reflective cracking conditions following the four pavement rehabilitation strategies are contained in state transportation agencies' PMIS. In Iowa, this information is contained in the Iowa PMIS database and the IPMP and is collected non-destructively via two sets of laser measurements and photologging for later conversion to the pavement condition index. The Iowa PMIS database contains data about pavement condition, construction history, and materials from 1991 until the present for all of the state-maintained roads (Interstate, national, and state highways). The IPMP database is a pavement condition information database for paved roads on the local system (counties and cities) in Iowa. Both databases include continuous testing and subsequent quantification that provides 100 percent coverage length of the network and roadway surface, as opposed to a smaller sample of representative sections. The surface distress information in both databases is based on the same technology and are collected in the same manner utilizing the same contractor. Therefore, information in the two databases is comparable with each other and they follow the same method for pavement performance surveys, as defined in the "Distress Identification Manual for the Long-Term Pavement Performance (LTPP) Project" (Smadi and Maze 1998). The literature has shown that reflective cracking can be rated in the same manner as transverse cracking for composite pavements (Lytton et al. 2010, Zhou et al. 2010). In this study, only transverse cracks are considered as reflective cracks for each test section in the PMIS and IPMP databases.

The performance data are collected on a two-year cycle in the state. The surface distresses, international roughness index (IRI), rutting, and faulting data are collected using a mobile device equipped with sensors, cameras, GPS unit (used to determine location), and a position and orientation system that determines roll, pitch, heading and velocity to capture the roadway geometry. The Iowa DOT has a contract with an independent contractor to collect the required information for their pavement management system. IRI is collected in each wheel path utilizing two laser sensors (South Dakota Profiler-SDP: Class I profiling device according to ASTM E950) behind the two front wheels. These two sensors measure the longitudinal profile of the road to determine IRI. The same laser sensors are used to determine the faulting between slabs in concrete pavements too. In the back of the mobile device, two scanning lasers are used to measure the transverse profile of the pavement surface (14 ft wide) to determine rutting for asphalt pavements. Because of the wide foot print of the two lasers, the edge drop off can also be determined. Surface distresses such as cracking and patching are collected using a 2D camera that captures images of the pavement surface and are later analyzed using image analysis and pattern recognition to determine the type of cracking and severity. Once all of the surface distresses are collected, the Iowa DOT calculates a PCI for each homogenous pavement management section. The sections can range between 0.5 to over 5 miles in length based on the original construction and rehabilitation history. The PCI calculation is based on pavement type (concrete, asphalt, and composite) and system (Interstate and other).

In order to track the growth rate of reflective cracking and composite pavement performance over time for each type of rehabilitation method, survival analysis, or more generally, time-to-event analysis is used. The term survival analysis s(t) is used predominately in biomedical and healthcare sciences where the interest is in observing the time to death of either patients or of laboratory animals. The engineering sciences have also contributed to the development of survival analysis, wherein it is referred to as "reliability analysis" or "failure time analysis." Early survival analysis application relies more on empirical methods than statistical procedures. The survival analysis approach simply considers the cumulative traffic as a surrogate for pavement life (Vepa et al. 1996). In recent years, more complicated survival analysis applications were conducted using comprehensive pavement databases and advanced statistical software (e.g., JMP, SAS, Minitab). Bausano et al. (2004) compared the reliability of four different types of HMA pavement maintenance treatments using the Michigan PMIS database. Dong and Huang (2012) employed the survival function to evaluate four types of HMA pavement cracks using the LTPP database. Survival analysis focusing on the hazard function was applied by Yang (2009) to estimate the duration of pavement life in Florida. Survival data are generally described and modeled in terms of two related functions, namely the survival function s(t), and hazard function h(t), which are inter-related (see Equation 1). If either s(t) or h(t) is known, the other can be determined. Consequently, either can be the basis of statistical analysis (Hosmer and Lemeshow 1998). The survival function s(t) measures the survival probability beyond a time t, while h(t) measures the failure probability occurring in the next instant, given survival to time t.

$$h(t) = -\frac{d}{dt}[\log s(t)] \qquad (1)$$

In this report, three pavement performance indicators are applied, including reflective cracking, IRI, and PCI, with the emphasis on reflective cracking. From the perspective of statistics, the specific difference related to survival analysis arises largely from the fact that survival data should be divided into censored and uncensored groups. Censoring occurs when an observation is incomplete due to some random cause. In the area of pavement performance, censored data occurs if a pavement project performs well during the observation time and reaches the planned end of study, or is lost to follow up, while uncensored data (failure) is obtained when a pavement project is distressed beyond the performance indicators' threshold values during the observation period.

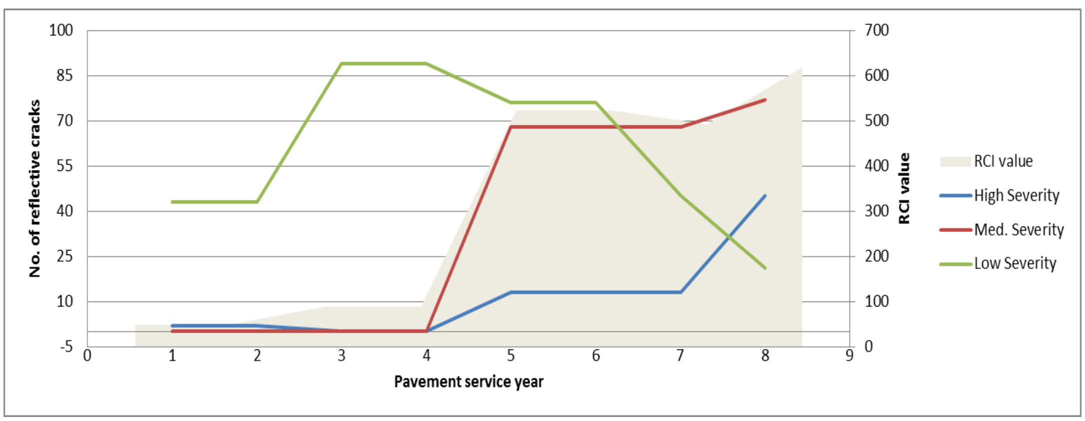
3.3 Threshold Value

Threshold values are used to delineate the censored and uncensored data. The threshold values are defined as the lowest acceptable pavement condition level before pavement preservation treatments become necessary. A lower threshold value is used for local county roads, as they usually have much lower traffic and longer service lives. Although there do not appear to be universal threshold values for the pavement maintenance or rehabilitation treatments, the IRI and PCI values shown in Table 2 are generally used for pavements in fair or poor condition (Papagiannakis et al. 2009). The range and description for each performance index are also provided. To quantify the severity and extent of reflective cracking, a simple reflective cracking index (RCI) formula is developed, as shown in Table 2.

Table 2. Summary of three performance indicators

Pavement Condition Index	Range	Trigger	Description
Reflective crack index	0 to inf.	420 (primary road) 390 (county road)	RCI=$Low \times 1 + Med \times 3 + High \times 6$; Low, Med., High: represent numbers of low, medium and high severity reflective cracks per km.
International Roughness Index	(0 to inf.) in./mi	125 in./mi (primary road) 120 in./mi (county road)	Irregularities in pavement surface. Higher values indicate a rougher road. Measured in m/km and converted to in/mi. in this study.
Pavement Condition Index	0 to 100	64 (primary road) 68 (county road)	Composite index including cracking, ride quality & rutting. Lower values indicate poorer road conditions.

The index is based upon the extent of reflective cracking and a weighting function of the crack severity to account for the condition of reflective cracking. Taking three levels of crack severity into consideration, the RCI provides a distress condition rather than merely evaluating only one facet of the cracking, such as the total crack length or amount of cracks per kilometer or mile. In Figure 13, a typical ascending trend for RCI can be observed.



IA 12 highway project, STP-12-(16)-2C-97

Figure 13. A typical relationship for reflective cracking and RCI

The RCI value is represented by the shaded area whose height is measured on the right axis. On the left axis, reflective crack numbers in the low severity level develop quickly at the beginning, and start to decrease later as more cracks move into medium and high severity levels in later service life. In other words, the RCI can represent not only changes in the total number of cracks, but also show the influence and dimensions of their severity. The threshold value for RCI is set to 420 by considering common concrete joint spacing (4.5 to 6.1 m) and the possible number of reflective cracks per kilometer. Based upon this threshold value, at least 420 low severity, 140 medium severity, or 70 high severity cracks are allowed per kilometer before triggering the threshold. This threshold is similar to those recommended by other highway agencies for reflective or transverse cracking. The threshold value used in the pavement health track analysis tool is 1,500 ft/mi. for primary and secondary roads, and Wisconsin calls for remedial action if more than 25 cracks per 100 meter section are found (Titus-Glover et al. 2010, Scott et al. 2011).

3.4 Data Preparation

This study utilizes pavement performance, traffic, and pavement structural data from the Iowa PMIS and IPMP databases and represents pavements constructed mainly from 1998 through 2008. The performance of these projects was tracked until the latest 2012 pavement performance survey representing 154 projects. These include 42 projects for mill and fill treatment, 31 projects for heater scarification, 51 HMA overlay projects, and 30 rubblization projects. Detailed pavement data extracted from the PMIS and IPMP database are presented in Appendix A. The life distribution and survival platform is used for the data analysis via JMP software (SAS 2012).

3.5 Discussion of Results

Kaplan-Meier Estimator

In statistical analyses, it is prudent to perform a univariate analysis before proceeding to more complicated models. In survival analysis, it is highly recommended to look at the Kaplan-Meier curves for all the categorical predictors. This will provide insight into the shape of the survival function for each group and provide an idea of whether or not the groups are proportional. The Kaplan-Meier estimator is a nonparametric maximum likelihood estimator of the survival function. It incorporates information from all of the observations available, both uncensored and censored, by considering the survival function at any point in time as a series of steps defined by the observed and censored times (Hosmer and Lemeshow 1998). Figure 14 compares the Kaplan-Meier estimate for the four different rehabilitation methods on reflective cracking.

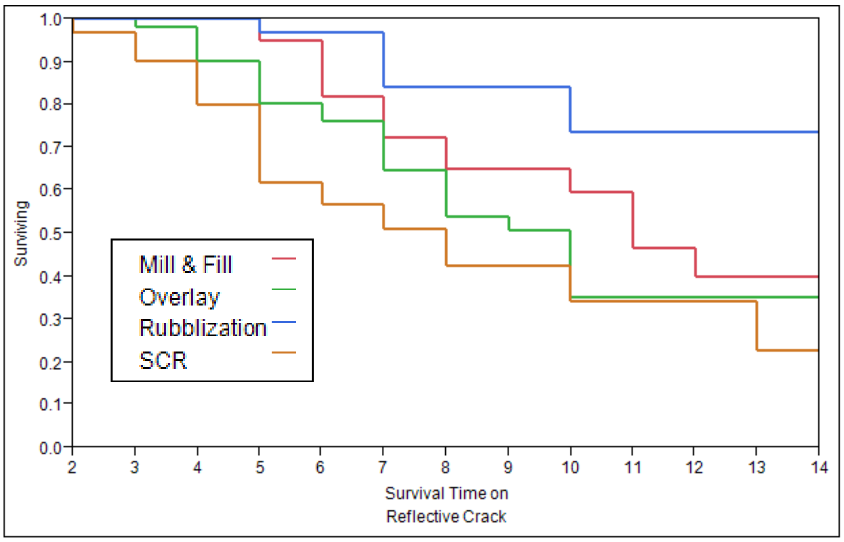


Figure 14. Kaplan-Meier estimator curves for reflective cracking treatments

The largest time length shown is 14 years, which is the maximum survival time from 1998 to 2012. As expected, the survival function decreases as the pavement age increases. The survival function for the rubblization treatment lies completely above the other three treatments and it has a long right-tail with relatively constant survival probability. The survival function for the HMA overlay is quite close to the mill and fill treatment in early service life and it gradually drops down and touches the curve for the SCR treatment, suggesting that the HMA overlay has an unfavorable survival experience in later service life with respect to reflective cracking. The estimated survivorship function for the SCR treatment lies completely below that of the other three treatments, giving it the poorest reflective cracking performance. A typical pattern for both the SCR and HMA overlay treatments is relatively early rapid descending survivor function with a gradually longer tail in the later service life. This is the result of a number of early failures and a few projects with survival near the maximum follow-up time. Table 3 summarizes the median survival time, as well as other percentiles, which are determined by linear interpolation.

Table 3. Percentile summaries and tests between groups for reflective cracking

Group	Number failed	Number censored	70 Percentile (yrs)	Median (yrs)	30 Percentile (yrs)
Mill and fill	16	26	7.6	10.7	N/A
SCR	17	14	4.5	6.8	11.5
Overlay	26	25	6.5	9.0	N/A
Rubblization	5	25	N/A	N/A	N/A
Combined	64	90	6.5	9.5	N/A

Test	Chi Square	DF	Prob>Chi Sq
Log-Rank	16.3	3	0.0010*
Wilcoxon	19.5	3	0.0002*

The median value, or 50th survival percentile, is considered to be the service life that a pavement can sustain before failure (Gharaibeh and Darter 2003). The test statistics are further examined to determine whether or not the four types of treatments are significantly different in their survival functions for reflective cracking. Log-rank and Wilcoxon tests are two simple comparison methods used in JMP software. In general, the Log-rank test places more emphasis on the differences in the curves at later survival time values, while the Wilcoxon test places more weight on early survival time values. The results show that the rubblization treatment can significantly reduce the occurrence of reflective cracking compared to the other three treatment methods, which is the cause of the high probabilities of test separation in the Log-Rank and Wilcoxon test analyses for reflective cracking.

Figure 15 illustrates the relationship between survival function and pavement service life based on IRI and PCI.

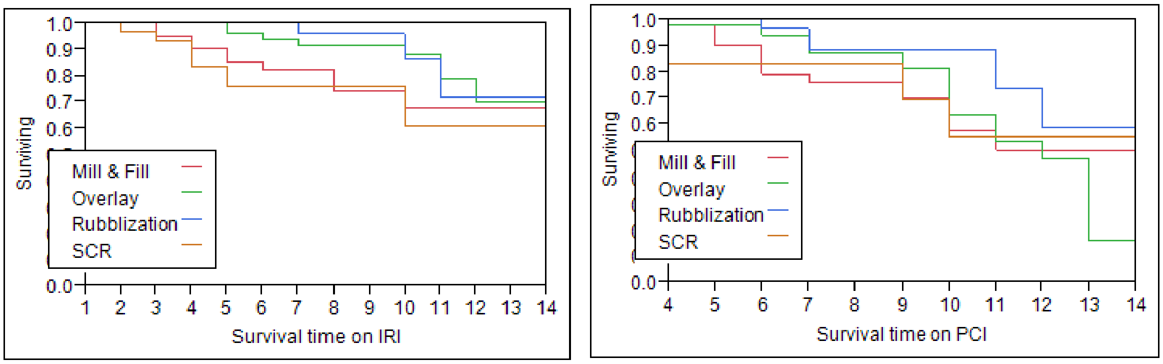


Figure 15. Kaplan-Meier estimator curves for IRI and PCI

The survival function for IRI falls within a relatively narrow band for each treatment method. All four different rehabilitation methods are effective in preserving the smoothness of composite pavements within 14 years of service life before dropping to 50 percent survival probability. Lower survival functions for IRI are observed for the mill and fill and SCR treatments compared to the HMA overlay and rubblization treatments. Table 4 also indicates a significant difference in IRI performance for the four treatments, especially in early survival time as indicated by the Wilcoxon test.

Table 4. Tests between groups for IRI and PCI

Test between groups for IRI		Test between groups for PCI	
Test	Prob>Chi Sq	Test	Prob> Chi Sq
Log-Rank	0.0252*	Log-Rank	0.391
Wilcoxon	0.0034*	Wilcoxon	0.184

This result is counter to previous studies which concluded that milling the existing HMA surface prior to overlay is effective in keeping the overlay smoother (Wiser 2011). This discrepancy could be due to differences in the initial IRI conditions of pavements at the time of treatment applications. Unlike pavement distress data which typically indicates an absence of cracks soon after rehabilitation, the roughness-based initial IRI values usually vary greatly from 50 to 90 in./mi. Use of RAP in the mill and fill and SCR treatments may also be a cause for the higher initial IRI values. Table 4 shows that there is no significant statistical difference among the survival curves for PCI.

As PCI is a composite index which gives a more comprehensive indicator of pavement condition, roads treated with only HMA overlay treatment are observed to have the poorest PCI conditions in later service life (Figure 15).

Model Fitting

The Kaplan-Meier estimator is used for describing the survival experience of a population, and does not require any specific distributional assumptions about the shape of the survival function.

The parametric model for survival analysis is considered next, as it may provide more information on the relationship between variables and the survival function. A best-fit model can also provide higher accuracy for predicting the survival of a given subject. Several parametric models are commonly used, including the Exponential, Weibull, Lognormal, and Logistic models. The most obvious distinguishing feature between the models is in the shape of the hazard function they assume the data to follow. The Weibull distribution model is appropriate when the hazard is always increasing or decreasing. In the Exponential model, the hazard is assumed to be constant over time, while the hazard function of the Logistic model follows an "S-curve" behavior. The Lognormal model is preferable when the hazard rises to a peak before decreasing.

A few diagnostic methods are available for the model fitness comparison, including both numerical and graphical approaches. Ideally, the selected model should reflect the physical pavement cracking and performance development patterns. In this study, Akaike's information criterion (AIC) is applied, as it performs well for both univariate and multivariable survival analyses. AIC as suggested by Akaike (1974) is an estimate of the relative distance between the unknown true-likelihood function of the data and the fitted likelihood function of the model. A lower AIC value means that a model is considered to be closer to the truth. For the general case, the method to estimate the AIC value is shown in Equation 2, where L is the maximum likelihood function, and k is the number of free parameters in the chosen model.

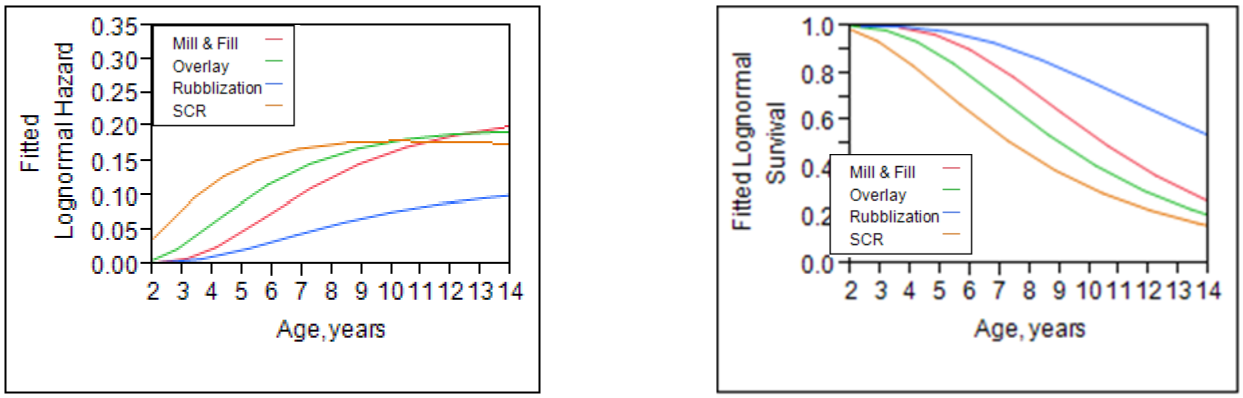
$$\text{Minimize AIC} = 2k - 2\ln(L) \tag{2}$$

For the univariate analysis performed herein, three parameters are considered; pavement service life, intercept, and error. As shown in Table 5, the Lognormal distribution appears to be the best-suited for modeling the general trend of reflective cracking and IRI, while the Weibull model provides the best fit for the PCI.

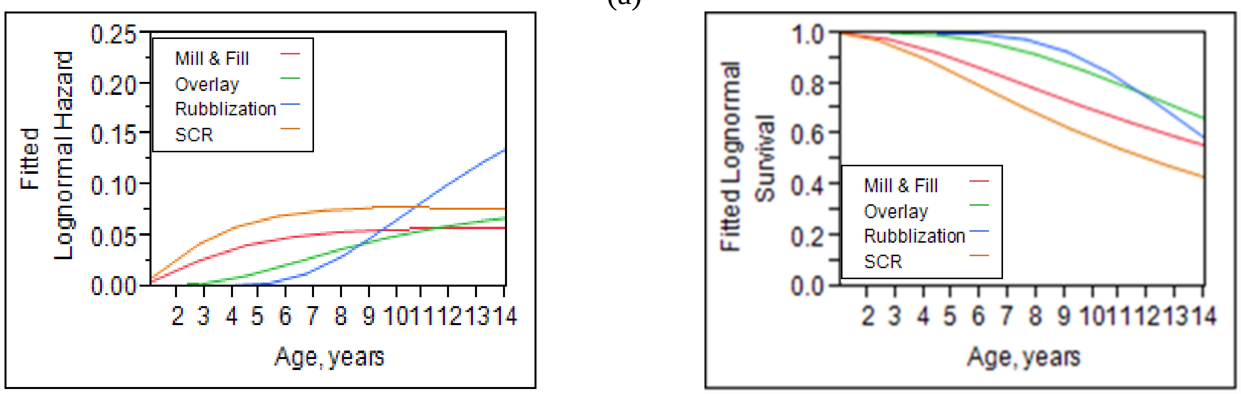
Table 5. Model comparisons by the AIC values

AIC value	Lognormal	Weibull	Logistic	Exponential
Reflective Crack	448.10	450.68	457.07	507.21
PCI	330.69	329.35	330.79	384.39
IRI	284.32	285.10	285.27	300.28

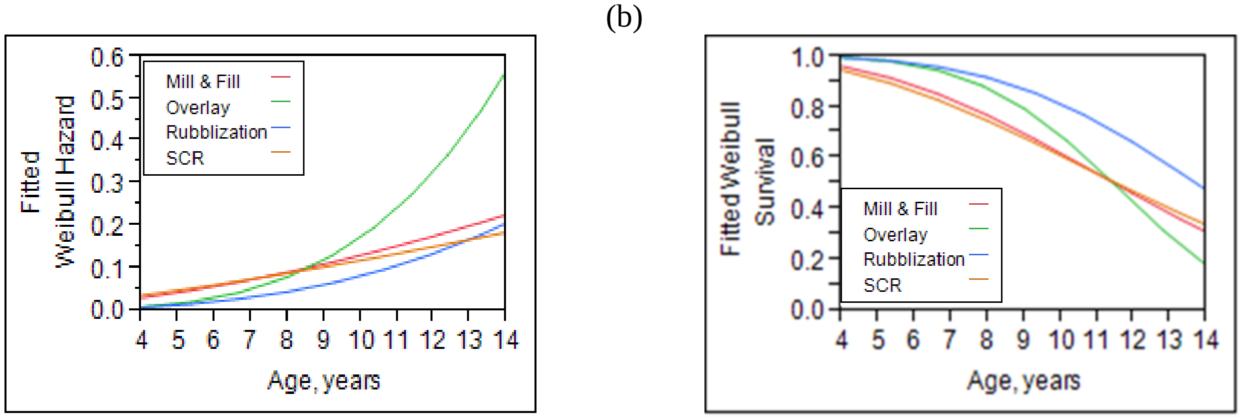
Further, the modeled hazard and survival functions are presented in Figure 16 for the three pavement condition indicators.



(a)



(b)



(c)

Figure 16. Summary of model fitted hazard and survival functions for (a) reflective cracking, (b) IRI, and (c) PCI

The hazard function typically provides clearer information about the underlying mechanism of failure than the survival function. Figure 16 (a) shows that there is early reflective cracking failure risk for the SCR and overlay treatments, followed by a constant hazard in the later stages of pavement life, while mill and fill has an accelerated failure rate in later service life. The hazard rate for rubblization treated pavements, on the other hand, is lowest and gradually increases during a natural failure process. In Figure 16 (b), higher hazard rates for IRI are clearly exhibited in the early life for the SCR and mill and fill treatments. As discussed previously, this could be attributed to the initial IRI condition. To test this hypothesis, the initial IRI values for all of the 155 pavement projects were sorted and displayed in the boxplot of Figure 17.

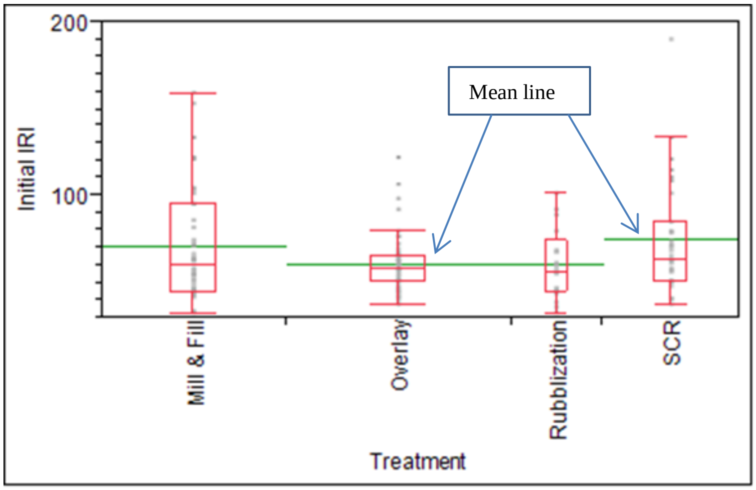


Figure 17. Initial IRI values for the four treatments

As indicated by the mean lines, the average initial IRI values for mill and fill and SCR treatments are slightly higher than the other two methods. Except for a few outliers, most of the roughness-based initial IRI values vary from 45 to 90 in/mile between the lower and upper quartiles. Subgrade condition, roadway speed requirement, asphalt concrete mix type, construction quality, and surveying time can all affect the initial IRI value. Although PCI has similar survival curves to those of reflective cracking and IRI, the hazard rate for PCI follows the Weibull distribution as shown in Figure 16 (c). The general trend is monotonically increasing, and thus the overall performance deterioration accelerates in later pavement service life for all four treatments.

Multivariate Survival Analysis

In the field, various factors or covariates can influence pavement performance. The relationship between reflective cracking and a number of such factors are evaluated here. In addition to pavement performance, the traffic, pavement thickness and pre-treatment condition are also collected in the PMIS database. Average daily traffic (ADT) information is recorded in the database and used to represent the general traffic level for each project. Multivariable survival analysis using parametric survival models was performed for the four pavement rehabilitation methods. Table 6 presents the best-fit parametric models for each treatment method via Akaike's information criterion.

Table 6. Summary of AIC test and likelihood ratio test results

Method	Fit model	Influence factors	Likelihood ratio test	
			L-R Chi Square	Prob>Chi Sq
Mill &Fill	Weibull	HMA thickness	9.365	0.002*
		Removal thickness	0.316	0.574
		ADT	0.548	0.458
SCR	Lognormal	HMA thickness	9.886	0.002*
		Removal thickness	0.025	0.875
		ADT	0.137	0.711
Overlay	Lognormal	HMA thickness	3.591	0.058
		Pre-condition	0.674	0.412
		ADT	1.346	0.246
Rubblization	Lognormal	Soil type	1.174	0.278
		Concrete thickness	1.860	0.173

The selected models may differ from those used in the univariate analysis due to the influence of the additional covariates. The likelihood ratio test results shown in Table 6 determine the significance of each covariate by comparing the log-likelihood from the fitted models. The significance level is 0.05 for this test, and corresponds to a 95 percent level of confidence. Figure 18 displays the failure function profiler for the four rehabilitation methods.

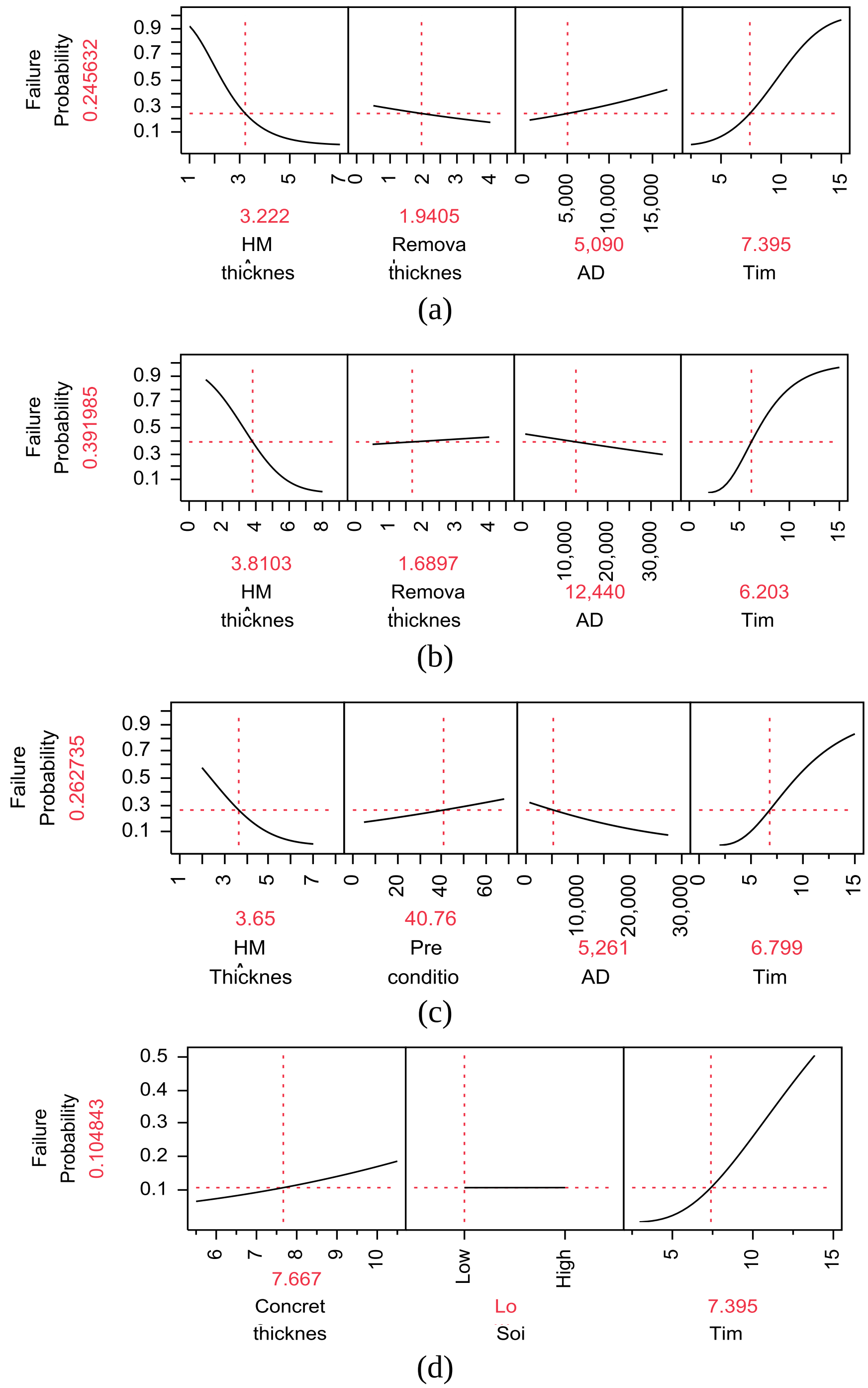


Figure 18. Influence factors on reflective cracking for (a) mill and fill, (b) SCR, (c) overlay, and (d) rubblization

The failure function/probability is one minus the survival function. This profiler can be used to show the failure probability as one of the covariates is varied while the others are held constant by dragging the red dot line in JMP. Observations from Figure 18 are discussed below.

Mill and Fill

According to the likelihood ratio tests in Table 6, the most significant factor for the failure probability of reflective cracking is the HMA thickness. The HMA thickness is the overlay thickness for the rehabilitation treatment, and the removal thickness is the milled asphalt concrete depth. In Figure 18 (a), the failure probability drops substantially as the thickness increases. The traffic level is not a significant factor; higher traffic only slightly accelerates the propagation of reflective cracking as shown in the failure probability profile.

Heater Scarification

Similar to mill and fill treatment, the most significant factor for the initiation of reflective cracking is the overlay thickness, as shown in Figure 18 (b). Removing the old HMA layer does not help retard the development of reflective cracking. In Figure 6 (b), pavements even exhibit a lower rate of reflective cracking failure with increasing traffic levels.

HMA Overlay

The overlay treatment does not require removal of the old HMA layer prior to placement of the new overlay during the construction process. Therefore, the pre-overlay pavement condition is involved in the analysis. The pre-condition refers to the old PCI values just before an overlay treatment. It is generally believed that cracks can more easily propagate through HMA overlays from severely cracked old pavements. However, Figure 18 (c) indicates that the pre-condition and failure function are not significantly related, which means that the pre-condition does not affect the reflective cracking in the new overlay. The most important factor for the initiation of reflective cracking is again the overlay thickness, although not significantly.

Rubblization

Three different rubblization types are usually performed. These include standard/full rubblization, modified rubblization, and crack and seat. Concrete pavement thickness and subgrade soil types are considered in the present analysis. Soil types at the project locations were investigated using data from the National Cooperative Soil Survey System. This system provides an interactive digital map for identifying the project locations. Soil information around these pavement sections are divided into two groups: high silt-clay and non-high silt-clay. The high silt-clay category refers to terrain reported to have more than 50 percent poorly-drained silty clay or clay loam (in ASSHTO soil classification belongs to the A-7 group). Figure 18 (d) shows that this specific categorization of soil type does not influence the survivability of pavements that have been rubblized. Modifying the rubblizing pattern to reduce impact energy and produce larger-sized broken concrete (e.g., modified rubblization and crack and seat) could provide an alternative to compensate for weak and poorly-drained subgrades. Reflective crack performance was also not significantly correlated to the underlying concrete thickness in composite pavement.

3.6 Chapter Conclusions

A method for understanding the performance of four pavement rehabilitation methods of traditional composite pavements, such as hot mix asphalt over PCC pavement, was outlined in this report. A large set of data from in-service pavements was used in survival analyses to evaluate the performance of four different composite pavement rehabilitation methods. These include mill and fill, HMA overlay, heater scarification, and rubblization.

Several conclusions are summarized as follows:

- The Kaplan-Meier estimator clearly illustrates that pavement rubblization can significantly retard reflective cracking development in composite pavements compared with the other three methods. The mill and fill treatment also exhibited better performance than HMA overlay in terms of reflective crack mitigation.
- The general trend of the hazard/failure function for reflective cracking follows a Lognormal distribution with an early-time increase followed by a constant or decreasing probability of failure. The corresponding survival function shows a sharp initial drop with a long tail in the later service life.
- No significant differences of PCI are seen in the survival analysis for the four rehabilitation methods. The hazard function for PCI, on the other hand, is best described by the Weibull distribution, which has an accelerated failure time pattern.
- The SCR method shows the lowest survival probability in terms of reflective cracking and IRI. Higher initial IRI values were found for the SCR and mill and fill treatments in the database. This finally leads to lower IRI survival probabilities for the two treatments.
- Traffic level was not a significant factor for reflective cracking according to the multivariate analysis performed in this study. Higher trafficked roads even demonstrated a lower probability of reflective cracking failure.
- Increasing the new pavement thickness is effective in retarding the propagation of reflective cracking for all four treatments. The removed pavement thickness does not significantly affect the survival probability.
- The literature shows that subgrade soil properties can influence the use of rubblization in the field (Battaglia and Paye 2011). However, this was not observed for the simple criteria considered in this report. Modifying the rubblization pattern to compensate for weaker subgrades is commonly performed by practitioners.

CHAPTER 4 PROJECT-LEVEL REFLECTIVE CRACKING MITIGATION STRATEGIES

4.1 Chapter Objective

The study objective in this chapter is to evaluate the modulus and performance of the four reflective cracking treatments. These include standard/full rubblization, modified rubblization, crack and seat, and rock interlayer. A total of 16 pavement sites were tested. In the first four sites, both FWD and SWM, were conducted for a preliminary analysis. Pavement performance surveys were also conducted during the field testing, which is intended to investigate the best treatment method in reflective cracking mitigation.

4.2 Background

Composite pavements comprise a large portion of the paved highway surfaces in Iowa and throughout the US Midwest. They are mostly the result of concrete pavement rehabilitation. The traditional pavement design approach in Iowa has been to construct thick full-depth PCC pavements. When they begin to fail years later they are overlaid with two to six inches of hot-mix of asphalt (HMA). Composite pavements, compared to traditional flexible or rigid pavements, can be a more cost-effective alternative because they may provide better levels of performance, both structurally and functionally. However, this type of pavement usually leads to reflective cracking at relatively rapid rates due to the horizontal and vertical movements in the underlying concrete slabs. The commonly attributed factors that cause movements at joints and cracks in the base PCC layer are low temperatures, wheel loads, freeze-thaw cycles, and shrinkage of PCC, HMA, and cement-treated base (Von Quintus et al. 2009). To minimize reflective cracking, four widely used treatment methods are as follows:

- Full rubblization
- Modified rubblization
- Crack and seat
- Rock inner layer

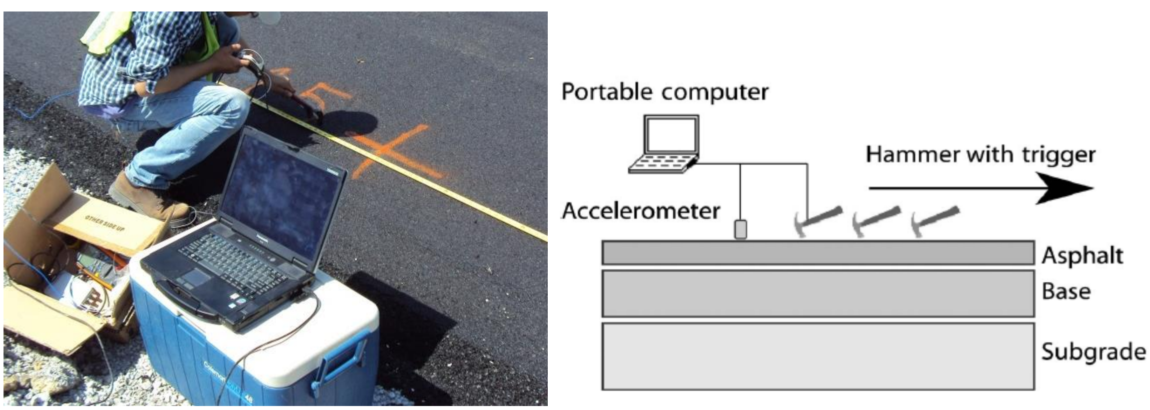
Both the rubblization and crack and seat methods are to covert an existing rigid concrete layer into a “flexible” base by breaking concrete slabs into smaller pieces. These treatments can reduce the effective slab length and minimize its horizontal movement because of thermal expansion and contraction. The sizes of broken pieces by the full rubblization are usually much smaller than the crack and seat technique. However, experience has shown that a smaller broken slab size does not always mean a better performance due to the poor subgrade condition, lack of aggregate base and the use of thin concrete pavements (Jansen 2006). One way to compensate for a weak subgrade is to modify the full rubblizing pattern to produce larger particle sizes which could maintain more of the existing concrete pavement’s structural support. The particle size specification and visual description for each treatment type follows:

- Full rubblization: typical 2 in. minus particles at surface, 6 in. to 12 in. particles at bottom of slab
- Modified rubblization: 12 in. minus particles on surface, significant surface spalling, surface appearance ranges from smooth to pulverized
- Crack and seat: typically 18 in. to 36 in. spaced cracks at surface, little to no surface spalling, spider web appearance

Rock interlayer, on the other hand, adds a “flexible” rock layer above the concrete layer to absorb the slab movement energy. The rock interlayer is generally 1 in. to 3 in. thick consisting of 3/4 in. choke stone placed wet through an asphalt paver and then static rolled (APAI 2012). The rock interlayer is surprisingly strong and durable under construction traffic. It can be directly placed over a failing PCC pavement for reflective cracking control or serves as a leveling course for pavement that has received rubblization and crack and seat treatments.

4.2 Seismic Wave Method

To measure the pavement structural modulus with the four types of treatments, nondestructive FWD and SWM testing were conducted. FWD Deflection data were collected using the JILS-20 FWD equipment by applying a step loading sequence of 9 kips at each testing location. Appendix B presents FWD surface deflections in each test section. Different from the large strain/high deflection measurement by the FWD testing, moduli obtained from the SWM testing are usually in very low strain range. Appendix C provides plots of SWM dispersion curves collected in this study. The use of SWM for nondestructive testing of pavements is not new, and its field applications become more popular after the appearance of modern spectral analyzers and powerful microcomputer (e.g., Nazarian 1984, Park et al. 1998, Ryden et al. 2002, Lin and Ashlock 2011, Lin and Ashlock 2014). Surface wave testing in this study was carried out using the multichannel simulation with one receiver (MSOR) testing system developed by Lin and Ashlock (2011, 2014). The set-up of the equipment for testing is shown in Figure 19 (a).



(a) (b)

Figure 19. Set-up of (a) surface wave equipment and (b) portable seismic acquisition system

To conduct the MOSR surface wave testing, a ball-peen hammer (12 oz.) attached with an accelerometer was used as the moving trigger impact and the other accelerometer was fixed at zero offset at the asphalt surface. The first impact offset was 10cm and the remaining impacts were equally spaced at either 5cm or 10cm increments. Tests were conducted using 12 impact locations at 10 cm incremental spacing, and then repeated using 24 impacts at 5 cm spacing. (see Figure 19 b). The dispersion data of the tested sites was extracted from the field data using the phase-velocity and intercept-time scanning scheme (Lin and Ashlock 2014). The frequencies of the dispersion data range from 100 Hz to 5000 Hz, the wavelength of which could cover the interested rock interlayer thickness. Finally, the hybrid genetic simulated annealing algorithm (Lin and Ashlock 2014) was used to back-calculate shear-wave velocity profiles for the determination of Young's modulus.

4.3 Field Data Collection and Analysis

Field tests were performed from September to November 2013. A total 17 pavement sites were tested including one traditional composite pavement (concrete without any treatment), three crack and seat pavements, two full rubblization pavement, three pavement sites only with the rock interlayer treatment and eight modified rubblization pavements. The modified rubblization takes a large portion of treatments in Iowa compared with other treatment methods due to the wide-spread silty and clayey subgrade (AASHTO A-6 to A-7 soil types). A summary of the sixteen projects route number, county, treatment type, and structural information are all listed out in Table 7.

Table 7. A summary of the sixteen projects

Location	Treatment	Structures	Location	Treatment	Structures
P29 (North), Webster Co.	Modified rubblization	6" HMA + 1" Rock + 6" PCC	L55, Mills Co.	Full Rubblization	7.5" HMA + 6" PCC
P29 (South), Webster Co.	Modified rubblization	6" HMA + 1" Rock + 6" PCC	D16, Black Hawk Co.	Full Rubblization	5" HMA + 7" PCC
D14, Webster Co.	Modified rubblization	4" HMA + 1" Rock + 6" PCC	P43, Webster Co.	No treatment	6" HMA + 8" PCC
P59, Webster Co.	Modified rubblization	4" HMA + 1" Rock + 6" PCC	Y4E, Scott Co.	Rock Interlayer	5" HMA + 1.5" Rock + 6" PCC
G61 (east), Adair Co.	Modified rubblization	4" HMA + 6" PCC	H14,Montgometry Co.	Rock Interlayer	4" HMA + 1.5" Rock + 6" PCC
G61 (west), Adair Co.	Modified rubblization	4" HMA + 6" PCC	J 40 (east), Davis Co.	Rock Interlayer	5" HMA + 2" Rock + 6" PCC
N72, Adair Co.	Modified rubblization	4" HMA + 6" PCC	J 40 (west), Davis Co.	Crack and seat	5" HMA + 6" PCC
H24, Union Co.	Modified rubblization	6" HMA + 7" PCC	Y48, Scott Co.	Crack and seat	6" HMA+8" PCC

Rock refers to the rock interlayer in this study

The route numbers for the 16 projects are designated as the project names in this study for simplicity. At each pavement site, SWM testing was taken at three to four locations.

In the first four pavement sites, modulus value for each pavement layer and underlying subgrade were measured by both the FWD and SWM for a preliminary analysis. It is intended to examine the comparability and accuracy of the measured moduli by the two methods. Temperature adjustment was not considered for the FWD and SWM moduli since the tests were performed at the same location and time in one day. The moduli values for all four test sections are shown in Figure 20.

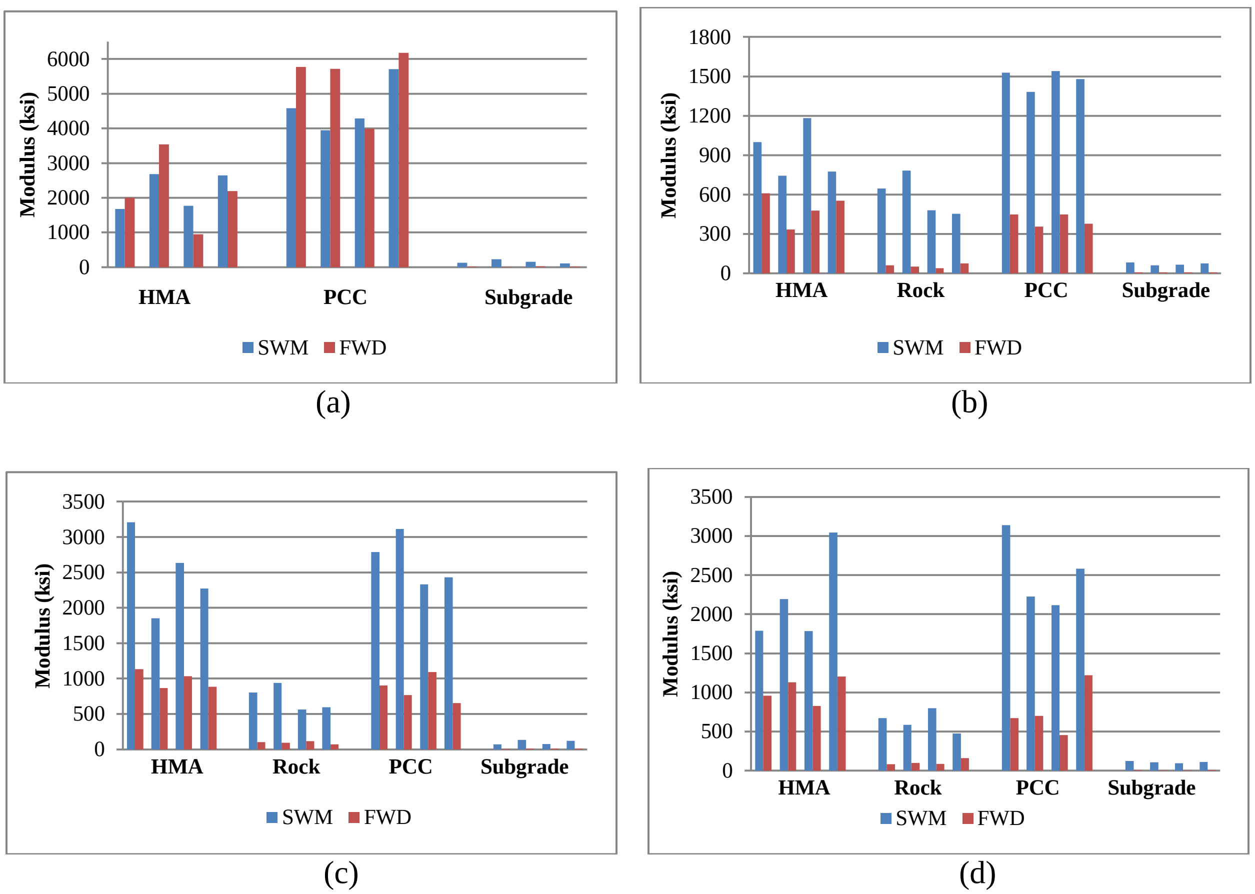


Figure 20. Comparison of FWD and SWM results for (a) D43 project, (b) P59 project, (c), P29 North project, and (d) P29 South project

As shown, the SWM moduli range from 4000 to 6000 ksi for the concrete layer without treatment. The moduli values for the other three modified rubblization sections are around 1500 ksi to 3000 ksi by the SWM testing. A good agreement was obtained between the concrete layer moduli measured by the SWM and FWD for the traditional composite pavement and the FWD test results even show slightly higher in three of the testing locations. The results of FWD subgrade moduli are almost invisible in the figure since the average subgrade modulus is just around 16 ksi. The effect of low strain amplitude becomes more evident for modified rubblization concrete layer moduli. Moduli of the modified rubblization concrete layer for the

SWM are typically higher than the FWD values by a factor of at least three as shown in Figure 20 (b, c, d). The difference could be due, in large part, to the larger strains involved with the FWD test (nonlinear behavior). As the strain increases, the moduli generally decrease (Bardet et al. 2000, Ryden and Mooney 2009) and the non-linear behavior further decreases the testing moduli values. The gap on the subgrade modulus is more obvious. The SWM values range from 65 to 200 ksi, while FWD subgrade moduli are restrained between 6.5 to 20 ksi. The average FWD subgrade modulus for the modified rubblization sections is around 8 ksi, which is lower than that of the control project (P43) without any treatment. According to the minimum strength requirement (10 ksi) for the foundation layers of rubblization pavement specified by WisDOT, the results indicate that the foundation layer of Iowa rubblized sections cannot provide sufficient strength (WisDOT 2007). Moreover, it is noticed that the FWD back-calculation is quite insensitive to yield realistic predictions of pavement response for the rock interlayer, and a wide range of moduli can be obtained between 10 to 400 ksi. In this case, the researcher should consider choosing the right initial back-calculation value to decide which output is the most representative one. Finally, the initial back-calculation value is chosen to be 90 ksi as reported by Chen et al. (2013) in his report and the final back-calculated moduli are restrained to 40–160 ksi for the rock-interlayer. It is also noticed that higher Young's moduli of the HMA surface layer are determined from the SWM and FWD tests for the two P29 project sections. Changes in the thickness of the HMA layer could be the reason. The P29 projects are placed with 6 inches HMA, while the P59 project has only a 4 inch overlay. A thicker HMA overlay could easily lead to a higher modulus on both surface and base layers.

PCC layer moduli are expected to decrease with smaller sizes of broken concrete pieces.

Figure 21 shows the average PCC layer and the rock interlayer moduli measured by the SWM in all 16 projects.

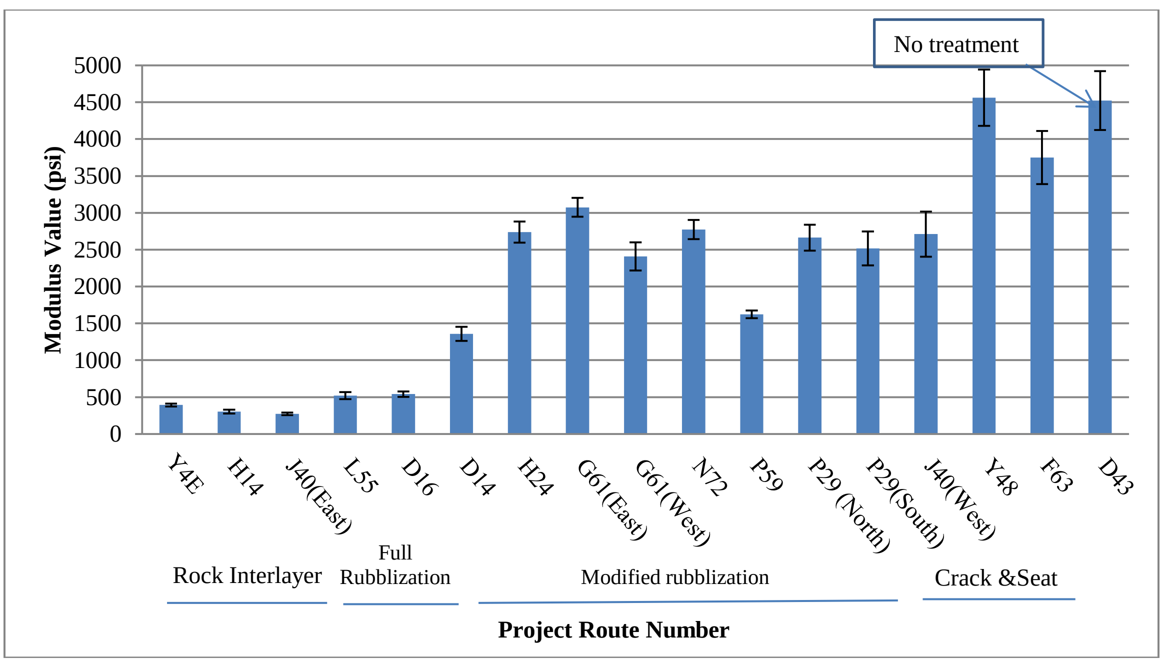


Figure 21. Mean modulus value for each project

It is apparent that the moduli for rock interlayer and full rubblized layer are much lower than the modified rubblization and crack and seat treatments. During the full rubblization, the PCC is converted to small interconnected pieces that serve as an aggregate base course. It behaves like a high-strength granular base, with stiffness very similar to the rock-interlayer formed by dense-graded choke stone. This figure also reveals that the Y48 project with the crack and seat treatment has higher moduli. This is because the Y48 project has an 8 inch thick concrete layer, as listed in Table 7, and the project also used high density steel slags in the HMA layer. The standard error bar for each project is added. It exhibits higher variability as the material stiffer. In order to evaluate whether these methods used above have statistical moduli difference or not, the all pairs Tukey-Kramer honest significant difference (HSD) method is perform for multiple comparisons. The test performs an actual comparison when the sample sizes are unequal and gives more conservative results compared with other multi-comparison tests (Hayter 1984). As can be seen in Table 8, the statistical test shows that the intact PCC layer gives significant higher values, while the rock interlayer and full rubblization both belong to the lowest group. Crack and seat and modified rubblization layers both sit in the middle level.

Table 8. PCC layer moduli by multi-comparison test

Method	Ranking	Mean (ksi)
No treatment	A	4630.4
Crack and Seat	A	3673.3
Modified rubblization	B	2381.4
Rubblization	C	529.8
Rock interlayer	C	322.6

The moduli of PCC layer is expected to decrease as the size of broken concrete pieces decreases.

Figure 21 shows the average PCC layer and the rock interlayer moduli values measured by the SWM for all 16 projects. It is apparent that the moduli for rock interlayer and full rubblized layer are much lower than the modified rubblization and crack and seat treatments. During the full rubblization, the PCC slab is broken into small interconnected pieces that serve as an aggregate base course. It behaves more like a high-strength granular base, with stiffness close to the rock-interlayer formed by dense-graded choke stone. The results also demonstrate that the Y48 project with the crack and seat treatment has unreasonably high moduli. This is because this project has an 8 inch thick concrete layer, as listed in Table 7, and high density steel slags in the HMA layer. The error bar for each project is added indicating standard error. As expected, the error increases as the material get stiffer. In order to evaluate whether these methods used above have a statistical moduli difference or not, the all pairs Tukey-Kramer HSD method was used for multiple comparisons. The test can perform actual comparison when the sample sizes are unequal and give more conservative results compared with other multi-comparison tests (Hayter 1984).

Pavement performance surveys were conducted after the field testing on a randomly selected 0.4 mi. section along each pavement project. It is intended to investigate which treatment could be more effective in minimizing reflective cracking based on field performance. Considering the common plain concrete pavement joint spacing, transverse cracks in regular and appropriate space interval (around five to six meters apart) are considered reflective cracks. The distress survey for reflective cracking follows the method defined in the "Distress Identification Manual for the Long-Term Pavement Performance (LTPP) Project." Reflective cracking survey results are summarized in Table 9 and Appendix D provides some selected pictures from visual distress surveys.

Table 9. Summary of pavement project reflective cracking condition

Project	Service Year	Reflective / Transverse cracking Condition	RCI	Project	Service Year	Reflective / Transverse cracking Condition	RCI
P29 (North)	1	No cracks	0	L55	9	4 small, 15 medium, and 3 large size	67
P29 (South)	2	No cracks	0	D16	9	3 small, 16 medium size	51
D14	3	No cracks	0	P42	8	6 small, 4 medium and 19 large size	132
P59	3	No cracks	0	Y4E	2	No cracks	0
G61 (east)	9	15 small, 15 medium, and 5 large size	90	H14	6	1 medium and 4 large size	13
G61 (west)	9	3 small, 5 medium size	18	J 40 (east)	8	6 small, 7 medium and 2 large size	39
N72	9	2 medium and 4 large size	14	J 40 (west)	8	7 small, 8 medium and 4 large size	55
H 24	8	2 small, 4 medium and 2 large size	26	Y48	3	No cracks	0

Severity levels: Low, Medium, High; representing numbers of low, medium and high severity reflective cracks

In general, none of the pavement sites have severe rutting problems implying that both the rubblized concrete fragments and the choke stone materials could possess enough shear strength for rutting resistance on low traffic-volume county roads. A lack of comparable control pavement sections prevents a firm conclusion about the ability for these treatments in reflective cracking mitigation. However, it is still obvious that pavements received the treatments exhibited good performance (no reflective cracks) within the first three years of service time. To quantify the amount of reflective cracking, a simple RCI formula is developed in Chapter 3 and shown in Equation 3.

$$RCI = Low \times 1 + Medium \times 3 + High \times 6 \qquad (3)$$

The index is calculated based on the extent of reflective cracking and a weighting function of the crack severity to account for the condition of reflective cracking. A larger size reflective cracking has a higher weighting factor. Results show that the P42 project with no treatment exhibits the worst condition/highest RCI value. Both two projects by full rubblization developed moderate amount of reflective cracking, which are not well-performed as expected. The only one comparable section is the two J40 projects. The first part used crack and seat, and due to fears of pavement fails and potential cost, rock interlayer was placed later in the east part. It appears to show that the crack and seat is less effective than the rock interlayer for reflective cracking control, but not obviously. Most of the rock interlayer and modified rubblization projects have good pavement performance with slight amount of cracking. However, more projects should be investigated to support the idea. Appendix D provides some selected pictures from the visual distress surveys.

Finally, the measured SWM moduli in this study are compared to others' research finding as presented in Table 10.

Table 10. Comparison of layer moduli values

Technology	Composite pavement (PCC layer)	Crack and Seat PCC	Full rubblized PCC	Rock interlayer / Granular base
SWM in this study	3940–5708 ksi	1118–5323 ksi	441–587 ksi	230–430 ksi
SWM	3512–6492 ksi (Alexander 1992)	N/A	80–400 ksi (Gucunski et al. 2009)	N/A
FWD	6929–9426 ksi (Alexander 1992)	1232–7977 ksi (Korsgaard et al. 2005)	38–122ksi (Ceylan et al. 2008)	43–100 ksi (Chen et al. 2013)

As a relatively new method, no literature was found for the modified rubblization information and it is not involved in the comparison. Alexander (1992) conducted both the SWM and FWD tests on traditional composite pavement where the PCC layer moduli obtained by SWM are slightly lower the FWD test. The same trend is seen in this study and our measured moduli are very close to his results. Using the FWD for crack and seat concrete moduli testing, Korsgaard et al. (2005) noticed that the moduli could change significantly before and after the asphalt overlay, and "between" or "on" the cracks. Its values vary from 1200 ksi to 7900 ksi. Gucunski et al. (2009) performed the SWM test directly on highly crushed rubblized concrete layer and the results are listed in Table 10. It shows that our SWM moduli for the full rubblized layer are slightly higher and much less variable when tested on top of a HMA overlay.

4.4 Chapter Conclusions

Four pavement reflective cracking mitigation treatments were evaluated in this report. These include full rubblization, modified rubblization, crack and seat, and rock interlayer. Both modulus and pavement performance were assessed and the conclusions summarized are as follows:

- SWM is a viable method for in situ material characterization of pavement systems. PCC modulus values from the SWM compares well with the FWD result on traditional composite pavement.
- The effect of SWM low strain amplitude was evident in the measurement of modified rubblization layer. The SWM moduli are typically two to three times higher than the values predicted by the FWD.
- The SWM can be used effectively to determine the moduli of thin rock interlayer, while the FWD has difficulty in measuring and back-calculating the thin layer moduli.
- For the four treatment methods, the crack and seat treatment has the highest moduli, followed by the modified rubblization layer. The full rubblization layer and the rock interlayer give similar, but lower, moduli.
- Field performance show that the traditional composite pavement site has the highest amount of reflective cracking. A moderate amount of reflective cracking was observed for the full rubblization projects. Poor subgrade soil properties could be the reason to influence the use of rubblization.
- It is recommended to use the rock interlayer and modified rubblization in the field. However, more projects should be monitored to support the idea.

CHAPTER 5 ECONOMIC COST ANALYSIS

5.1 Cost Analysis

An accurate economic and pavement performance evaluation is difficult due to the lack of detailed construction materials information and relatively scattered pavement performance. The following are some limited pavement life and cost comparisons.

Firstly, one should rate the mitigation strategies based on their success and risks in real application. After that, the service life was further used for the cost-effective analysis. A high risk (via a low probability value) implies that there is no confidence the treatment method will perform as expected or designed. Conversely, a high risk means that there is full confidence that the method would perform well. The overall risk rating is partly based upon the research results conducted in the previous part of the this research study and partly based on the previous literature review results as shown in Table 11.

Table 11. Risk of reflective cracking failure by various pavement rehabilitation methods

Methods	Risk of Reflective Cracking Failure (this study)	Risk of Reflective Cracking Failure (Von Quintus et al. 2010)
Full rubblization	Moderate	Low
Crack and Seat	Moderate	Low
Modified rubblization	Low	N/A
Asphalt milling	Moderate	Low
Heater scarification	High	Moderate
Rock interlayer	Low	Moderate
Asphalt flexible interlayer	Moderate	Moderate
Direct asphalt overlay	High	High

It should be noted that a higher probability value does not necessarily mean that the strategy listed is the most cost effective repair method for the conditions noted. As can be seen, in the study conducted by Von Quintus et al. (2010), full rubblization and crack and seat were considered to be the best reflective cracking mitigation treatments. However, due to the excess of fat clay (clay of high plasticity) in Iowa, modified rubblization and rock interlayer are the alternatives for the use of full rubblization. In both studies, the direct asphalt concrete overlay seems to have the highest risk in reflective cracking. A higher reflective cracking failure risk could lead to a shorter pavement service life and more frequent pavement rehabilitations.

The cost estimates in Table 12 suggest that modified rubblization, crack and seat, rock interlayer and heater scarification are the cheapest treatments. However, heater scarification has been considered to have the lowest survival probability in terms of reflective cracking and IRI and the crack and seat is a less effective treatment compared to the rock interlayer for reflective cracking control in previous project chapters. It has to be noted that all these values are all approximated results. For example, the in-place compaction density and asphalt content should be used in the calculation of the placing of HMA overlay price. However, in this study, we can just assume that all of the projects share a same HMA overlay price due to the lack of information.

Besides the treatment cost, time is another important factor that the transportation agencies should consider before determining which treatment should be applied. Major reconstruction treatments can cause a significant amount of congestion and can be a costly and time consuming irritant for drivers. Agencies often receive criticism that their major multi-year capacity improvement projects create a large amount of extra travel time. Especially when an existing roadway must be removed completely and the new and expanded pavement built from the ground up. Table 13 lists out the ranking of cost, time, and energy use for each type of treatment.

Full Rubblization

The cost and time savings of full rubblization is ranked in the middle, which cost 60 percent less than the cost of normal PCC removal and take approximately one-fifth of the time (RMI 2014). However, the performance of full rubblization is not well-performing in Iowa, although it might have better performance in other areas. One example is shown below.

One rubblization project was conducted on I-88 by the Illinois State Tollway Authority. An 8 inch layer of asphalt was placed in three lifts over the newly crushed concrete aggregate. The method increases pavement life expectancy, and is less expensive than removal of the concrete and total replacement of the road. The construction cost for the pilot project was $3.7 million. The direct savings in construction cost by using the rubblization technique as opposed to removal of the old pavement and replacement with imported material was estimated to exceed $1 million.

Heater Scarification

Heater scarification increases the chance of reflective cracking. It is up to 100 percent recycled material and reduces hauling process; therefore it is a low cost treatment. Compared to a conventional mill and fill, it saves more time and money. The heater scarification technique scarifies the existing HMA surface layer to a depth of approximately one to two inches so that the upper portion of any crack can be removed. The lower portion of the crack is sealed because of the heating process and a rejuvenating agent is applied to soften the surface of the oxidized or aged HMA. The heater scarification method has been widely used in highway pavements as a reflective crack mitigating strategy. Some projects exhibited good results, while others have not. In Arizona, heater scarification with Reclamite plus a 1.3 in. (32 mm) wearing course was ranked as the third best among 18 test treatments (Way 1980). After six years only 7.4 percent of reflective cracking was observed. In Quebec, however, scarification with Reclamite plus a 1.3 in. (32 mm) wearing course resulted in 100 percent reflective cracking after only two years (Poon 1986). In New Mexico (McKeen and Pavlovich 1984), 0.75 in. (19 mm) scarification with a rejuvenating agent plus 0.63 in. (3 mm) seal coat and 2 in. 50 mm) surface course resulted in 70 percent of reflective cracking within four years. This could be because the heater scarification can only remove the surface cracking and the deeper cracks cannot be eliminated. The Canada Construction Association analyzed energy costs for different treatments. The “hot in situ” recycling of the existing road surface can save some energy use compared with traditional HMA overlays. This method can not only lead to a 15 percent reduction in overall energy use per ton laid down, but also save a considerable quantity of resources.

Crack and Seat

The crack and seat method is a quick concrete breaking process, which usually takes one hour for one mile of treatment. However, the treatment only creates hairline cracking on the pavement surface (many of them are not visible). Sebesta et al. (2005) reported that crack and seat has been widely used in West Texas but there has been little or no evaluation of the success or failure of this treatment. Most of the treatments have been reported to be working well; however, problems have been encountered when using this treatment on pavements with untreated subgrades. Crack and seat was the poorest performing treatment on the US 59 experimental sections that were constructed just north of Corrigan in the Lufkin District of Texas.

Structural Overlay

In general, increasing thickness of the HMA overlay can reduce the load-associated damage by reducing the effect of poor load transfer across a crack or a joint in the underlying pavement, and thus, can effectively improve the pavement performance. It is suggested that the overlay thickness required to retard the reflective cracks depends on four factors:

1. Type of pavement being overlaid – HMA or PCC; HMA overlay thickness of flexible pavements is generally less than that for JPCP or jointed reinforced concrete pavement (JRCP).
2. Type of distress of the pavement – alligator cracking, block cracking, transverse cracking, longitudinal cracking, or PCC joint cracking; thicker overlays are generally needed for any type of transverse crack or joint because of the horizontal movements.
3. Climate – the greater the variations in seasonal and daily temperatures, the greater the HMA overlay thickness.
4. Number and weight of axle loads – the higher wheel loads or weights and the higher the traffic volume, the greater the overlay thickness.

The greatest benefit from the use of thicker overlays of PCC slabs as can be seen from the result in this research study. A study done by Gulden and Brown (1984) in Georgia also found that the occurrence of reflective cracks decrease considerably as the overlay thickness increases. They recommended a minimum overlay thickness of 100 mm (4 in.) when no other treatment is used. The energy use for HMA structural overlay is high as shown in Table 13. The production of hot-mix asphalt concrete was the most energy intensive activity in the road rehabilitation process, accounting for about 70 percent of the total energy use. Transportation accounted for between 20 and 25 percent of the total energy consumed, and heavy equipment use accounted for less than 10 percent of all energy consumed.

Asphalt Interlayer

Interlayer mixes will typically cost more than a conventional HMA mixes since highly polymer modified asphalt is used in the design. To determine the additional cost of using an interlayer, the published bid quantities from a demonstration paving project done by Des Moines Asphalt on US 169 in Adel, Iowa were used to analyze the cost differences between the pavement section with and without the interlayer. The length of the interlayer pavement section was 1,975.5 ft and the length of the non-interlayer section was 1,976 ft, the bid quantities were appropriately divided for assessing the cost of the two pavement sections.

The total cost for constructing the HMA overlay without the interlayer was $157,759.03 and the total cost for constructing the HMA overlay with the interlayer was $174,479.61 (see Table 13).

Table 12. Interlayer cost comparison from contractor bid tab

Item Description	Quantity (Ton)	Unit Price	Amount
Overlay with no Interlayer			
HMA 1/2" Surface Course	817.35	$ 55.00	$ 44,954.25
HMA 1/2" Intermediate Course	826.00	$ 55.00	$ 45,430.00
Asphalt Binder PG 58-28	126.17	$ 534.00	$ 67,374.78
Total			**$ 157,759.03**
Overlay with Interlayer			
HMA 1/2" Surface Course	817.35	$ 55.00	$ 44,954.25
HMA 3/8" Interlayer Course	412.00	$ 74.00	$ 30,488.00
HMA 3/8" Intermediate Course	413.80	$ 74.00	$ 30,621.20
Asphalt Binder PG 58-28	94.53	$ 534.00	$ 50,479.02
Asphalt Binder PG 64-34	24.70	$ 726.20	$ 17,937.14
Total			**$ 174,479.61**

This equates to a 10.6 percent increase in materials and paving costs for constructing an HMA overlay with an interlayer. The benefit of the additional costs was realized from the 29 percent reduction in transverse cracking after the first year of paving with a decrease in the severity of the transverse cracks (41 percent moderate severity versus 4 percent moderate severity). Furthermore, the reduction in cracking would more than likely have been greater if the field produced interlayer met the volumetric and laboratory performance testing requirements.

PCC Layer Removal

Concrete pavement breaking and removal is a full-depth repair process as discussed previously. There are two main methods for concrete slab break and removal: lift-out and break up. It is preferable to lift the deteriorated concrete whenever possible. Lifting the old concrete imparts no damage to the subbase and is usually faster and requires less labor than any method that breaks the concrete before removal. Sometimes concrete joints or cracks are so deteriorated that it is unsafe to remove them by lift-out. In these cases it is necessary to break the deteriorated concrete into smaller fragments for removal by a backhoe.

Rock/Stone Interlayer

The Louisiana Department of Transportation and Design (LADOTD) has been successfully using a form of stone interlayer as a mean to reduce HMA reflective cracking from soil cement layers. The initial stone/aggregate interlayer field section (one mile) was placed in 1991 on IA 97 near Jennings. The stone interlayer design (3.5 in. HMA/4 in. unbound aggregate base/6 in. cement stabilized base) was compared to a conventional design (3.5 in. HMA/8.5 in. cement stabilized base). After 10 years of service the stone interlayer section had about 50 percent less cracking than the conventional section. Furthermore, the majority of cracking on the stone interlayer section was in the slow severity level (Buchanan 2010). In Iowa, the cost of stone/rock interlayer would be further cost-effective, since the thickness of rock interlayer in Iowa usually ranges from one to three inches thick. The advantages of a rock interlayer with an HMA overlay are low cost and fast to construct as shown in Table 13.

Table 13. Cost, time, and energy consumption for each treatment

Methods	Cost	Time	Energy
Full rubblization	Medium	Medium	Moderate
Modified rubblization	Low	Short	Low
Crack and Seat	Low	Short	Low
Flexible interlayer	High	Medium	Moderate
Rock interlayer	Low	Medium	Low
Heater Scarification	Low	Short	Moderate
Asphalt Milling	Medium	Long	High
Direct asphalt overlay	Medium	Medium	High
Reconstruction of PCC layer	High	Long	High

Another major benefit is the potential for reduced energy demand relative to conventional flexible and rigid pavement systems. The total end use energy demand of unbound “granular materials” or aggregate is about 80 percent less than hot mix asphalt or concrete. However, the problem for deformation resistance of stone interlayer to deformation on high-volume road is still a question.

Modified Rubblization

"Modified rubblization" is referred as "aggressive crack and seat." On weak subgrades and a thin concrete layer, the multi-head breaker offers the advantage of being able to lower the hammer height and separate the impacts to allow the PCC to be broken in a less aggressive manner that produces a more suitable surface for an HMA overlay. This "modified rubblization" technique with the MHB typically results in fractured concrete pieces larger than the sizing criteria normally allowed, but still can provide full-depth fracture and a surface suitable for an HMA overlay. A few states formally recognize this "modified rubblization" process where particle size criteria is waived by having a separate bid price (typically slightly less costly) and an estimated quantity in the bid documents. "Modified rubblization" should only be used when it has been deemed that the thin slabs and weak support conditions are preventing the specified size criteria from being met. The modified rubblization treatment is most commonly used in Iowa.

5.2 Rehabilitation Strategy Selection

The key to designing an adequate rehabilitation strategy over a design period is to select the right treatment method for the right condition application. As a part of this research project, a small interactive Windows program was developed to assist engineers in selecting the best treatment for reflective cracking mitigation. The decision strategies are mainly based on the decision trees proposed by Von Quintus et al. (2010). The detailed internal logic for the decision tree can be seen in Figure 11 and Figure 12. Based upon the decision tree information provided, the pre-existing pavement cracking conditions shown in Table 14 were selected. Different reflective cracking mitigation methods should be used when a specific pre-existing pavement distress condition exists.

Table 14. Selected pavement distress for reflective cracking mitigation

Rigid Pavement	Flexible Pavement
Faulting	Structural Fatigue Cracking
Load Transfer	Moisture Damage
Subgrade Soil Condition	Transverse & Longitudinal Cracking

The main screen is in Figure 22.

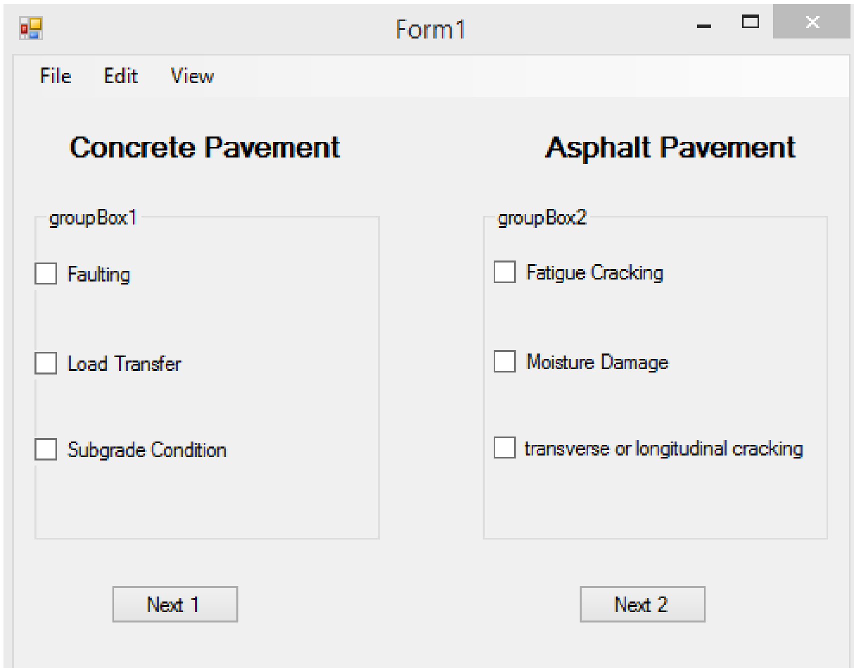


Figure 22. Main screen of interactive program

In the current program we list out six different types of distresses for both concrete pavement and asphalt pavement. These distress types are considered as the pavement condition before rehabilitation or reconstruction. In order to check each distress type option, just click the checkbox left to the distress type (i.e., Load Transfer), then click the **Next** button.

Once clicked the **Next** button, a second-level window would be opened as shown in Figure 23.

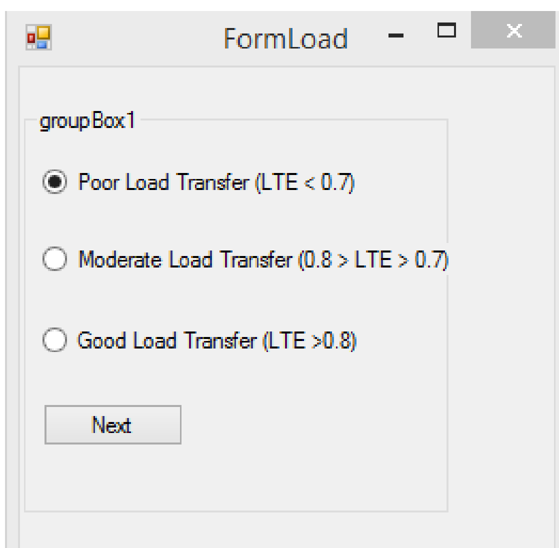


Figure 23. Second-level window in interactive program

The second-level window provides another two to three different levels of the pavement distress condition from severe condition to good condition. Figure 23 shows an example of concrete pavement load transfer. If the exsiting pavement has a poor load transfer with LTE < 0.7 (LTE is the load transfer efficiency), then click the first option for the Poor Load Transfer.

Finally, a message box would pop out showing which treatment would be recommended as shown in Figure 24.

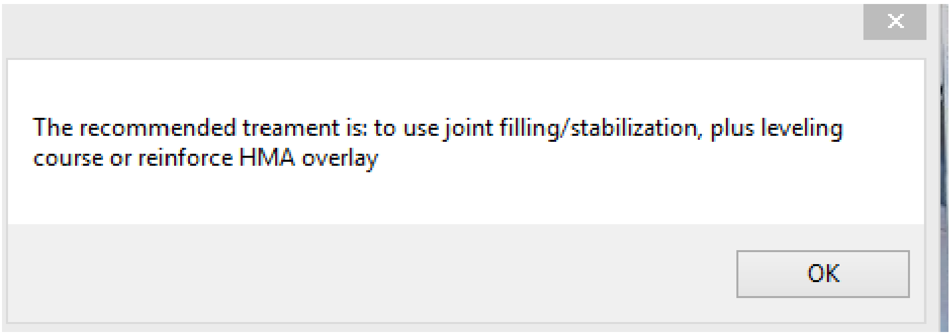


Figure 24. Message box for treatment selection

In the example for moderate load transfer, the recommended treatment is to use joint filling/stabilization, plus leveling course or reinforced HMA overlay.

CHAPTER 6 PERFORMANCE REVIEW FOR COLD-IN-PLACE RECYCLING

6.1 Introduction

A steady increase in the use of cold in-place recycling for pavement rehabilitation has highlighted the need for a performance review of the CIR pavements on the Iowa network. This review will be of interest to owner/agencies, contractors and the research community. This chapter presents pavement performance data of cold in-place recycling rehabilitation tracked over time. Other pavement rehabilitation strategies included in the report employ a statistical-based survivability analysis; however, it is premature to perform such an analysis on the CIR dataset because of the low number of pavement failures. Failures for reflective cracking were initially evaluated using the RCI. The RCI value is calculated from counting transverse cracks, compared to the other rehabilitation strategies. The trigger value to indicate failure was 420 cracks/km; however, it was found that pavements rehabilitated with CIR have a relatively low RCI value compared to other rehabilitation strategies today. The reflective cracking index was calculated for CIR pavements and only four projects out of 100 CIR projects had sections reaching the 420 cracks/km and thus cannot be directly compared to the other rehabilitation strategies in this report. Reflective cracking is common in composite pavements, often occurring in the asphalt overlay above a joint or crack in the underlying PCC layer (Huang 2004). CIR lends itself to a particular type of underlying pavement structure, which may be less prone to reflective cracking; however, ride quality and the occurrence of cracking still has significant economic impact. Underlying pavement structure may not include PCC joints which would reduce the occurrence of reflective cracking. Incorporating a successful long-term pavement rehabilitation strategy into initial pavement design of a pavement will help to find the most economical pavement while maintaining performance. Performance reviews of successful pavement rehabilitation strategies can be implemented into lifecycle cost analyses that will assist engineers in making the best rehabilitation choices for their pavement network.

6.2 Research Plan and Methodology

Pavement performance data was provided for CIR projects from the PMIS collected by the Iowa DOT. The PMIS provides valuable information about various types of cracking, rutting, IRI, and other pavement characteristics that are tracked biennially. Approximately 100 CIR projects on the Iowa network have been tracked in the PMIS system to date. The oldest available CIR pavement monitored in the PMIS database is from 1995; however, older CIR pavements exist in Iowa. Figure 25 shows the number of CIR projects per year and trends show a steady increase in CIR projects. Figure 25 is also highly correlated with the total number of CIR lane-miles observed for performance and each project shown represents several sections that were recorded in the PMIS database during the pavement condition biennial review, totaling 735 follow-up CIR observations for up to 19 years post-construction.

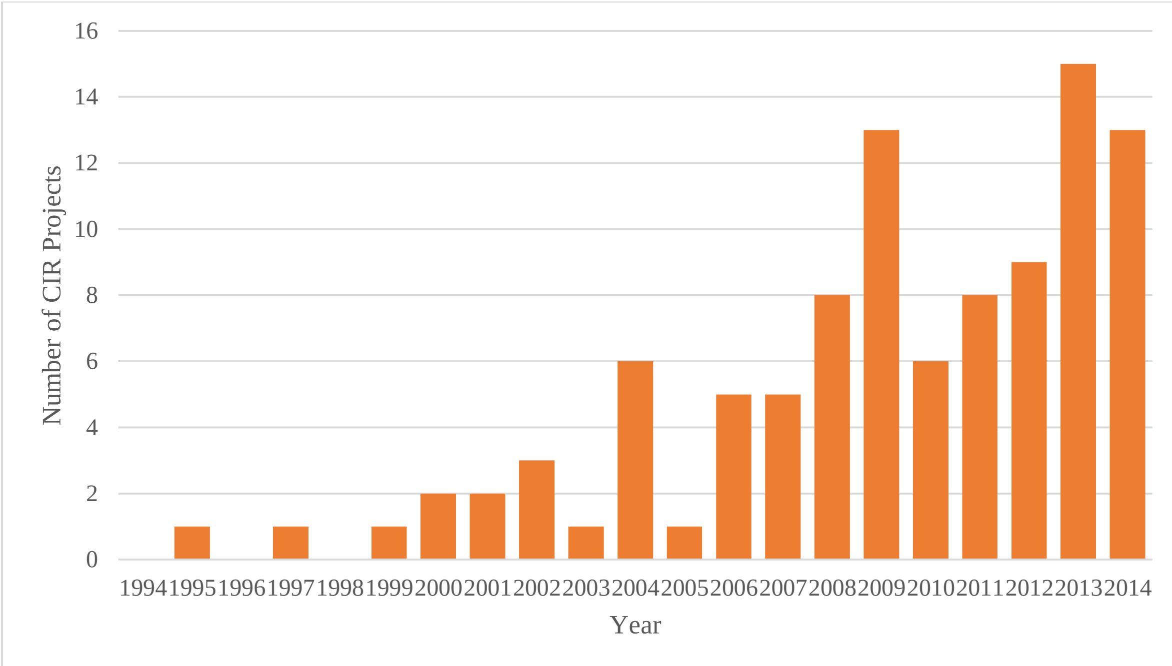


Figure 25. Number of CIR projects tracked in PMIS database

Figure 26 shows the number of CIR pavement sections that were observed for each year following the construction of a CIR treatment.

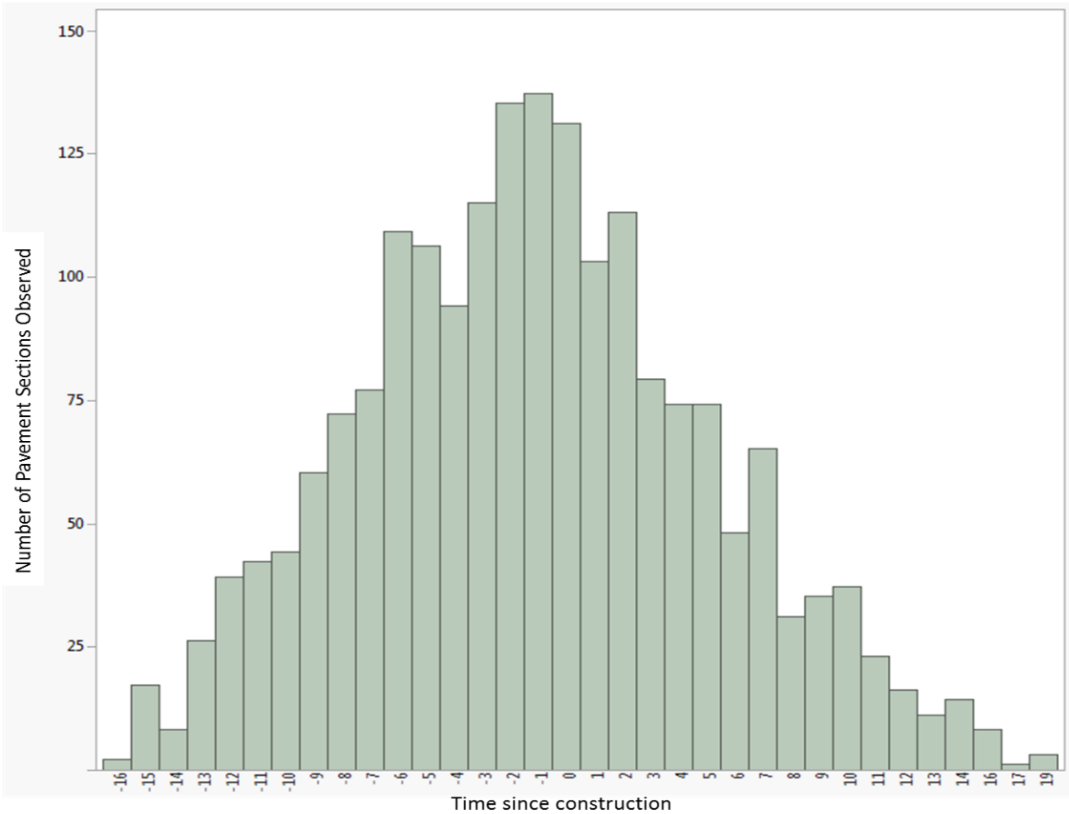


Figure 26. Number of pavement observations for each year past construction

Due to the low number of projects observed past 12 years post-rehabilitation, one cannot apply general conclusions to the data set for latter years. The current trends in the latter years are shown, but are only a small sampling of total projects that currently exist. As pavement monitoring continues, a larger population of pavements will have post-rehabilitation data 12 years beyond the CIR placement and more general conclusions can be drawn. The other concern was the length of the observed sections. The average length only varied slightly over time and the distress observations are normalized over the length; for these reasons, it is assumed that the length of the observed sections is negligible.

This chapter will look at CIR pavement performance as a function of time. The pavement performance results will be shown as pre- and post-rehabilitation. Pre-rehabilitation data is indicated by a negative number for the time since construction, the x-axis, represent the number of years prior to rehabilitation. The pavement distresses summarized in this report include transverse cracking, longitudinal cracking, wheel path longitudinal cracking, fatigue cracking, rutting, patching, and IRI. Further analysis of the IRI included a predictive model based on the observed values showing the influence of the CIR layer thickness and time on IRI.

6.3 Transverse Cracking

Transverse cracks are defined as predominately perpendicular to the pavement centerline. The cracks are categorized in the LTPP Distress Identification Manual as follows (Miller and Bellinger 2003):

Low (TRANS_L) - an unsealed crack with a mean width ≤ 6 mm; or a sealed crack with sealant material in good condition and with a width that cannot be determined

Moderate (TRANS_M) - any crack with a mean width > 6 mm and ≤ 19 mm; or any crack with a mean width ≤ 19 mm and adjacent low severity random cracking

High (Trans_H) - any crack with a mean width > 19 mm; or any crack with a mean width ≤ 19 mm and adjacent moderate to high severity random cracking

Iowa pavements are susceptible to transverse cracking because of large variations in temperature and very cold temperatures. Transverse cracking can also appear in the form of reflective cracking from joints in PCC pavement deeper in the pavement structure (Huang 2004). The transverse cracking in the CIR sections are shown in Figure 27.

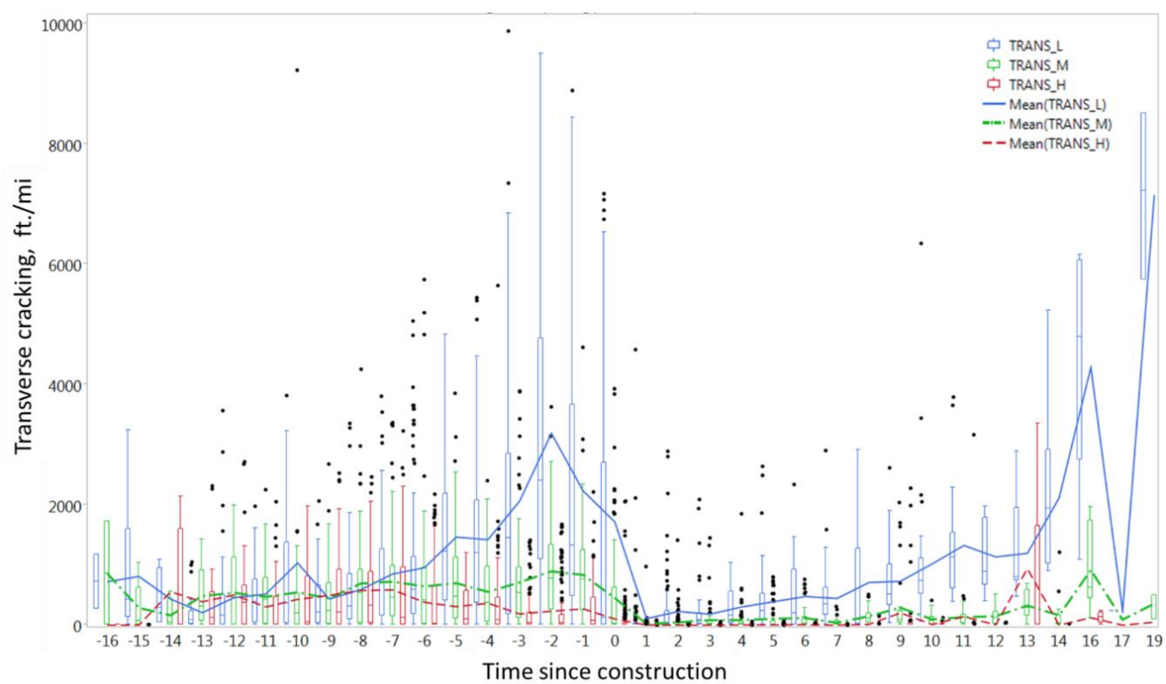


Figure 27. Transverse cracking before and after CIR rehabilitation

The low severity transverse cracking shows a steady increase while medium and high transverse cracking stay relatively constant prior to the time of pavement rehabilitation at time zero. After the CIR rehabilitation on the pavement, there is no measurable transverse cracking in any of the pavements rehabilitated. There is a slow but steady increase in the low-severity transverse cracking but high and medium severity cracking stays consistently low for the first 12 years. The low distresses seem to increase after fourteen years of observation but this interpretation may be misleading due to the decreasing number of projects monitored during latter years, as shown in Figure 27. There are also some sections that are shown as outliers having approximately 2000 ft/mi. of low-severity transverse cracking after a few years in service. Several of the data points that are showing reduced performance in just a few years after construction are different sections within the same projects. In the future, a more complete data set will be available which will allow for a better understanding of CIR performance after 12 years post-construction and once failure triggers are met, roadways can be evaluated using a survivability analysis or others as appropriate.

6.4 Longitudinal Cracks and Wheel Path Longitudinal Cracking

Longitudinal cracking is defined as cracks predominantly parallel to the pavement centerline. Location within the lane (wheel path versus non-wheel path) is significant. The three severity levels are defined as follows:

Low - A crack with a mean ≤ 6 mm; or a sealed crack with sealed crack with sealant material in good condition and with a width that cannot be determined

Moderate - Any crack with a mean width > 6 mm and ≤ 19 mm; or any crack with a mean width ≤ 19mm and adjacent low severity random cracking

High - Any crack with a mean width > 19 mm; or any crack with a mean width ≤ 19 mm and adjacent moderate to high severity random cracking

There are two types of longitudinal cracking measured in the PMIS, wheel path and non-wheel path longitudinal cracking. Figure 28 displays the longitudinal cracking for the non-wheel path cracking and shows that the average longitudinal cracks are reduced for some time after rehabilitation and high and medium severity cracks remain low throughout the analysis period.

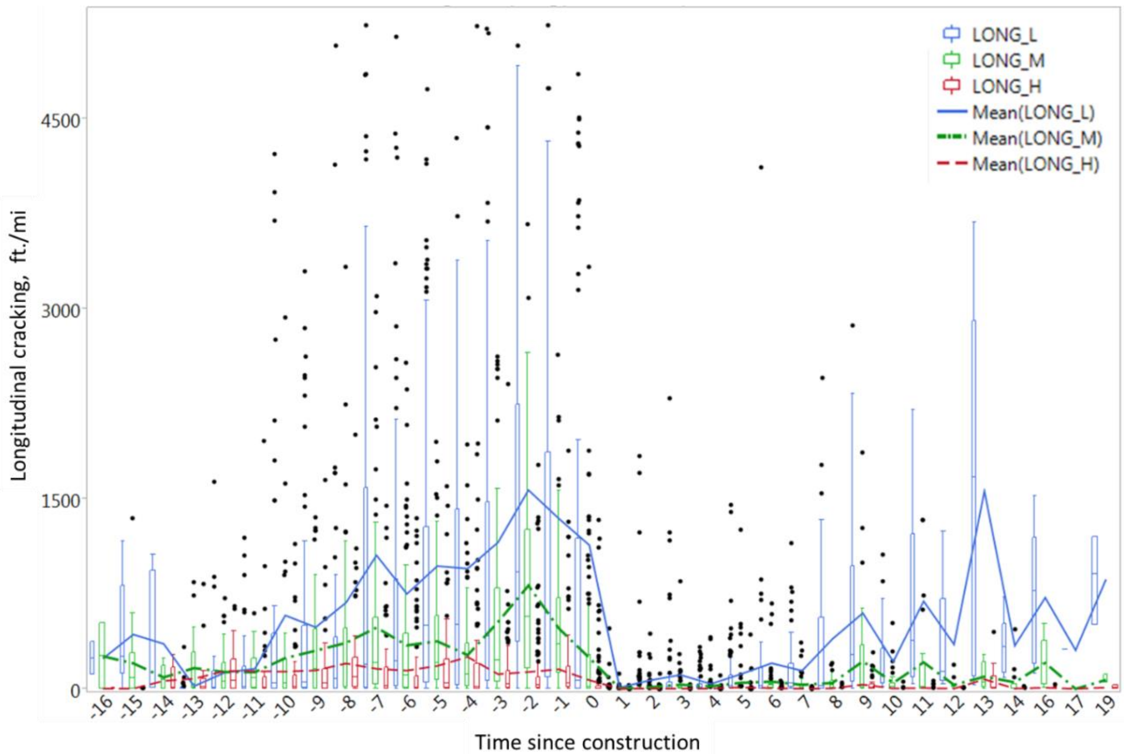


Figure 28. High, medium, and low severity non-wheel path longitudinal cracking

This is an indication that fatigue cracking will be low also. There is a slow increase in the longitudinal cracking but the averages shown for longer post-construction observations represent fewer projects. This limits the ability for global conclusions but the data shows an overall improvement in longitudinal cracking. The longitudinal cracking measured in the wheel path is shown in Figure 29.

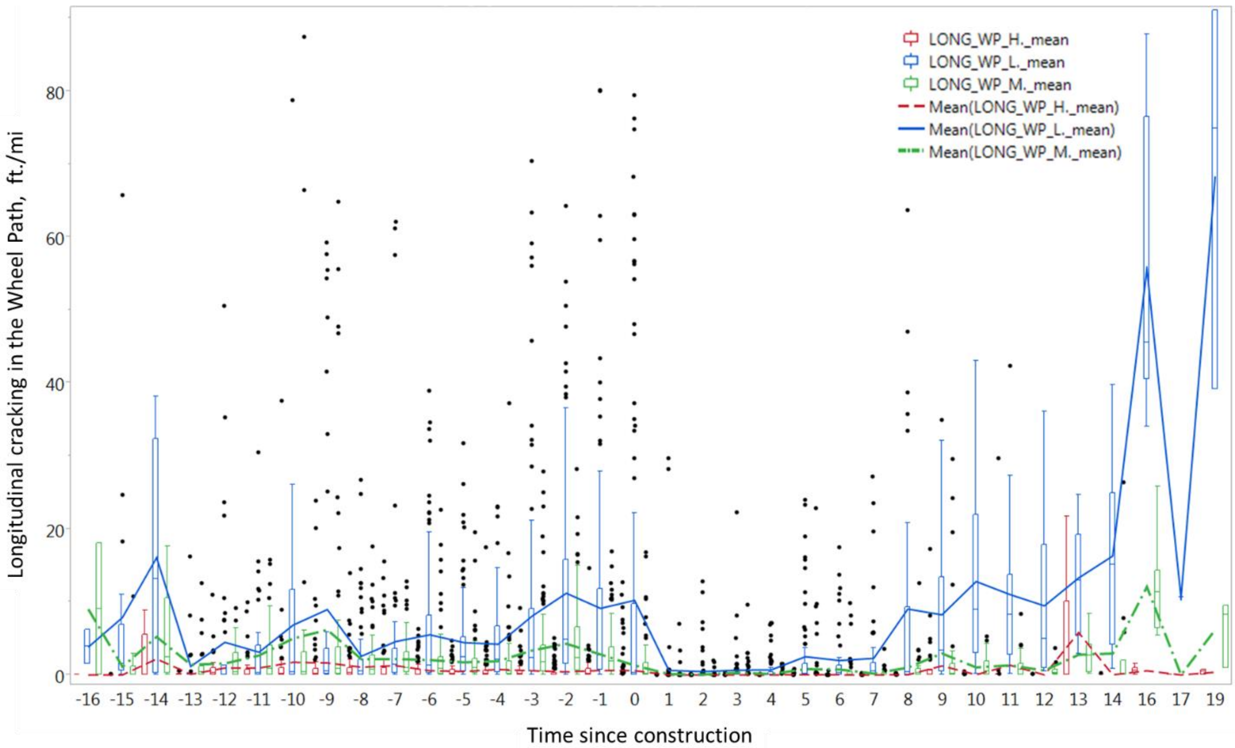


Figure 29. High, medium, and low severity longitudinal cracking in wheel path

The low severity cracking in the wheel path begins slow and then begins to steadily increase after eight years. The data past 10 years post construction only represent a small number of projects so the increasing in shown in 14–19 years post-construction may not be representative. Although low-severity cracking appears to increase, the medium severity and high-severity longitudinal cracking stay relatively low.

6.5 Fatigue Cracking

Fatigue cracking, also called alligator cracking is a load related distress and thus, does not typically occur until the pavement has been loaded many times. The cracking is generally slow to develop but begins to increase rapidly as the pavement weakens. The cracks usually begin as longitudinal parallel cracks but after repeated loading, form a series of interconnecting cracks that resemble the skin of an alligator (Huang 2004). The fatigue cracking is categorized as medium and high severity in the PMIS system. The fatigue cracking of the monitored pavement sections is shown in Figure 30.

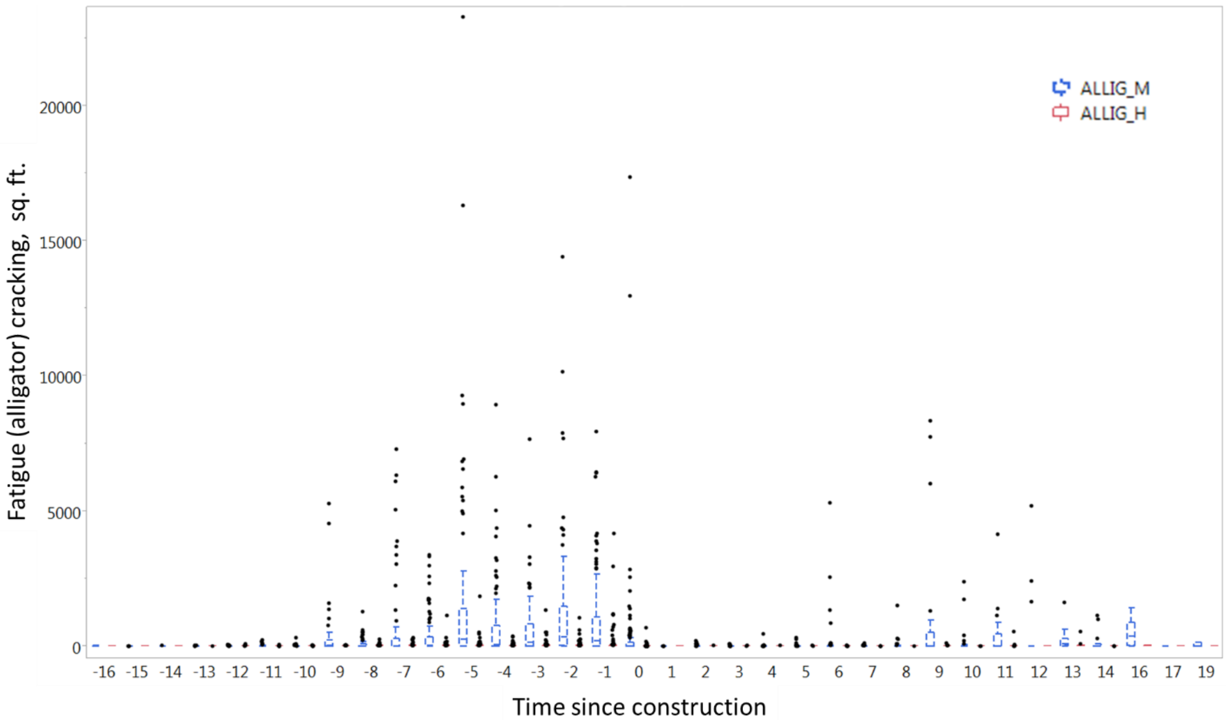


Figure 30. High and medium severity fatigue (alligator) cracking

The CIR rehabilitation shows a substantial decrease in the amount of fatigue cracking. There is a small amount of measureable medium-level severity cracking in year nine. There is no indication of an exponential trend in the data.

6.6 Rutting

Rutting is defined as a longitudinal surface depression in the wheel path. It may have associated transverse displacement. Rutting can be a safety issue because it may prevent adequate drainage of water in the wheel path. Rutting can occur when any of the pavement layers, including the subgrade, consolidate or lateral movement of the material occurs. Rutting can also be caused by plastic movement of the asphalt mix due to inadequate compaction during construction or hot weather (Huang 2004). The rutting depth for each pavement section was measured and reported in Figure 31.

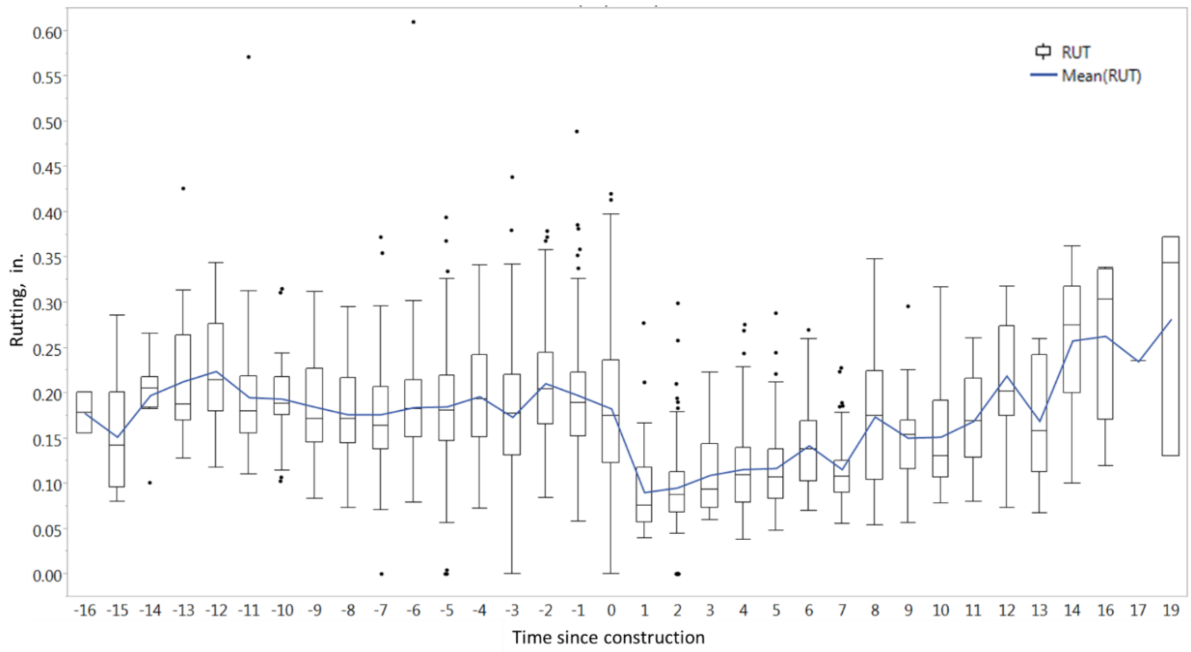


Figure 31. Measured rutting of pavement with time

The average rutting appeared to be constant prior to construction. At the time of rehabilitation the rutting depth decreased and steadily increased to the original rutting depth. The later years show a higher average rutting but the latter years are represented by fewer projects. Overall, it appears that CIR does not completely eliminate rutting problems however it does show improved rutting for approximately the first ten years. Data to predict more accurate trends for long-term performance will be available as long as the PMIS information continues to be collected. The data presented shows that the strength of the underlying layers are still important for good performance in a CIR pavement. For the best performance results, areas with poor subgrade should be identified and improved. A of couple sections show approximately a quarter inch of rutting right after construction. This may be due to construction problems, wet weather delaying the setting of the emulsion, or another condition/issue. A fast-setting emulsion may also help to mitigate areas where problems have been identified.

6.7 Patching

Patching is defined as a portion of the pavement surface, greater than 0.1 m^2, that has been removed and replaced or additional material applied to the pavement after original construction (Miller and Bellinger 2003). In the PMIS database, patching is measured in square feet per mile. Figure 32 shows the square feet of patching after the CIR rehabilitation, the square feet of patching required is markedly lower.

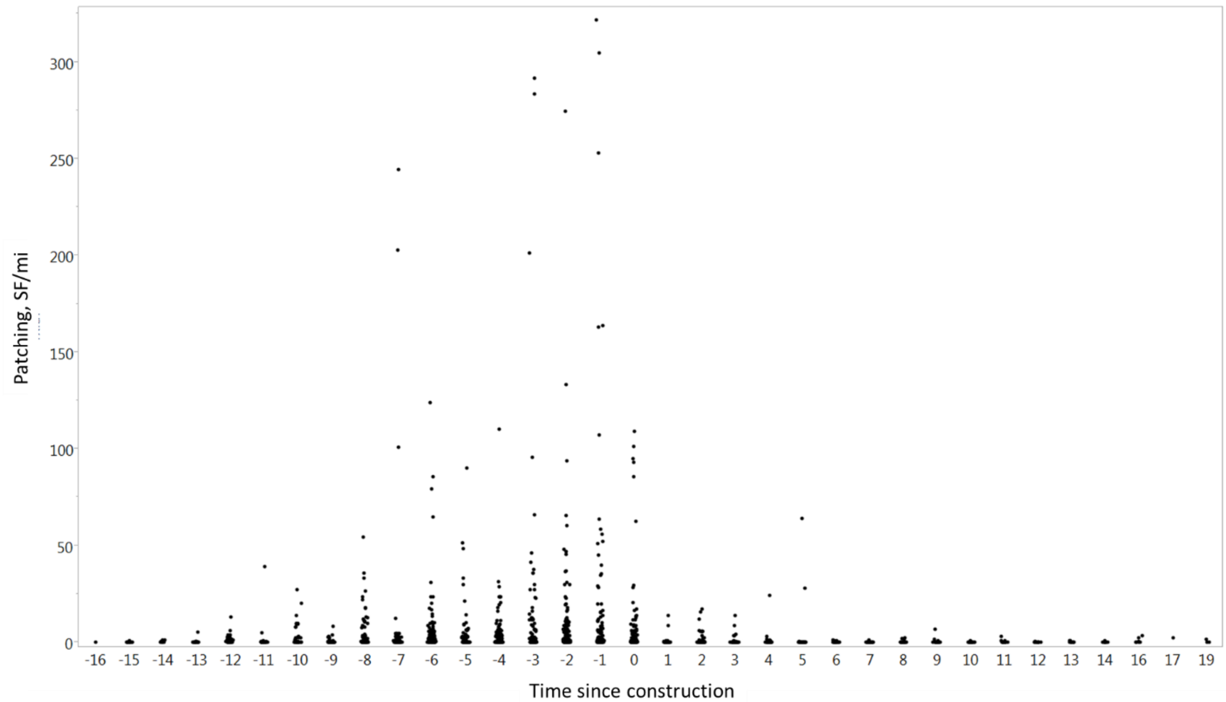


Figure 32. Square feet of patching versus time

The patching at time zero still has a higher average because these measurements were likely taken prior to rehabilitation during the same year of construction. The reduction in patching can be translated into an annual savings. Patching disrupts traffic, diverts personnel and resources while being a temporary solution. A future analysis that tracks the amount of patching for additional projects over a longer duration will help to more accurately quantify the savings associated with the reduced patching costs. The more accurate costs can be incorporated into a lifecycle cost analysis system for Iowa.

6.8 International Roughness Index

IRI summarizes the longitudinal surface profile in the wheel path and is computed from surface elevation data (Huang 2004). The higher values indicate higher roughness in the pavement. Table 1 in this report documents the IRI trigger value for various levels of roadways. The average IRI for each pavement is shown in Figure 33 and the number of projects measured for each year is indicated by the red line.

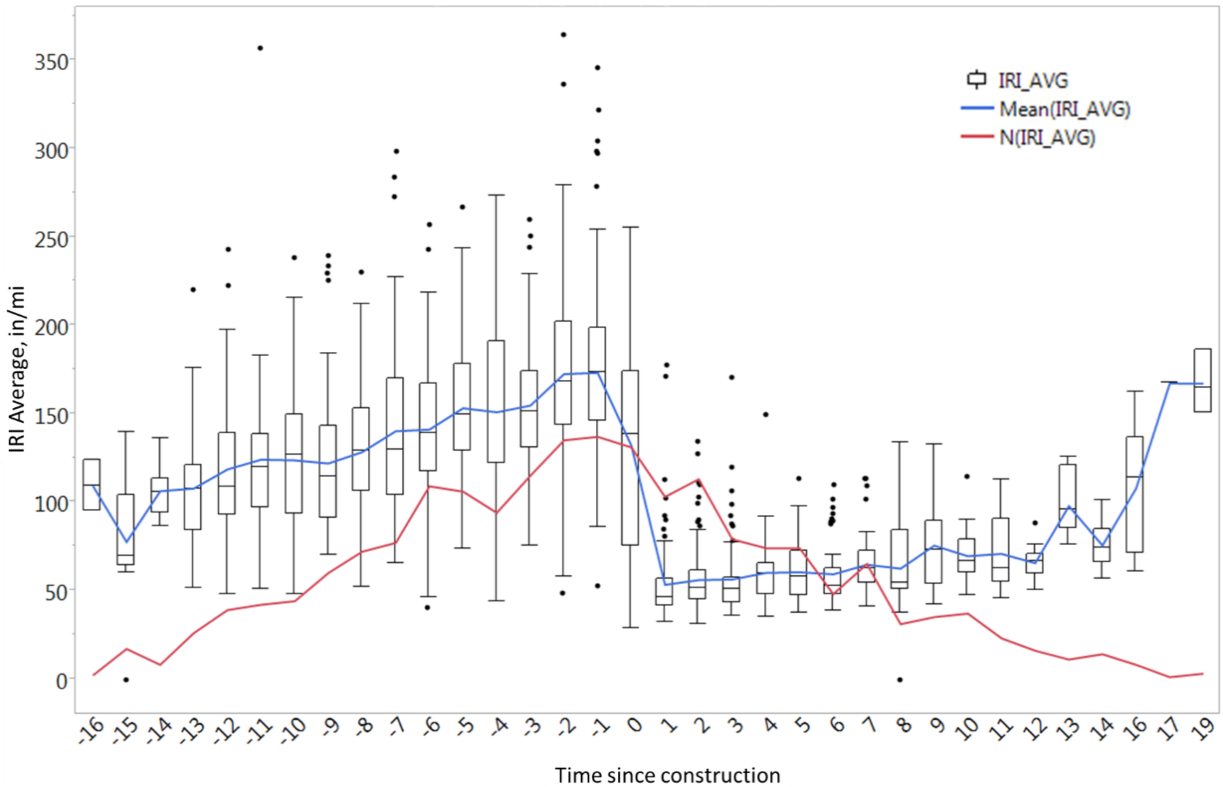


Figure 33. Average IRI with box plot and red line indicating the number of projects monitored

The IRI shows a steady increase in roughness until the time of rehabilitation. The IRI of the rehabilitated pavement shows a slow but steady increase over time. The IRI values from years 13 to 19 post-construction represent only a small percentage of rehabilitated pavements and these values should not be interpreted as an exponential increase in IRI. A factor that appears to influence the long-term IRI of the pavement is the thickness of the CIR layer. Figure 34 shows the IRI for each section categorized by thickness.

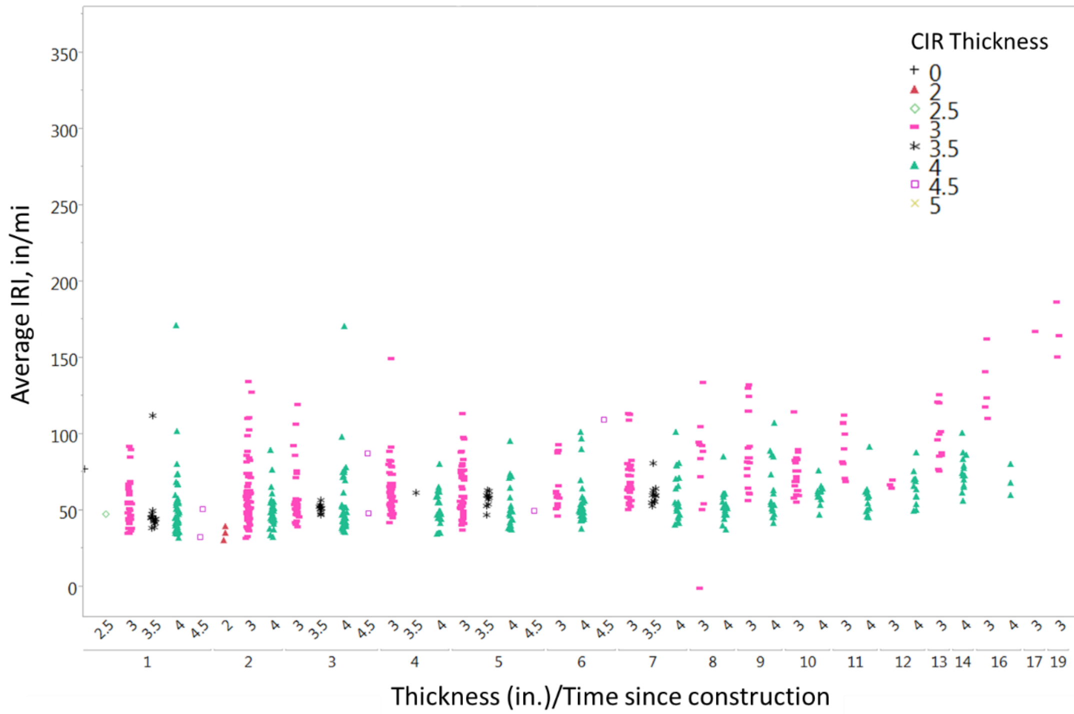


Figure 34. Average IRI by thickness and time in years since construction

The two most common CIR thicknesses are three inches, represented by pink dashes, and four inches, represented by green triangles. The averages are similar until approximately year seven when the average IRI for the four inch CIR tends to remain lower while the IRI for the three inch CIR trends upward. In time, more information will be available and two-inch CIR sections can be included in the analysis. On average, a CIR pavement with a four inch layer appears to have a reduced IRI over time compared to a CIR pavement with a three inch layer. This information can be used in a cost analysis to evaluate if the improved performance from the additional inch of CIR is worth the average improvement in IRI.

Since thickness appeared to be a significant factor in the IRI performance, a multiple regression model was developed using CIR thickness, interlayer thickness and surface thickness as a function of time to determine the IRI values. This model can be improved in the future as more data becomes available, especially for longer post-construction durations and low CIR thicknesses. The model was developed using a natural log transformation to ensure the rule of equal variance was met. The model results are shown in Figure 35 with the measured data points.

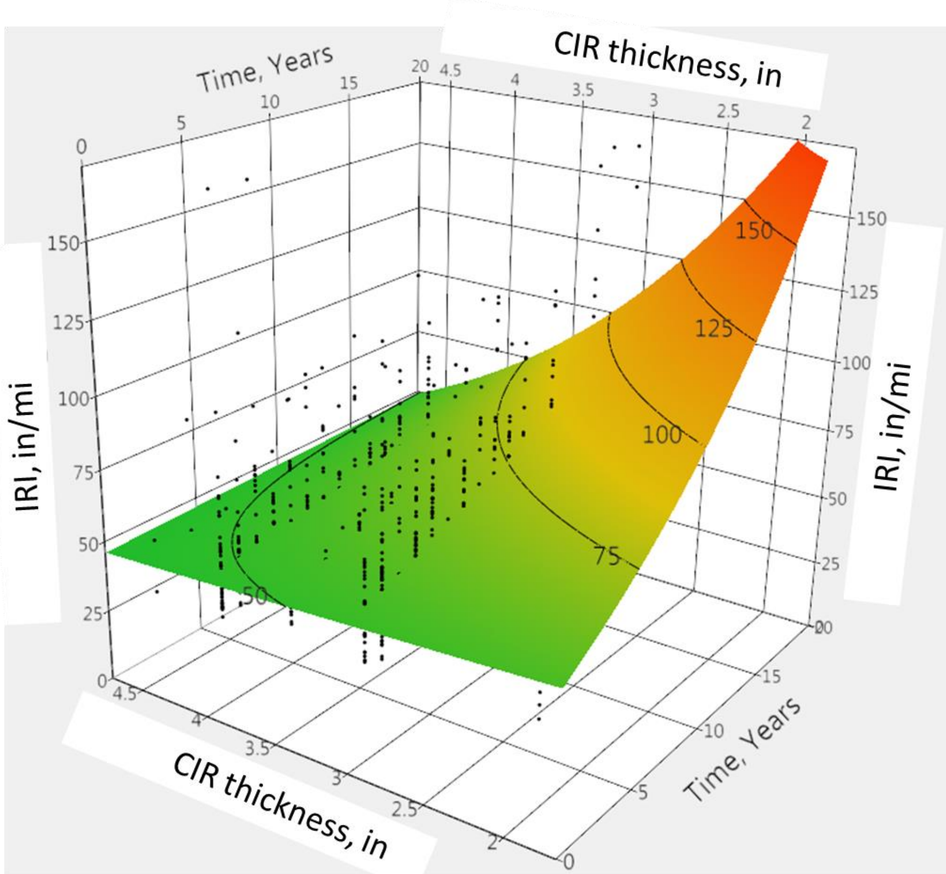


Figure 35. Predicted IRI as a function of CIR thickness and time shown with actual data points

To see if a model is over or under estimating, a plot of the residuals can be examined as shown in Figure 36.

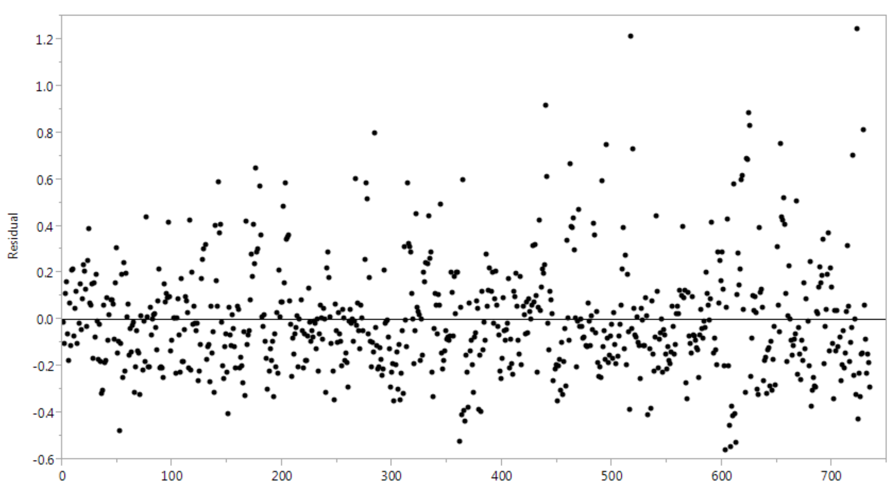


Figure 36. Plot of residuals by row number

The residuals show a high but consistent spread displaying that the model is a good representation of the overall average. The surface plot, Figure 35, also shows extrapolated values for lower thicknesses, areas of the model that are extrapolated beyond the data should not be used but do represent the potential performance curve of a pavement and as more data becomes available, the accuracy of these areas in the model will be improved. Overall, CIR rehabilitation significantly improves the pavement IRI and this increase shows excellent performance for the first 12 years. Once more projects have been monitored for a longer duration, a more complete analysis of pavement performance can be evaluated.

6.9 CIR Summary

The presented CIR performance data shows an overall improvement in pavement performance post-rehabilitation. This information can be used as guidance for assisting with making future decisions for pavement rehabilitation at the network level. Appropriate CIR pavement selection is still required for obtaining good performance. The CIR data was not analyzed using the statistical survivability techniques presented in Chapter 3 because not enough failures were observed at the recommended trigger levels. The survivability analysis will be useful in the future once additional failures have been observed. Instead of analyzing survivability, the transverse, longitudinal, fatigue, rutting, patching and IRI data was analyzed and discussed. The findings show that CIR significantly reduced transverse cracking. Many of the pavements are still being monitored and show excellent performance. Longitudinal cracking and rutting appears to reoccur more in the CIR compared to transverse cracking. Project selection, appropriate materials and adequate subgrade support will help to mitigate the occurrence of these types of distresses. Observed sections show the fatigue cracking and patching remains low post-rehabilitation. The overall pavement smoothness is measured by IRI. The IRI is improved after CIR rehabilitation. The sections were categorized by CIR thickness and the data showed the thicker layers remained smoother longer. A model was developed to capture this phenomenon. The overall model is preliminary due to the lack of data at the lower thicknesses and the low number of projects observed for projects past 11 years post-rehabilitation but a residual plot shows that it captures the overall average of the data fairly well.

CHAPTER 7 RECOMMENDATIONS AND CONCLUSIONS

In the first part of this research, network-level analysis, this study presented the research effort to develop a guideline for treatment selection, including mill and fill, SCR, HMA overlay, and rubblization, for reflective cracking control in composite pavement by survival analysis. The results of the evaluation showed the following:

- The Kaplan-Meier estimator clearly illustrated that pavement rubblization can significantly retard reflective cracking development in composite pavements compared to the other three methods. The mill and fill treatment also exhibited better performance than HMA overlay in terms of reflective cracking mitigation.
- The general trend of the hazard/failure function for reflective cracking followed a Lognormal distribution with an early-time increase followed by a constant or decreasing probability of failure. The corresponding survival function showed a sharp initial drop with a long tail in later service life.
- No significant differences in PCI were seen in the survival analysis for the four rehabilitation methods. The hazard function for the PCI, on the other hand, is best described by the Weibull distribution, which has an accelerated failure time pattern.
- The SCR method showed the lowest survival probability in terms of reflective cracking and IRI. Higher initial IRI values were found for the SCR and mill and fill treatments in the database. This finally led to lower IRI survival probabilities for the two treatments.
- According to the multivariate analysis performed in this study, traffic level was not a significant factor for reflective cracking. Higher trafficked roads even demonstrated a lower probability of reflective cracking failure.
- Increasing the new pavement thickness was effective in retarding the propagation of reflective cracking for all four treatments. The removed pavement thickness did not significantly affect the survival probability.
- The literature showed that subgrade soil properties can influence the use of rubblization in the field. However, this was not observed for the simple criteria considered in this report. Modifying the rubblization pattern to compensate for weaker subgrades was commonly performed by practitioners.

The second part of this research, project-level analysis, focused on the structural condition of existing treated composite pavements, including full rubblization, modified rubblization, rock interlayer, and crack and seat. The results of the evaluation show the following:

- SWM was a viable method for in situ material characterization of pavement systems. PCC modulus values from the SWM compared well with the FWD results on traditional composite pavement.
- The effect of SWM low strain amplitude was evident in the measurement of the modified rubblization layer. The SWM moduli were typically two to three times higher than the values predicted by the FWD.
- The SWM was used effectively to determine the moduli of thin rock interlayers, while the FWD had difficulty in measuring and back-calculating the thin layer moduli.

- For the four treatment methods, the crack and seat treatment had the highest moduli followed by the modified rubblization layer. The full rubblization layer and the rock interlayer give similar, but lower, moduli.
- Field performance data showed that the traditional composite pavement site had the highest amount of reflective cracking. A moderate amount of reflective cracking was observed for the full rubblization projects. Poor subgrade soil properties should be a consideration for whether to use rubblization or not.
- It is recommended to use the rock interlayer and modified rubblization methods in the field. However, more projects should be monitored to support this idea.
- CIR clearly indicated an overall improvement in performance for the first 12–14 years post-rehabilitation. Data are currently insufficient because the PMIS does not contain projects greater than 14 years old.

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APPENDIX A

Table 15. Summary of reflective/transverse cracking condition

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Mill and fill	IA 1	HSIPX-001-5(78)--3L-52	2000	10	0
Mill and fill	IA 1	STPN-1-6(21)--2J-57	1997	14	1
Mill and fill	IA 3	STPN-3-8(40)--2J-28	2006	6	1
Mill and fill	IA 4	STPN-004-6(23)--2J-32	2006	5	0
Mill and fill	IA 7	STPN-007-2(27)--2J-11	2007	5	1
Mill and fill	IA 9	STP-9-1(34)--2C-60	2001	13	1
Mill and fill	IA 9	STPN-009-4(38)--2J-32	2006	6	1
Mill and fill	IA 12	STP-12-1(16)--2C-97	1997	12	0
Mill and fill	IA 12	STPN-3-1(72)--2J-75	2004	8	1
Mill and fill	IA 14	STPN-14-6(37)--2J-38	2007	5	1
Mill and fill	IA 18	NHSN-18-2(82)--2R-21	2003	8	1
Mill and fill	IA 18	NHSN-018-9(83)--2R-22	2006	6	1
Mill and fill	IA 18	NHSN-18-2(82)--2R-21	2004	8	1
Mill and fill	US 18	NHSN-018-4(24)--2R-41	2006	6	1
Mill and fill	US 30	NHSX-30-3(35)--3H-37	2005	6	1
Mill and fill	US 30	NHSN-30-6(93)--2R-06	2003	6	0
Mill and fill	US 34	NHSX-34-9(91)--3H-44	2002	7	0
Mill and fill	US 34	NHSN-34-2(32)--2R-69	1998	11	0
Mill and fill	IA 48	STPN-48-2(41)--2J-69	2004	6	0
Mill and fill	IA 48	STPN-048-2(40)—2J-69	2004	8	1
Mill and fill	US 59	STP-59-4(28)--2C-83	2000	12	1
Mill and fill	US 61	NHSN-61-1(116)--2R-56	2005	5	0
Mill and fill	US 63	NHSN-063-8(65)--2R-19	2007	5	1
Mill and fill	US 67	NHSN-67-1(89)--2R-82	2001	8	0
Mill and fill	US 69	STP-69-7(20)--2C-99	1999	13	1
Mill and fill	US 71	MP-71-4(701)85--76-05	2002	6	0
Mill and fill	IA 92	STPN-92-9(112)--2J-58	2004	6	0
Mill and fill	IA 136	STPN-19-1(12)--2J-56	1997	13	1
Mill and fill	IA 144	STP-144-3(12)--2C-37	2004	8	1
Mill and fill	IA 149	STPN-149-1(63)--2J-54	2003	6	0
Mill and fill	IA 150	STPN-150-3(51)--2J-10	2005	7	1
Mill and fill	IA 151	STPN-151-1(18)--2J-40	2005	7	1
Mill and fill	IA 163	NHSN-163-1(60)--2R-77	2002	10	1
Mill and fill	US 169	STPN-169-8(51)--2J-55	2008	4	1
Mill and fill	US 218	STP-218-7(177)--2C-07	2002	8	0
Mill and fill	US 218	STPN-218-6(36)--2J-06	2001	7	0

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Mill and fill	US 218	STPN-218-9(116)--2J-66	2004	7	0
Mill and fill	IA 175	STPN-175-3(45)--2J-47	2005	7	1
Mill and fill	IA 175	STP-175-7(18)--2C-40	2004	8	1
Mill and fill	US 218	NHSN-218-2(41)--2R-44	1998	14	1
Mill and fill	US 218	STP-218-7(177)--2C-07	2001	10	1
Mill and fill	US 218	STPN-218-6(36)--2J-06	1998	11	0
SCR	IA 1	STPN-1-2(24)--2J-51	2005	7	1
SCR	IA 2	STPN-2-7(41)--2J-04	2008	4	1
SCR	IA 3	STP-3-4(36)--2C-99	1998	13	0
SCR	IA 3	STPN-3-5(52)--2J-12	1998	3	0
SCR	IA 3	STPN-003-3(65)--2J-35	2007	4	0
SCR	IA 3	STPN-13-2(41)--2J-28	2007	5	1
SCR	US 6	MP-6-4(701)54--76-15	2004	5	0
SCR	US 6	MP-6-4(701)54--76-15	2004	5	0
SCR	US 6	STPN-6-4(135)--2J-77	2003	8	0
SCR	IA 13	STPN-13-2(39)--2J-28	2007	4	1
SCR	IA 17	MP-17-1(706)20--76-08	2007	5	1
SCR	US 34	NHSN-34-6(71)--2R-59	2004	5	0
SCR	US 34	NHSN-34-6(71)--2R-59	2004	5	0
SCR	US 6	STPN-6-4(135)--2J-77	2003	4	0
SCR	US 30	NHSX-30-1(105)--3H-43	2005	11	1
SCR	US 30	NHSN-30-9(108)--2R-23	2003	2	0
SCR	US 30	NHSN-30-9(108)--2R-23	2003	3	0
SCR	US 34	NHSN-34-6(71)--2R-59	2005	6	0
SCR	US 34	NHSN-34-6(71)--2R-59	2005	5	0
SCR	US 34	MP-034-5(703)118--76-20	2005	7	1
SCR	IA 57	MP-57-2(1)8--76-38	2003	4	0
SCR	US 61	MP-61-5(703)55--76-29	2007	5	1
SCR	IA 64	STPN-64-2(51)--2J-49	2006	6	1
SCR	IA 64	STPN-65-2(54)--2J-49	2006	5	1
SCR	US 69	STPN-069-4(72)--2J-77	2007	5	1
SCR	IA 110	STPN-110-1(9)--2J-81	1998	14	1
SCR	IA 127	STPN-127-1(13)--2J-43	1999	14	1
SCR	IA 163	NHSX-163-1(60)--3H-77	2002	7	0
SCR	IA 202	MP-202-5(701)0--76-26	2005	6	0
SCR	US 218	NHSN-218-2(42)--2R-44	1999	10	0
SCR	IA 330	NHSN-330-2(50)--2R-64	2005	7	1
Overlay	IA 1	STP-1-1(23)--2C-89	2000	12	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Overlay	IA 1	STPN-1-2(24)--2J-51	2005	7	1
Overlay	IA 2	STP-2-9(16)--2C-89	2003	6	0
Overlay	IA 3	STPN-3-8(36)--2J-28	2004	5	0
Overlay	IA 3	STPN-3-8(40)--2J-28	2006	6	1
Overlay	IA 4	STP-4-5(27)--2C-74	2001	12	1
Overlay	IA 4	STPN-4-2(36)--2J-37	2006	6	1
Overlay	IA 5	STPN-5-1(37)--2J-04	2000	7	0
Overlay	US 6	STPN-6-4(135)--2J-77	2003	9	1
Overlay	US 6	STP-6-8(28)--2J-70	2001	11	1
Overlay	US 6	STP-6-8(29)--2S-70	2000	8	0
Overlay	IA 8	STPN-8-2(4)--2J-06	1999	13	1
Overlay	IA 9	STP-9-8(29)--2C-96	2001	11	1
Overlay	IA 10	STP-10-4(9)--2C-11	2004	8	1
Overlay	IA 12	STPN-12-1(21)--2J-97	2001	7	0
Overlay	IA 14	STP-14-3(35)--2C-63	2005	7	0
Overlay	US 20	NHSX-020-1(86)--3H-97	2003	8	1
Overlay	US 20	NHSX-20-4(42)--3H-40	2003	5	0
Overlay	IA 21	STP-21-4(25)--2C-06	2000	9	0
Overlay	IA 25	STP-25-5(10)--2C-37	2001	7	0
Overlay	US 30	NHSX-30-4(65)--3H-08	2004	7	1
Overlay	US 30	NHSN-30-5(151)--2R-85	1999	10	0
Overlay	US 30	NHSN-30-2(79)--2R-14	1999	14	1
Overlay	US 30	NHSX-30-3(35)--3H-37	2005	6	1
Overlay	US 34	NHSX-34-6(65)--3H-68	2003	6	0
Overlay	US 34	NHSN-34-5(17)--2R-20	2000	5	0
Overlay	US 34	NHSN-034-1(76)--2R-65	2006	6	1
Overlay	IA 44	STPN-44-2(41)--2C-83	2002	9	1
Overlay	US 52	STPN-52-2(68)--2J-31	2001	3	0
Overlay	US 52	STP-52-2(87)--2C-31	2001	4	0
Overlay	IA 56	STP-56-2(5)--2C-22	2004	4	0
Overlay	IA 57	STP-57-1(3)--2C-12	2002	10	1
Overlay	IA 59	STPN-59-8(22)--2J-71	2001	8	0
Overlay	US 61	NHSN-61-5(129)--2R-82	2005	7	1
Overlay	US 63	NHSX-63-6(65)--3H-07	2000	8	0
Overlay	US 65	STP-65-7(31)--2C-35	2001	10	0
Overlay	US 65	MP-65-2(705)218--76-98	2004	4	0
Overlay	US 69	STP-69-1(28)--2C-27	2002	7	0
Overlay	US 69	STP-69-1(28)--2C-27	2002	10	1
Overlay	US 69	STPN-69-8(18)--2J-41	1998	10	0
Overlay	US 71	NHSN-71-1(22)--2R-73	2000	5	0

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Overlay	I-80	IM-80-6(241)205--13-48	2005	7	1
Overlay	IA 86	STPN-86-1(2)--2J-30	1998	10	0
Overlay	IA 92	STP-92-6(35)--2C-63	2001	8	0
Overlay	IA 140	STP-140-1(5)--2C-97	2004	8	1
Overlay	IA 141	STP-141-4(25)--2C-14	2003	9	1
Overlay	IA 150	STPN-150-4(48)--2J-33	2000	5	0
Overlay	IA 163	NHSX-163-2(48)--2H-50	2000	12	1
Overlay	IA 330	NHSN-330-1(24)--2R-50	2006	4	0
Rubblization	IA 3	STPN-3-8(36)--2J-28	2004	5	0
Rubblization	IA 3	STPN-003-8(40)--2J-28	2004	8	1
Rubblization	IA 9	STPN-009-7(27)--2J-45	2006	6	1
Rubblization	IA 139	STP-139-0(10)--2C-96	2001	11	1
Rubblization	IA 139	STP-139-0(10)--2C-96	2001	11	1
Rubblization	IA 141	NHSX-141-7(22)--3H-77	2002	7	0
Rubblization	US 52	NHSX-52-5(30)--3H-96	2002	10	1
Rubblization	US 63	NHS-63-1(42)--19-26	1999	13	1
Rubblization	US 218	NHSN-218-2(41)-2R-44	1998	14	1
Rubblization	12 mile road	STP-S-C031(32)--5E-31	2001	10	1
Rubblization	F-33	STP-S-C082(29)--5E-82	2004	7	1
Rubblization	Y-68 (north)	STP-S-C082(30)--5E-82	2004	7	1
Rubblization	Y-68 (south)	STP-S-C082(30)--5E-82	2004	7	1
Rubblization	D-38	STP-S-C007(67)--5E-07	2002	9	1
Rubblization	V-43	STP-S-C007(69)--5E-07	2003	8	1
Rubblization	V-43	FM-C007(60)--55-07	2001	10	0
Rubblization	D-16	FM-C007(59)--55-07	2001	10	1
Rubblization	S-71	FM-C063(83)—55-63	2007	4	1
Rubblization	T-14	STP-S-CO50(69)--5E-50	2004	7	1
Rubblization	R-35	STP-SC020(50)--5E-20	2004	7	1
Rubblization	P-27	FM-C088(36)--55-88	2006	5	1
Rubblization	P-17	FM-C088(36)--55-88	2006	5	1
Rubblization	H-24	FM-C088(34)--55-88	2006	5	1
Rubblization	G-61 (west)	STP-S-C001(60)--5E-01	2004	7	1
Rubblization	G-61 (east)	FM-C001(64)--55-01	2004	7	0
Rubblization	S62	STP-S-50(44)--5E-50	1998	10	0
Rubblization	E50	STP-S-CO23(74)--5E-23	2006	5	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Rubblization	N72	FM-C001(59)--55-01	2004	7	1
Rubblization	L55	STP-S-65(40)--5E-65	2000	13	1
Rubblization	D16	FM-C007(59)--55-07	2002	11	1

Table 16. Summary of IRI information

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Mill and fill	IA 1	HSIPX-001-5(78)--3L-52	2000	12	1
Mill and fill	IA 1	STPN-1-6(21)--2J-57	1997	6	0
Mill and fill	IA 3	STPN-3-8(40)--2J-28	2006	6	1
Mill and fill	IA 4	STPN-004-6(23)--2J-32	2006	5	0
Mill and fill	IA 7	STPN-007-2(27)--2J-11	2007	4	0
Mill and fill	IA 9	STP-9-1(34)--2C-60	2001	13	1
Mill and fill	IA 9	STPN-009-4(38)--2J-32	2006	4	0
Mill and fill	IA 12	STP-12-1(16)--2C-97	1997	10	0
Mill and fill	IA 12	STPN-3-1(72)--2J-75	2004	8	1
Mill and fill	IA 14	STPN-14-6(37)--2J-38	2007	5	1
Mill and fill	IA 18	NHSN-18-2(82)--2R-21	2003	8	1
Mill and fill	IA 18	NHSN-018-9(83)--2R-22	2006	6	1
Mill and fill	IA 18	NHSN-18-2(82)--2R-21	2004	8	1
Mill and fill	US 18	NHSN-018-4(24)--2R-41	2006	6	1
Mill and fill	US 30	NHSX-30-3(35)--3H-37	2005	6	1
Mill and fill	US 30	NHSN-30-6(93)--2R-06	2003	9	1
Mill and fill	US 34	NHSX-34-9(91)--3H-44	2002	10	1
Mill and fill	US 34	NHSN-34-2(32)--2R-69	1998	12	1
Mill and fill	IA 48	STPN-48-2(41)--2J-69	2004	7	1
Mill and fill	IA 48	STPN-048-2(40)—2J-69	2004	8	1
Mill and fill	US 59	STP-59-4(28)--2C-83	2000	12	1
Mill and fill	US 61	NHSN-61-1(116)--2R-56	2005	7	0
Mill and fill	US 63	NHSN-063-8(65)--2R-19	2007	5	1
Mill and fill	US 67	NHSN-67-1(89)--2R-82	2001	11	1
Mill and fill	US 69	STP-69-7(20)--2C-99	1999	13	1
Mill and fill	US 71	MP-71-4(701)85--76-05	2002	3	0
Mill and fill	IA 92	STPN-92-9(112)--2J-58	2004	8	1
Mill and fill	IA 136	STPN-19-1(12)--2J-56	1997	8	0
Mill and fill	IA 144	STP-144-3(12)--2C-37	2004	8	1
Mill and fill	IA 149	STPN-149-1(63)--2J-54	2003	5	0
Mill and fill	IA 150	STPN-150-3(51)--2J-10	2005	7	1
Mill and fill	IA 151	STPN-151-1(18)--2J-40	2005	1	1
Mill and fill	IA 163	NHSN-163-1(60)--2R-77	2002	8	0
Mill and fill	US 169	STPN-169-8(51)--2J-55	2008	4	1
Mill and fill	US 218	STP-218-7(177)--2C-07	2002	3	0
Mill and fill	US 218	STPN-218-6(36)--2J-06	2001	13	1
Mill and fill	US 218	STPN-218-9(116)--2J-66	2004	8	1
Mill and fill	IA 175	STPN-175-3(45)--2J-47	2005	7	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Mill and fill	IA 175	STP-175-7(18)--2C-40	2004	8	1
Mill and fill	US 218	NHSN-218-2(41)--2R-44	1998	14	1
Mill and fill	US 218	STP-218-7(177)--2C-07	2001	3	0
Mill and fill	US 218	STPN-218-6(36)--2J-06	1998	14	1
SCR	IA 1	STPN-1-2(24)--2J-51	2005	7	1
SCR	IA 2	STPN-2-7(41)--2J-04	2008	4	1
SCR	IA 3	STP-3-4(36)--2C-99	1998	5	0
SCR	IA 3	STPN-3-5(52)--2J-12	1998	14	0
SCR	IA 3	STPN-003-3(65)--2J-35	2007	4	0
SCR	IA 3	STPN-13-2(41)--2J-28	2007	5	1
SCR	US 6	MP-6-4(701)54--76-15	2004	3	0
SCR	US 6	MP-6-4(701)54--76-15	2004	8	1
SCR	US 6	STPN-6-4(135)--2J-77	2003	9	1
SCR	IA 13	STPN-13-2(39)--2J-28	2007	4	1
SCR	IA 17	MP-17-1(706)20--76-08	2007	5	1
SCR	US 34	NHSN-34-6(71)--2R-59	2004	8	1
SCR	US 34	NHSN-34-6(71)--2R-59	2004	8	1
SCR	US 6	STPN-6-4(135)--2J-77	2003	8	1
SCR	US 30	NHSX-30-1(105)--3H-43	2005	11	1
SCR	US 30	NHSN-30-9(108)--2R-23	2003	2	0
SCR	US 30	NHSN-30-9(108)--2R-23	2003	5	0
SCR	US 34	NHSN-34-6(71)--2R-59	2005	8	1
SCR	US 34	NHSN-34-6(71)--2R-59	2005	8	1
SCR	US 34	MP-034-5(703)118--76-20	2005	7	1
SCR	IA 57	MP-57-2(1)8--76-38	2003	4	0
SCR	US 61	MP-61-5(703)55--76-29	2007	4	0
SCR	IA 64	STPN-64-2(51)--2J-49	2006	6	1
SCR	IA 64	STPN-65-2(54)--2J-49	2006	5	1
SCR	US 69	STPN-069-4(72)--2J-77	2007	5	1
SCR	IA 110	STPN-110-1(9)--2J-81	1998	14	1
SCR	IA 127	STPN-127-1(13)--2J-43	1999	10	0
SCR	IA 163	NHSX-163-1(60)--3H-77	2002	6	0
SCR	IA 202	MP-202-1(2)-2C-26	2005	14	1
SCR	US 218	NHSN-218-2(42)--2R-44	1999	14	1
SCR	IA 330	NHSN-330-2(50)--2R-64	2005	7	1
Overlay	IA 1	STP-1-1(23)--2C-89	2000	12	1
Overlay	IA 1	STPN-1-2(24)--2J-51	2005	7	1
Overlay	IA 2	STP-2-9(16)--2C-89	2003	8	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Overlay	IA 3	STPN-3-8(36)--2J-28	2004	7	1
Overlay	IA 3	STPN-3-8(40)--2J-28	2006	6	1
Overlay	IA 4	STP-4-5(27)--2C-74	2001	12	1
Overlay	IA 4	STPN-4-2(36)--2J-37	2006	6	1
Overlay	IA 5	STPN-5-1(37)--2J-04	2000	8	1
Overlay	IA 6	STPN-007-2(27)--2J-11	2007	4	1
Overlay	US 6	STPN-6-4(135)--2J-77	2003	9	1
Overlay	US 6	STP-6-8(28)--2J-70	2001	11	1
Overlay	US 6	STP-6-8(29)--2S-70	2000	11	1
Overlay	IA 8	STPN-8-2(4)--2J-06	1999	13	1
Overlay	IA 9	STP-9-8(29)--2C-96	2001	11	1
Overlay	IA 10	STP-10-4(9)--2C-11	2004	10	1
Overlay	IA 12	STPN-12-1(21)--2J-97	2001	7	1
Overlay	IA 14	STP-14-3(35)--2C-63	2005	11	1
Overlay	US 20	NHSX-020-1(86)--3H-97	2003	8	1
Overlay	US 20	NHSX-20-4(42)--3H-40	2003	8	1
Overlay	IA 21	STP-21-4(25)--2C-06	2000	11	1
Overlay	IA 25	STP-25-5(10)--2C-37	2001	10	1
Overlay	US 30	NHSX-30-4(65)--3H-08	2004	7	1
Overlay	US 30	NHSN-30-5(151)--2R-85	1999	12	0
Overlay	US 30	NHSN-30-2(79)--2R-14	1999	14	1
Overlay	US 30	NHSX-30-3(35)--3H-37	2005	6	1
Overlay	US 34	NHSX-34-6(65)--3H-68	2003	8	1
Overlay	US 34	NHSN-34-5(17)--2R-20	2000	10	0
Overlay	US 34	NHSN-034-1(76)--2R-65	2006	6	1
Overlay	IA 44	STPN-44-2(41)--2C-83	2002	9	1
Overlay	US 52	STPN-52-2(68)--2J-31	2001	5	0
Overlay	US 52	STP-52-2(87)--2C-31	2001	5	0
Overlay	IA 56	STP-56-2(5)--2C-22	2004	7	0
Overlay	IA 57	STP-57-1(3)--2C-12	2002	10	1
Overlay	IA 59	STPN-59-8(22)--2J-71	2001	11	1
Overlay	US 61	NHSN-61-5(129)--2R-82	2005	7	1
Overlay	US 63	NHSX-63-6(65)--3H-07	2000	11	0
Overlay	US 65	STP-65-7(31)--2C-35	2001	11	1
Overlay	US 65	MP-65-2(705)218--76-98	2004	8	1
Overlay	US 69	STP-69-1(28)--2C-27	2002	10	1
Overlay	US 69	STP-69-1(28)--2C-27	2002	10	1
Overlay	US 69	STPN-69-8(18)--2J-41	1998	14	1
Overlay	US 71	NHSN-71-1(22)--2R-73	2000	11	0
Overlay	I-80	IM-80-6(241)205--13-48	2005	7	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Overlay	IA 86	STPN-86-1(2)--2J-30	1998	14	1
Overlay	IA 92	STP-92-6(35)--2C-63	2001	11	1
Overlay	IA 140	STP-140-1(5)--2C-97	2004	8	1
Overlay	IA 141	STP-141-4(25)--2C-14	2003	9	1
Overlay	IA 150	STPN-150-4(48)--2J-33	2000	12	1
Overlay	IA 163	NHSX-163-2(48)--2H-50	2000	12	1
Overlay	IA 330	NHSN-330-1(24)--2R-50	2006	7	1
Rubblization	IA 3	STPN-3-8(36)--2J-28	2004	7	0
Rubblization	IA 3	STPN-003-8(40)--2J-28	2004	8	1
Rubblization	IA 9	STPN-009-7(27)--2J-45	2006	6	1
Rubblization	IA 139	STP-139-0(10)--2C-96	2001	11	0
Rubblization	IA 139	STP-139-0(10)--2C-96	2001	11	1
Rubblization	IA 141	NHSX-141-7(22)--3H-77	2002	10	1
Rubblization	US 52	NHSX-52-5(30)--3H-96	2002	10	1
Rubblization	US 63	NHS-63-1(42)--19-26	1999	13	1
Rubblization	US 218	NHSN-218-2(41)-2R-44	1998	14	1
Rubblization	12 mile road	STP-S-C031(32)--5E-31	2001	10	0
Rubblization	F-33	STP-S-C082(29)--5E-82	2004	7	1
Rubblization	Y-68	STP-S-C082(30)--5E-82	2004	7	1
Rubblization	Y-68	STP-S-C082(30)--5E-82	2004	7	1
Rubblization	D-38	STP-S-C007(67)--5E-07	2002	9	1
Rubblization	V-43	STP-S-C007(69)--5E-07	2003	8	1
Rubblization	V-43	FM-C007(60)--55-07	2001	10	1
Rubblization	D-16	FM-C007(59)--55-07	2001	10	1
Rubblization	S-71	FM-C063(83)—55-63	2007	4	1
Rubblization	T-14	STP-S-CO50(69)--5E-50	2004	7	1
Rubblization	R-35	STP-SC020(50)--5E-20	2004	7	1
Rubblization	P-27	FM-C088(36)--55-88	2006	5	1
Rubblization	P-17	FM-C088(36)--55-88	2006	5	1
Rubblization	H-24	FM-C088(34)--55-88	2006	7	1
Rubblization	G-61 (west)	STP-S-C001(60)--5E-01	2004	9	1
Rubblization	G-61 (east)	FM-C001(64)--55-01	2004	9	1
Rubblization	S62	STP-S-50(44)--5E-50	1998	10	1
Rubblization	E50	STP-S-CO23(74)--5E-23	2006	5	1
Rubblization	N72	FM-C001(59)--55-01	2004	7	1
Rubblization	L55	STP-S-65(40)--5E-65	2000	13	1
Rubblization	L55	STP-S-65(40)--5E-65	2000	13	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Rubblization	D16	FM-C007(59)--55-07	2002	11	1

Table 17. Summary of PCI information

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Mill and fill	IA 1	HSIPX-001-5(78)--3L-52	2000	12	1
Mill and fill	IA 1	STPN-1-6(21)--2J-57	1997	6	0
Mill and fill	IA 3	STPN-3-8(40)--2J-28	2006	6	1
Mill and fill	IA 4	STPN-004-6(23)--2J-32	2006	4	0
Mill and fill	IA 7	STPN-007-2(27)--2J-11	2007	4	1
Mill and fill	IA 9	STP-9-1(34)--2C-60	2001	13	1
Mill and fill	IA 9	STPN-009-4(38)--2J-32	2006	5	0
Mill and fill	IA 12	STP-12-1(16)--2C-97	1997	10	0
Mill and fill	IA 12	STPN-3-1(72)--2J-75	2004	8	1
Mill and fill	IA 14	STPN-14-6(37)--2J-38	2007	5	1
Mill and fill	IA 18	NHSN-18-2(82)--2R-21	2003	8	1
Mill and fill	IA 18	NHSN-018-9(83)--2R-22	2006	6	1
Mill and fill	IA 18	NHSN-18-2(82)--2R-21	2004	8	1
Mill and fill	US 18	NHSN-018-4(24)--2R-41	2006	6	1
Mill and fill	US 30	NHSX-30-3(35)--3H-37	2005	6	1
Mill and fill	US 30	NHSN-30-6(93)--2R-06	2003	9	1
Mill and fill	US 34	NHSX-34-9(91)--3H-44	2002	7	0
Mill and fill	US 34	NHSN-34-2(32)--2R-69	1998	12	1
Mill and fill	IA 48	STPN-48-2(41)--2J-69	2004	6	0
Mill and fill	IA 48	STPN-048-2(40)—2J-69	2004	8	1
Mill and fill	US 59	STP-59-4(28)--2C-83	2000	12	1
Mill and fill	US 61	NHSN-61-1(116)--2R-56	2005	8	1
Mill and fill	US 63	NHSN-063-8(65)--2R-19	2007	5	1
Mill and fill	US 67	NHSN-67-1(89)--2R-82	2001	11	1
Mill and fill	US 69	STP-69-7(20)--2C-99	1999	13	1
Mill and fill	US 71	MP-71-4(701)85--76-05	2002	5	0
Mill and fill	IA 92	STPN-92-9(112)--2J-58	2004	8	1
Mill and fill	IA 136	STPN-19-1(12)--2J-56	1997	9	0
Mill and fill	IA 144	STP-144-3(12)--2C-37	2004	8	1
Mill and fill	IA 149	STPN-149-1(63)--2J-54	2003	5	0
Mill and fill	IA 150	STPN-150-3(51)--2J-10	2005	7	1
Mill and fill	IA 151	STPN-151-1(18)--2J-40	2005	7	1
Mill and fill	IA 163	NHSN-163-1(60)--2R-77	2002	10	1
Mill and fill	US 169	STPN-169-8(51)--2J-55	2008	4	1
Mill and fill	US 218	STP-218-7(177)--2C-07	2002	6	0
Mill and fill	US 218	STPN-218-6(36)--2J-06	2001	11	0
Mill and fill	US 218	STPN-218-9(116)--2J-66	2004	8	1
Mill and fill	IA 175	STPN-175-3(45)--2J-47	2005	7	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Mill and fill	IA 175	STP-175-7(18)--2C-40	2004	8	1
Mill and fill	US 218	NHSN-218-2(41)--2R-44	1998	14	1
Mill and fill	US 218	STP-218-7(177)--2C-07	2001	6	0
Mill and fill	US 218	STPN-218-6(36)--2J-06	1998	10	0
SCR	IA 1	STPN-1-2(24)--2J-51	2005	7	1
SCR	IA 2	STPN-2-7(41)--2J-04	2008	4	1
SCR	IA 3	STP-3-4(36)--2C-99	1998	9	0
SCR	IA 3	STPN-3-5(52)--2J-12	1998	14	0
SCR	IA 3	STPN-003-3(65)--2J-35	2007	5	1
SCR	IA 3	STPN-13-2(41)--2J-28	2007	5	1
SCR	US 6	MP-6-4(701)54--76-15	2004	4	0
SCR	US 6	MP-6-4(701)54--76-15	2004	8	1
SCR	US 6	STPN-6-4(135)--2J-77	2003	9	1
SCR	IA 13	STPN-13-2(39)--2J-28	2007	4	1
SCR	IA 17	MP-17-1(706)20--76-08	2007	5	1
SCR	US 34	NHSN-34-6(71)--2R-59	2004	8	1
SCR	US 34	NHSN-34-6(71)--2R-59	2004	8	1
SCR	US 6	STPN-6-4(135)--2J-77	2003	8	1
SCR	US 30	NHSX-30-1(105)--3H-43	2005	11	1
SCR	US 30	NHSN-30-9(108)--2R-23	2003	4	0
SCR	US 30	NHSN-30-9(108)--2R-23	2003	4	0
SCR	US 34	NHSN-34-6(71)--2R-59	2005	8	1
SCR	US 34	NHSN-34-6(71)--2R-59	2005	8	1
SCR	US 34	MP-034-5(703)118--76-20	2005	7	1
SCR	IA 57	MP-57-2(1)8--76-38	2003	4	0
SCR	US 61	MP-61-5(703)55--76-29	2007	4	0
SCR	IA 64	STPN-64-2(51)--2J-49	2006	6	1
SCR	IA 64	STPN-65-2(54)--2J-49	2006	5	1
SCR	US 69	STPN-069-4(72)--2J-77	2007	5	1
SCR	IA 110	STPN-110-1(9)--2J-81	1998	13	1
SCR	IA 127	STPN-127-1(13)--2J-43	1999	10	0
SCR	IA 163	NHSX-163-1(60)--3H-77	2002	6	0
SCR	IA 202	MP-202-1(2)-2C-26	2005	10	1
SCR	US 218	NHSN-218-2(42)--2R-44	1999	11	1
SCR	IA 330	NHSN-330-2(50)--2R-64	2005	7	1
Overlay	IA 1	STP-1-1(23)--2C-89	2000	12	1
Overlay	IA 1	STPN-1-2(24)--2J-51	2005	7	1
Overlay	IA 2	STP-2-9(16)--2C-89	2003	8	1

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Overlay	IA 3	STPN-3-8(36)--2J-28	2004	7	0
Overlay	IA 3	STPN-3-8(40)--2J-28	2006	6	1
Overlay	IA 4	STP-4-5(27)--2C-74	2001	12	1
Overlay	IA 4	STPN-4-2(36)--2J-37	2006	6	1
Overlay	IA 5	STPN-5-1(37)--2J-04	2000	7	0
Overlay	IA 6	STPN-007-2(27)--2J-11	2007	4	1
Overlay	US 6	STPN-6-4(135)--2J-77	2003	9	1
Overlay	US 6	STP-6-8(28)--2J-70	2001	11	1
Overlay	US 6	STP-6-8(29)--2S-70	2000	11	1
Overlay	IA 8	STPN-8-2(4)--2J-06	1999	12	0
Overlay	IA 9	STP-9-8(29)--2C-96	2001	11	1
Overlay	IA 10	STP-10-4(9)--2C-11	2004	10	1
Overlay	IA 12	STPN-12-1(21)--2J-97	2001	7	1
Overlay	IA 14	STP-14-3(35)--2C-63	2005	10	0
Overlay	US 20	NHSX-020-1(86)--3H-97	2003	8	1
Overlay	US 20	NHSX-20-4(42)--3H-40	2003	8	1
Overlay	IA 21	STP-21-4(25)--2C-06	2000	10	0
Overlay	IA 25	STP-25-5(10)--2C-37	2001	10	1
Overlay	US 30	NHSX-30-4(65)--3H-08	2004	7	1
Overlay	US 30	NHSN-30-5(151)--2R-85	1999	10	0
Overlay	US 30	NHSN-30-2(79)--2R-14	1999	14	1
Overlay	US 30	NHSX-30-3(35)--3H-37	2005	6	1
Overlay	US 34	NHSX-34-6(65)--3H-68	2003	8	1
Overlay	US 34	NHSN-34-5(17)--2R-20	2000	7	0
Overlay	US 34	NHSN-034-1(76)--2R-65	2006	6	1
Overlay	IA 44	STPN-44-2(41)--2C-83	2002	9	1
Overlay	US 52	STPN-52-2(68)--2J-31	2001	9	0
Overlay	US 52	STP-52-2(87)--2C-31	2001	9	0
Overlay	IA 56	STP-56-2(5)--2C-22	2004	7	1
Overlay	IA 57	STP-57-1(3)--2C-12	2002	10	1
Overlay	IA 59	STPN-59-8(22)--2J-71	2001	11	1
Overlay	US 61	NHSN-61-5(129)--2R-82	2005	7	1
Overlay	US 63	NHSX-63-6(65)--3H-07	2000	11	0
Overlay	US 65	STP-65-7(31)--2C-35	2001	10	0
Overlay	US 65	MP-65-2(705)218--76-98	2004	8	1
Overlay	US 69	STP-69-1(28)--2C-27	2002	10	1
Overlay	US 69	STP-69-1(28)--2C-27	2002	10	1
Overlay	US 69	STPN-69-8(18)--2J-41	1998	13	0
Overlay	US 71	NHSN-71-1(22)--2R-73	2000	6	0
Overlay	I-80	IM-80-6(241)205--13-48	2005	4	0

Treatment	Route No.	Project No.	Year	Survival Time	Censored
Overlay	IA 86	STPN-86-1(2)--2J-30	1998	13	0
Overlay	IA 92	STP-92-6(35)--2C-63	2001	10	0
Overlay	IA 140	STP-140-1(5)--2C-97	2004	8	1
Overlay	IA 141	STP-141-4(25)--2C-14	2003	9	1
Overlay	IA 150	STPN-150-4(48)--2J-33	2000	12	1
Overlay	IA 163	NHSX-163-2(48)--2H-50	2000	12	1
Overlay	IA 330	NHSN-330-1(24)--2R-50	2006	7	1
Rubblization	IA 3	STPN-3-8(36)--2J-28	2004	7	0
Rubblization	IA 3	STPN-003-8(40)--2J-28	2004	8	1
Rubblization	IA 9	STPN-009-7(27)--2J-45	2006	6	1
Rubblization	IA 139	STP-139-0(10)--2C-96	2001	11	1
Rubblization	IA 139	STP-139-0(10)--2C-96	2001	11	1
Rubblization	IA 141	NHSX-141-7(22)--3H-77	2002	10	1
Rubblization	US 52	NHSX-52-5(30)--3H-96	2002	10	1
Rubblization	US 63	NHS-63-1(42)--19-26	1999	13	0
Rubblization	US 218	NHSN-218-2(41)-2R-44	1998	14	1
Rubblization	12 mile road	STP-S-C031(32)--5E-31	2001	10	0
Rubblization	F-33	STP-S-C082(29)--5E-82	2004	7	1
Rubblization	Y-68	STP-S-C082(30)--5E-82	2004	7	1
Rubblization	Y-68	STP-S-C082(30)--5E-82	2004	7	0
Rubblization	D-38	STP-S-C007(67)--5E-07	2002	9	1
Rubblization	V-43	STP-S-C007(69)--5E-07	2003	8	1
Rubblization	V-43	FM-C007(60)--55-07	2001	7	0
Rubblization	D-16	FM-C007(59)--55-07	2001	10	1
Rubblization	S-71	FM-C063(83)—55-63	2007	4	1
Rubblization	T-14	STP-S-CO50(69)--5E-50	2004	7	1
Rubblization	R-35	STP-SC020(50)--5E-20	2004	7	1
Rubblization	P-27	FM-C088(36)--55-88	2006	5	0
Rubblization	P-17	FM-C088(36)--55-88	2006	5	1
Rubblization	H-24	FM-C088(34)--55-88	2006	7	1
Rubblization	G-61 (west)	STP-S-C001(60)--5E-01	2004	9	1
Rubblization	G-61 (east)	FM-C001(64)--55-01	2004	9	0
Rubblization	S62	STP-S-50(44)--5E-50	1998	10	1
Rubblization	E50	STP-S-CO23(74)--5E-23	2006	5	1
Rubblization	N72	FM-C001(59)--55-01	2004	7	1
Rubblization	L55	STP-S-65(40)--5E-65	2000	13	1

Table 18. Summary of pavement structural and traffic information

Treatment	Project No.	Initial IRI (mi)	Overlay Thickness (in.)	Removal Thickness (in.)	ADT
Mill and fill	HSIPX-001-5(78)--3L-52	94.4	3	0.5	9200
Mill and fill	STPN-1-6(21)--2J-57	121.7	4	3	5200
Mill and fill	STPN-3-8(40)--2J-28	53.9	3	3	2830
Mill and fill	STPN-004-6(23)--2J-32	54.5	3	3	2430
Mill and fill	STPN-007-2(27)--2J-11	44.4	3	3	7700
Mill and fill	STP-9-1(34)--2C-60	158.4	5	2	1240
Mill and fill	STPN-009-4(38)--2J-32	103.3	3	3	7800
Mill and fill	STP-12-1(16)--2C-97	61.5	4	3	7000
Mill and fill	STPN-3-1(72)--2J-75	42.5	5	1.5	1540
Mill and fill	STPN-14-6(37)--2J-38	64.0	7	3.5	2350
Mill and fill	NHSN-18-2(82)--2R-21	45.0	3	2	10500
Mill and fill	NHSN-018-9(83)--2R-22	48.8	5	3	3510
Mill and fill	NHSN-18-2(82)--2R-21	50.1	3	2	10200
Mill and fill	NHSN-018-4(24)--2R-41	43.7	3	1	5700
Mill and fill	NHSX-30-3(35)--3H-37	44.4	3	2	3980
Mill and fill	NHSN-30-6(93)--2R-06	55.1	3.5	1.5	8000
Mill and fill	NHSX-34-9(91)--3H-44	52.0	4	1	7500
Mill and fill	NHSN-34-2(32)--2R-69	81.7	3	2	3620
Mill and fill	STPN-48-2(41)--2J-69	74.1	4.5	1.5	6400
Mill and fill	STPN-048-2(40)—2J-69	41.2	5	4	1640
Mill and fill	STP-59-4(28)--2C-83	71.0	3.5	1	2720
Mill and fill	NHSN-61-1(116)--2R-56	32.3	3.5	2	10800
Mill and fill	NHSN-063-8(65)--2R-19	103.9	3	0.5	3670
Mill and fill	NHSN-67-1(89)--2R-82	41.2	4	2	16700
Mill and fill	STP-69-7(20)--2C-99	133.1	4	1.5	2630
Mill and fill	MP-71-4(701)85--76-05	46.3	2	2	4980
Mill and fill	STPN-92-9(112)--2J-58	95.0	2	1.5	3160
Mill and fill	STPN-19-1(12)--2J-56	57.0	4	4	4450
Mill and fill	STP-144-3(12)--2C-37	101.4	6	3	670
Mill and fill	STPN-149-1(63)--2J-54	41.2	3.5	2	3620
Mill and fill	STPN-150-3(51)--2J-10	43.7	4	0.5	4860
Mill and fill	STPN-151-1(18)--2J-40	85.5	4	0.5	5600
Mill and fill	NHSN-163-1(60)--2R-77	152.1	3	3	11200
Mill and fill	STPN-169-8(51)--2J-55	120.4	4	0.5	3530
Mill and fill	STP-218-7(177)--2C-07	58.3	3.5	1.5	2220
Mill and fill	STPN-218-6(36)--2J-06	33.6	3	1.5	6200
Mill and fill	STPN-218-9(116)--2J-66	65.9	3.5	0.5	2850
Mill and fill	STPN-175-3(45)--2J-47	45.6	3.5	3	850

Treatment	Project No.	Initial IRI (mi)	Overlay Thickness (in.)	Removal Thickness (in.)	ADT
Mill and fill	STP-175-7(18)--2C-40	62.7	4	3	1790
Mill and fill	NHSN-218-2(41)--2R-44	120.4	8	1	6200
Mill and fill	STP-218-7(177)--2C-07	63.4	3.5	1.5	2220
Mill and fill	STPN-218-6(36)--2J-06	72.2	3	1.5	4270
SCR	STPN-1-2(24)--2J-51	57.7	3	1.5	5000
SCR	STPN-2-7(41)--2J-04	50.7	4	0.5	2900
SCR	STP-3-4(36)--2C-99	101.4	8	2	5700
SCR	STPN-3-5(52)--2J-12	69.7	2	1	4380
SCR	STPN-003-3(65)--2J-35	114.1	3	1.5	5700
SCR	STPN-13-2(41)--2J-28	40.6	6.5	3	2410
SCR	MP-6-4(701)54--76-15	110.3	3	3	625
SCR	MP-6-4(701)54--76-15	60.9	4.5	4	614
SCR	STPN-6-4(135)--2J-77	50.7	5	1.5	946
SCR	STPN-13-2(39)--2J-28	37.4	4	0.5	4030
SCR	MP-17-1(706)20--76-08	79.9	3	1.5	2990
SCR	NHSN-34-6(71)--2R-59	62.1	4	1	3060
SCR	NHSN-34-6(71)--2R-59	63.4	4	1	2920
SCR	STPN-6-4(135)--2J-77	72.9	3.5	2	8300
SCR	NHSX-30-1(105)--3H-43	64.7	5	1	4740
SCR	NHSN-30-9(108)--2R-23	121.1	1.5	1.5	6400
SCR	NHSN-30-9(108)--2R-23	133.1	1.5	1.5	6400
SCR	NHSN-34-6(71)--2R-59	55.8	4	1	4200
SCR	NHSN-34-6(71)--2R-59	56.4	4	1	3700
SCR	MP-034-5(703)118--76-20	84.3	2	1.5	3700
SCR	MP-57-2(1)8--76-38	57.7	1	0.5	2350
SCR	MP-61-5(703)55--76-29	107.8	3.5	2	6200
SCR	STPN-64-2(51)--2J-49	47.6	3.5	1.5	2810
SCR	STPN-65-2(54)--2J-49	50.1	6.5	3	1750
SCR	STPN-069-4(72)--2J-77	190.2	3	2	32700
SCR	STPN-110-1(9)--2J-81	39.9	6	3	1200
SCR	STPN-127-1(13)--2J-43	78.6	3	1	1720
SCR	NHSX-163-1(60)--3H-77	68.5	3	3	23100
SCR	MP-202-1(2)-2C-26	58	3	2	N/A
SCR	NHSN-218-2(42)--2R-44	71.0	4.5	1.5	8800
SCR	NHSN-330-2(50)--2R-64	49.5	4.5	1.5	3920

Treatment	Project No.	Initial IRI (mi)	Overlay Thickness (in.)	Removal Thickness (in.)	ADT
		Initial IRI	**Overlay thickness**	**Pre-condition**	**ADT**
Overlay	STP-1-1(23)--2C-89	69.1	4	22	2300
Overlay	STPN-1-2(24)--2J-51	76.7	3	33	4110
Overlay	STP-2-9(16)--2C-89	63.4	3.5	44	1490
Overlay	STPN-3-8(36)--2J-28	60.9	3.5	5	940
Overlay	STPN-3-8(40)--2J-28	63.4	7	47	2380
Overlay	STP-4-5(27)--2C-74	41.2	2	60	1950
Overlay	STPN-4-2(36)--2J-37	76.1	3	29	1260
Overlay	STPN-5-1(37)--2J-04	51.4	3.5	67	3040
Overlay	STPN-007-2(27)--2J-11	72.3	3	22	7600
Overlay	STPN-6-4(135)--2J-77	57.1	3.5	39	9600
Overlay	STP-6-8(28)--2J-70	50.7	3.5	22	2050
Overlay	STP-6-8(29)--2S-70	57.1	4	35	3432
Overlay	STPN-8-2(4)--2J-06	41.8	3	20	990
Overlay	STP-9-8(29)--2C-96	57.1	4	63	3230
Overlay	STP-10-4(9)--2C-11	47.6	3	35	1810
Overlay	STPN-12-1(21)--2J-97	50.1	4	43	3650
Overlay	STP-14-3(35)--2C-63	58.3	4	39	3970
Overlay	NHSX-020-1(86)--3H-97	79.9	4	39	20800
Overlay	NHSX-20-4(42)--3H-40	56.4	3.5	54	7200
Overlay	STP-21-4(25)--2C-06	51.4	5	51	1490
Overlay	STP-25-5(10)--2C-37	57.7	4	39	890
Overlay	NHSX-30-4(65)--3H-08	64.7	4.5	38	10400
Overlay	NHSN-30-5(151)--2R-85	67.2	3.5	52	24900
Overlay	NHSN-30-2(79)--2R-14	37.4	5	55	5300
Overlay	NHSX-30-3(35)--3H-37	56.4	3.5	29	3960
Overlay	NHSX-34-6(65)--3H-68	44.4	3.5	57	2650
Overlay	NHSN-34-5(17)--2R-20	91.9	3.5	36	10200
Overlay	NHSN-034-1(76)--2R-65	107	5	33	9500
Overlay	STPN-44-2(41)--2C-83	50.1	4	11	1380
Overlay	STPN-52-2(68)--2J-31	122	3	63	3030
Overlay	STP-52-2(87)--2C-31	64.7	2.5	49	6000
Overlay	STP-56-2(5)--2C-22	97.6	3	21	1980
Overlay	STP-57-1(3)--2C-12	62.8	4	68	2460
Overlay	STPN-59-8(22)--2J-71	50.7	3	22	1010
Overlay	NHSN-61-5(129)--2R-82	69.1	4.5	44	25500
Overlay	NHSX-63-6(65)--3H-07	65.3	3.5	59	5900
Overlay	STP-65-7(31)--2C-35	61.5	3.5	44	2390

Treatment	Project No.	Initial IRI (mi)	Overlay Thickness (in.)	Removal Thickness (in.)	ADT
Overlay	MP-65-2(705)218--76-98	52	3	46	1980
Overlay	STP-69-1(28)--2C-27	53.3	3.5	18	1490
Overlay	STP-69-1(28)--2C-27	58.3	3.5	35	670
Overlay	STPN-69-8(18)--2J-41	54.5	4	58	5800
Overlay	NHSN-71-1(22)--2R-73	65.9	3.5	59	1900
Overlay	IM-80-6(241)205--13-48	43.7	3	50	27400
Overlay	STPN-86-1(2)--2J-30	60.2	3	57	3780
Overlay	STP-92-6(35)--2C-63	57.7	3.5	41	2770
Overlay	STP-140-1(5)--2C-97	51.4	4	19	1600
Overlay	STP-141-4(25)--2C-14	64.7	3.5	38	2150
Overlay	STPN-150-4(48)--2J-33	45	4	62	2900
Overlay	NHSX-163-2(48)--2H-50	46.9	5	70	8400
Overlay	NHSN-330-1(24)--2R-50	39.3	3	39	5900
		County	**Initial IRI (mi)**	**PCC Thickness (in.)**	**Soil Condition**
Rubblization	STPN-3-8(36)--2J-28	Delaware	60.8	9	Low
Rubblization	STPN-003-8(40)--2J-28	Delaware	35.5	10	High
Rubblization	STPN-009-7(27)--2J-45	New Hampton	32.3	8	Low
Rubblization	STP-139-0(10)--2C-96	Winneshiek	58.3	7	Low
Rubblization	STP-139-0(10)--2C-96	Winneshiek	43.7	7	Low
Rubblization	NHSX-141-7(22)--3H-77	Polk	67	10	Low
Rubblization	NHSX-52-5(30)--3H-96	Appanoose	45	8	Low
Rubblization	NHS-63-1(42)--19-26	Davis	45	8.5	Low
Rubblization	NHSN-218-2(41)-2R-44	Henry	54.5	9	Low
Rubblization	STP-S-C031(32)--5E-31	Dubuque	92	8.5	Low
Rubblization	STP-S-C082(29)--5E-82	Scott	38	6	High
Rubblization	STP-S-C082(30)--5E-82	Scott	44.5	6	Low
Rubblization	STP-S-C082(30)--5E-82	Scott	44.3	6	Low
Rubblization	STP-S-C007(67)--5E-07	Black Hawk	88.7	6	Low
Rubblization	STP-S-C007(69)--5E-07	Black Hawk	79.8	6	Low
Rubblization	FM-C007(60)--55-07	Black Hawk	101.2	7	High
Rubblization	FM-C007(59)--55-07	Black Hawk	101.2	7	High
Rubblization	FM-C063(83)—55-63	Marion	56	7	Low
Rubblization	STP-S-CO50(69)--5E-50	Jasper	46.6	6	Low
Rubblization	STP-SC020(50)--5E-20	Clarke	N/A	6	Low
Rubblization	FM-C088(36)--55-88	Union	N/A	6	High
Rubblization	FM-C088(36)--55-88	Union	N/A	7	High
Rubblization	FM-C088(34)--55-88	Union	N/A	7	High

Treatment	Project No.	Initial IRI (mi)	Overlay Thickness (in.)	Removal Thickness (in.)	ADT
Rubblization	STP-S-C001(60)--5E-01	Adair	N/A	6	Low
Rubblization	FM-C001(64)--55-01	Adair	N/A	6	Low
Rubblization	STP-S-50(44)--5E-50	Jasper	68.4	6	Low
Rubblization	STP-S-CO23(74)--5E-23	Clinton	60	6	Low
Rubblization	FM-C001(59)--55-01	Adair	N/A	6	High
Rubblization	STP-S-65(40)--5E-65	Mills	N/A	6	Low

APPENDIX B

Table 19. P29 (South) FWD Station 1

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable
1	959,136	0.3	1	6.0	Yes
2	81392	0.35	1	1.0	Yes
3	669.848	0.25	1	6.0	Yes
4	7281	0.4	1	95.0	Yes
5	60,000	0.35	1	0.0	No

Air Temp (F): 46.5

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	9.54	8.79	8.17	7.32	6.48	4.96	3.80
Meas Defl, mil	9.76	8.66	8.04	7.22	6.46	5.08	3.91

Table 20. P29 (South) FWD Station 2

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable
1	1129,314	0.3	1	6.0	Yes
2	99523	0.35	1	1.0	Yes
3	700,480	0.25	1	6.0	Yes
4	5988	0.4	1	95.0	Yes
5	60,000	0.35	1	0.0	No

Air Temp (F): 48

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	9.48	8.94	8.45	7.65	6.86	5.36	4.18
Meas Defl, mil	9.76	8.82	8.27	7.52	6.81	5.48	4.31

Table 21. P29 (South) FWD Station 3

Layer	Modulus (PSI)	Poisson’s	Interface	Thickness (in)	Changeable
1	826,049	0.3	1	6.0	Yes
2	86,156	0.35	1	1.0	Yes
3	454.283	0.25	1	6.0	Yes
4	5.780	0.4	1	95.0	Yes
5	60,000	0.35	1	0.0	No

Air Temp (F): 49

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	11.30	10.52	9.82	8.77	7.74	5.88	4.46
Meas Defl, mil	11.59	10.34	9.62	8.65	7.73	6.06	4.62

Table 22. P29 (South) FWD Station 4

Layer	Modulus (PSI)	Poisson’s	Interface	Thickness (in)	Changeable
1	1201,792	0.3	1	6.0	Yes
2	159,637	0.35	1	1.0	Yes
3	1219,099	0.25	1	6.0	Yes
4	6,898	0.4	1	95.0	Yes
5	60,000	0.35	1	0.0	No

Air Temp (F): 49

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	7..67	7.13	6.73	6.18	5.64	4.54	3.60
Meas Defl, mil	7.81	7.06	6.66	6.12	5.59	4.59	3.68

Table 23. P29 (North) FWD Station 1

Layer	Modulus (PSI)		Poisson's		Interface	Thickness (in)	Changeable
1	1134,781		0.3		1	6.0	Yes
2	100,911		0.35		1	1.0	Yes
3	902,356		0.25		1	6.0	Yes
4	6,788		0.4		1	95.0	Yes
5	60,000		0.35		1	0.0	No
Air Temp (F): 51							
Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	8.96	8.06	7.54	6.85	6.21	5.00	3.95
Meas Defl, mil	8.72	8.18	7.70	6.95	6.25	4.90	3.84

Table 24. P29 (North) FWD Station 2

Layer	Modulus (PSI)		Poisson's		Interface	Thickness (in)	Changeable
1	867,604		0.3		1	6.0	Yes
2	94,204		0.35		1	1.0	Yes
3	764,456		0.25		1	6.0	Yes
4	9,936		0.4		1	95.0	Yes
5	60,000		0.35		1	0.0	No
Air Temp (F): 40							
Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	7.93	7.37	6.82	6.03	5.28	3.95	2.88
Meas Defl, mil	8.27	7.18	6.64	5.89	5.23	4.04	3.06

Table 25. P29 (North) FWD Station 3

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable
1	1032,565	0.3	1	6.0	Yes
2	115,889	0.35	1	1.0	Yes
3	1094,250	0.25	1	6.0	Yes
4	9,801	0.4	1	95.0	Yes
5	60,000	0.35	1	0.0	No

Air Temp (F): 42.5

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	6.91	6.51	6.06	5.42	4.80	3.68	2.79
Meas Defl, mil	7.20	6.34	5.89	5.31	4.77	3.78	2.94

Table 26. P29 (North) FWD Station 4

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable
1	885,371	0.3	1	6.0	Yes
2	68,627	0.35	1	1.0	Yes
3	652,331	0.25	1	6.0	Yes
4	10,756	0.4	1	95.0	Yes
5	60,000	0.35	1	0.0	No

Air Temp (F): 45

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	7.86	7.20	6.57	5.72	4.96	3.68	2.66
Meas Defl, mil	8.13	7.02	6.41	5.64	4.95	3.76	2.81

Table 27. D43 FWD Station 1

Layer	Modulus (PSI)	Poisson’s	Interface	Thickness (in)	Changeable
1	1996,369	0.3	1	6.0	Yes
2	5774,250	0.2	1	8.0	Yes
3	15,240	0.4	1	90.0	Yes
4	60,000	0.35	1	0.0	No

Air Temp (F): 51

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	4.02	3.83	3.75	3.60	3.40	2.94	2.49
Meas Defl, mil	4.05	3.78	3.67	3.54	3.37	3.02	2.66

Table 28. D43 FWD Station 2

Layer	Modulus (PSI)	Poisson’s	Interface	Thickness (in)	Changeable
1	3538,301	0.3	1	6.0	Yes
2	5715,350	0.2	1	8.0	Yes
3	10,980	0.4	1	90.0	Yes
4	60,000	0.35	1	0.0	No

Air Temp (F): 50

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	3.84	3.59	3.52	3.39	3.21	2.84	2.42
Meas Defl, mil	3.89	3.59	3.48	3.34	3.18	2.84	2.50

Table 29. P43 FWD Station 3

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable	
1	947,329	0.3	1	6.0	Yes	
2	1990,820	0.2	1	8.0	Yes	
3	21,425	0.4	1	90.0	Yes	
4	60,000	0.35	1	0.0	No	
Air Temp (F): 53						
Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	4.95	4.58	4.34	4.00	3.69	3.07	2.54
Meas Defl, mil	5.09	4.49	4.27	3.99	3.71	3.15	2.64

Table 30. P43 FWD Station 4

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable		
1	2189,455	0.3	1	6.0	Yes		
2	6479,482	0.2	1	8.0	Yes		
3	16,467	0.4	1	90.0	Yes		
4	60,000	0.35	1	0.0	No		
Air Temp (F): 50							
Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	3.59	3.39	3.33	3.22	3.08	2.72	2.37
Meas Defl, mil	3.63	3.40	3.31	3.19	3.04	2.73	2.42

Table 31. P59 FWD Station 1

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable
1	609,877	0.3	1	4.0	Yes
2	59,338	0.35	1	1.0	Yes
3	447,227	0.25	1	6.0	Yes
4	6,967	0.4	1	90.0	Yes
4	60,000	0.35	1	0.0	No

Air Temp (F): 48

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	15.03	13.40	12.18	10.30	8.69	6.13	4.37
Meas Defl, mil	15.40	13.10	11.84	10.24	8.81	6.38	4.50

Table 32. P59 FWD Station 2

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable
1	333,482	0.3	1	4.0	Yes
2	51,353	0.35	1	1.0	Yes
3	356,039	0.25	1	6.0	Yes
4	6,302	0.4	1	90.0	Yes
4	60,000	0.35	1	0.0	No

Air Temp (F): 49

Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	17.78	15.44	14.04	11.73	9.71	6.50	4.38
Meas Defl, mil	18.22	15.10	13.53	11.59	9.84	6.90	4.69

Table 33. P59 FWD Station 3

Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable		
1	476,914	0.3	1	4.0	Yes		
2	38,545	0.35	1	1.0	Yes		
3	447,427	0.25	1	6.0	Yes		
4	7,126	0.4	1	90.0	Yes		
4	60,000	0.35	1	0.0	No		
Air Temp (F): 48							
Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	16.12	13.97	12.70	10.58	8.74	5.88	4.07
Meas Defl, mil	16.40	13.67	12.20	10.42	8.86	6.28	4.34

Table 34. P59 FWD Station 4

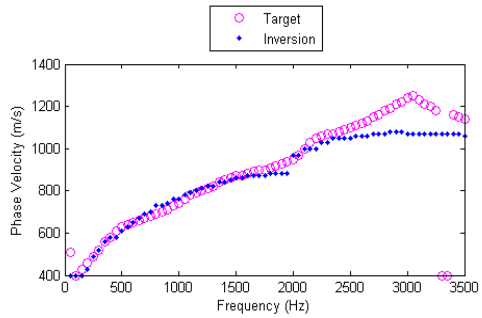
Layer	Modulus (PSI)	Poisson's	Interface	Thickness (in)	Changeable		
1	553,620	0.3	1	4.0	Yes		
2	76,227	0.35	1	1.0	Yes		
3	376,754	0.25	1	6.0	Yes		
4	7,159	0.4	1	90.0	Yes		
4	60,000	0.35	1	0.0	No		
Air Temp (F): 49							
Sensor	1	2	3	4	5	6	7
Offset	0.0	8.0	12.0	18.0	24.0	36.0	48.0
Calc Defl, mil	15.25	13.25	12.12	10.28	8.70	6.11	4.33
Meas Defl, mil	15.46	13.14	11.88	10.24	8.77	6.28	4.36

APPENDIX C

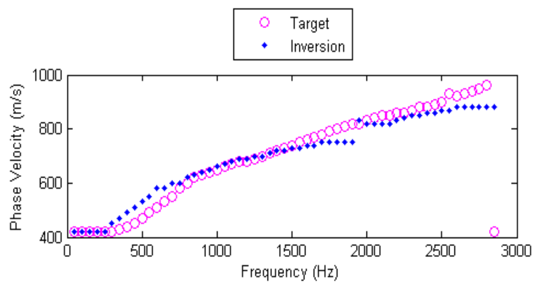
Table 35. Assumed values in SWM back-calculation

	Density (kg/m3)	Poisson Ratio
HMA	Measured (2240 – 2280)	0.3
PCC	2550	0.2
Rubblized PCC	2400	0.25
Modified Rubblized PCC	2450	0.25
Crack and Seat PCC	2500	0.25
Choke Stone/ rock interlayer	2100	0.35
Subgrade	1750	0.4

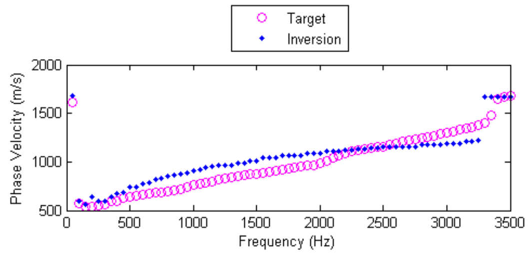
J40 (West)



Station 1



Station 2



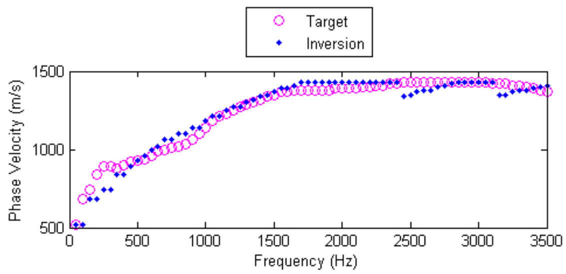
Station 3

Figure 37. Comparison of measured and theoretical dispersion curve for J40 (West) project

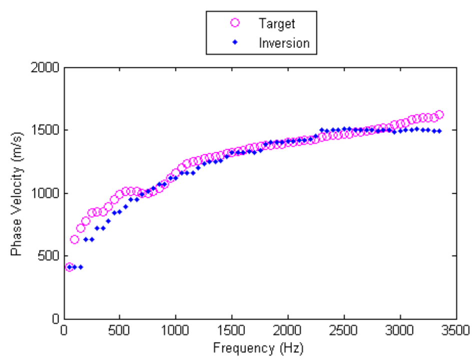
Table 36. Back-calculated shear velocity results for J40 (West) project

	Shear velocity (m/s)		
	HMA	Broken PCC	Subgrade
Station 1	1223.18	1775.71	386.09
Station 2	1078.84	1310.81	330.07
Station 3	1460.94	1997.73	536.84

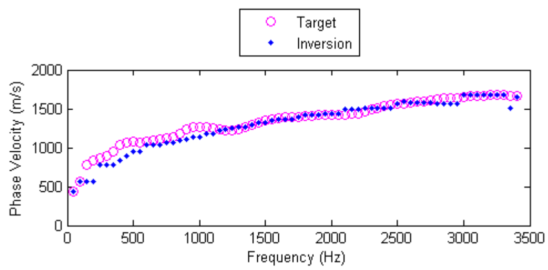
Y48



Station 1 (25 C)



Station 2 (25 C)



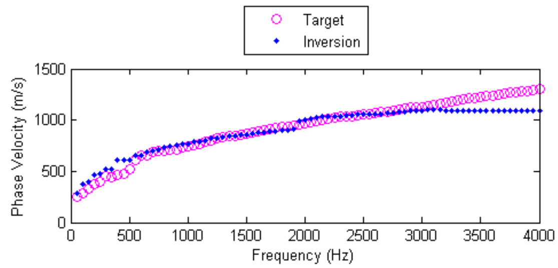
Station 3 (23 C)

Figure 38. Comparison of measured and theoretical dispersion curve for Y48 project

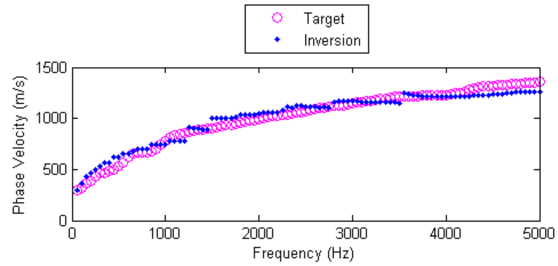
Table 37. Back-calculated shear velocity results for Y48 project

	Shear Velocity (m/s)		
	HMA	Broken PCC	Subgrade
Station 1	3127.82	2157.11	113.62
Station 2	3079.47	2423.60	149.33
Station 3	2961.37	2137.83	128.12

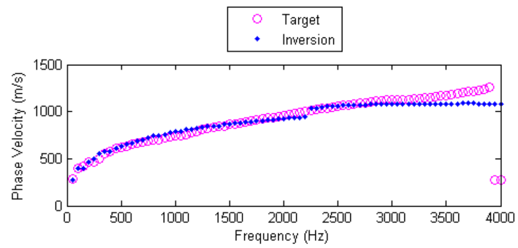
Y4E



Station 1 (18 C)



Station 2 (19 C)



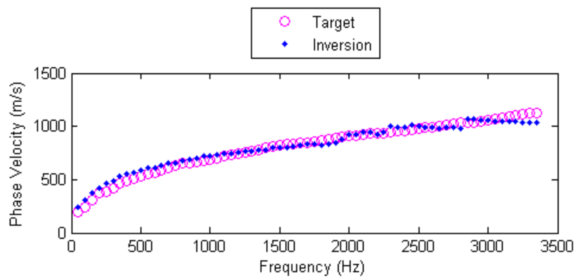
Station 3 (21 C)

Figure 39. Comparison of measured and theoretical dispersion curve for Y4E project

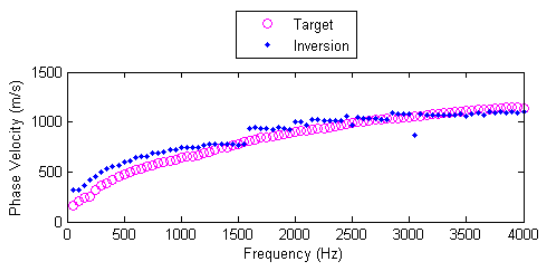
Table 38. Back-calculated shear velocity results for Y4E project

	Shear velocity (m/s)			
	HMA	Rock	PCC	Subgrade
Station 1	1304.08	673.00	1589.26	298.01
Station 2	1376.29	675.25	1538.69	285.39
Station 3	1259.89	723.99	1532.41	317.86

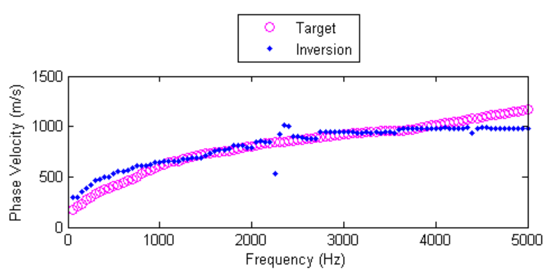
H-14



Station 1



Station 2



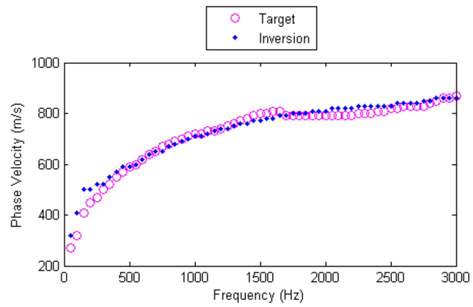
Station 3

Figure 40. Comparison of measured and theoretical dispersion curve for H14 project

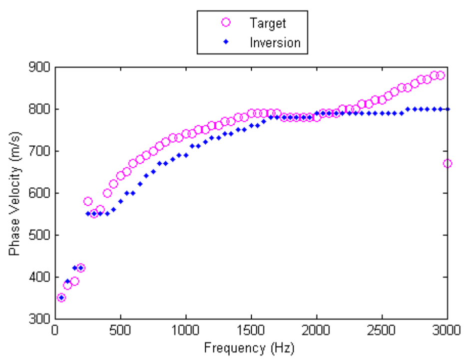
Table 39. Back-calculated shear velocity results for H14 project

	Shear velocity (m/s)			
	HMA	Rock	PCC	Subgrade
Station 1	1314.33	613.23	1427.80	275.72
Station 2	1369.82	671.54	1477.31	259.85
Station 3	1261.67	529.49	1148.16	235.67

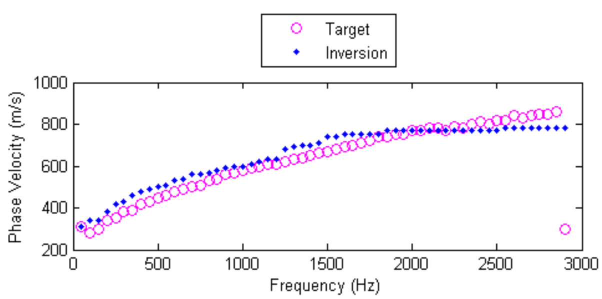
J-40 (East)



Station 1 (43 C)



Station 2 (40 C)



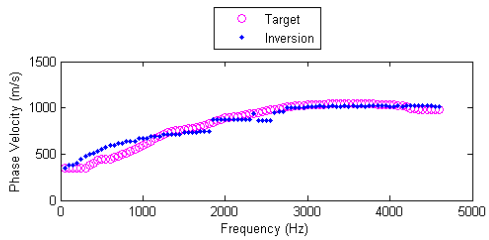
Station 3 (40 C)

Figure 41. Comparison of measured and theoretical dispersion curve for J40 (East) project

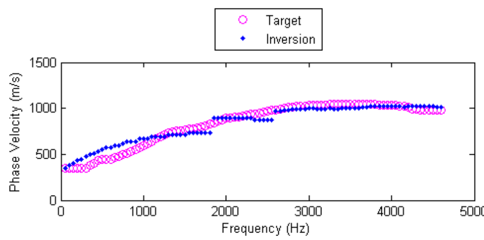
Table 40. Back-calculated shear velocity results for J40 (East) project

	Shear Velocity (m/s)			
	HMA	Rock	PCC	Subgrade
Station 1	968.79	608.60	1315.07	409.39
Station 2	912.14	562.85	1270.41	404.53
Station 3	911.13	513.02	1101.65	207.28

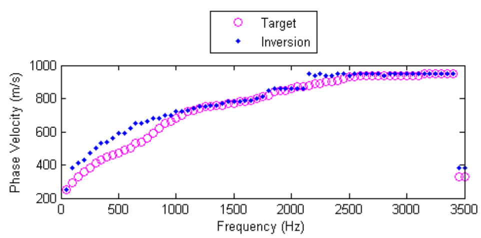
D 14



Station 1 (25 C)



Station 2 (25 C)



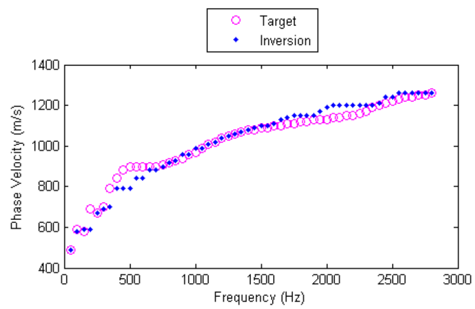
Station 3 (35 C)

Figure 42. Comparison of measured and theoretical dispersion curve for J40 (East) project

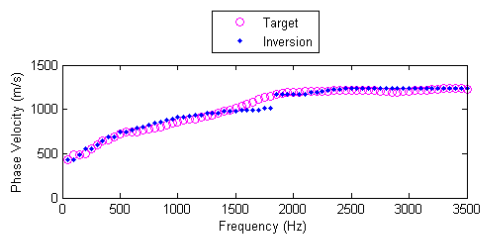
Table 41. Back-calculated shear velocity results for J40 (East) project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1137.45	1334.22	303.13
Station 2	1205.84	1182.41	356.02
Station 3	1299.00	1180.29	349.28

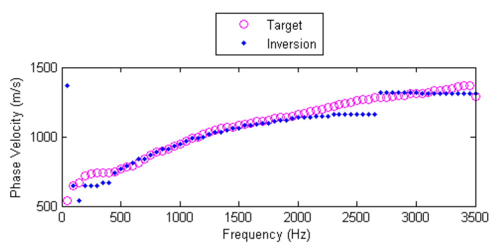
H24



Station 1



Station 2



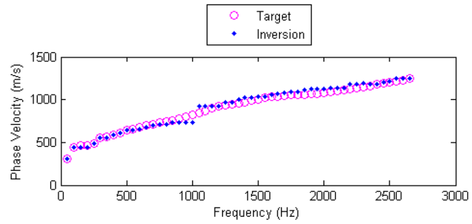
Station 3

Figure 43. Comparison of measured and theoretical dispersion curve for H24 project

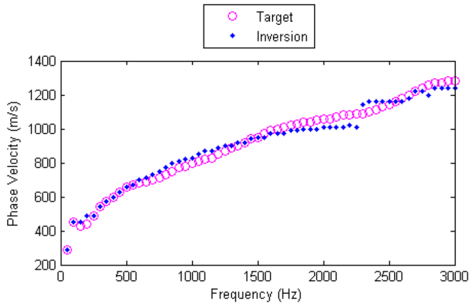
Table 42. Back-calculated shear velocity results for H24 project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1349.07	1791.18	593.39
Station 2	1281.88	1662.26	444.49
Station 3	1259.73	1803.31	512.85

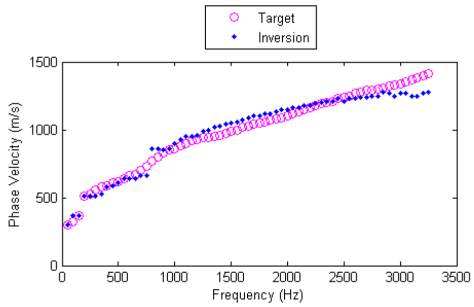
N72



Station 1 (22 C)



Station 2 (21 C)



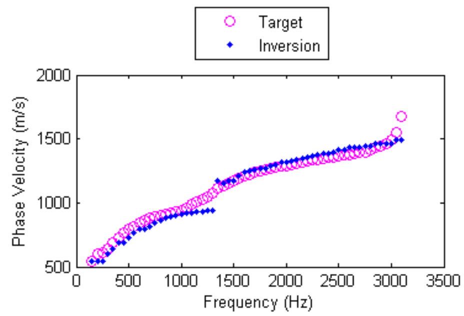
Station 3 (20 C)

Figure 44. Comparison of measured and theoretical dispersion curve for N72 project

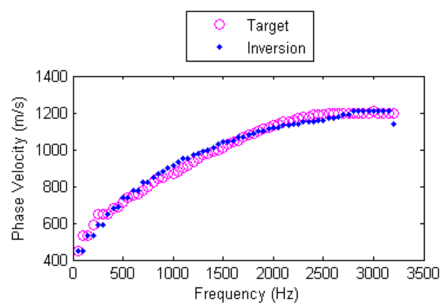
Table 43 Back-calculated shear velocity results for N72 project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1280.64	1739.90	329.97
Station 2	1102.07	1846.05	440.16
Station 3	1369.81	1712.01	300.60

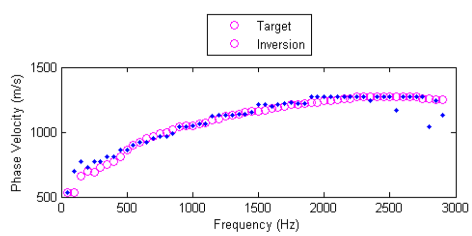
G61 (East)



Station 1 (20 C)



Station 2 (22 C)



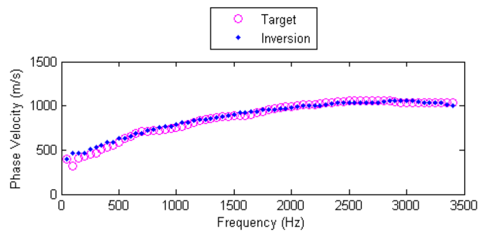
Station 3 (22 C)

Figure 45. Comparison of measured and theoretical dispersion curve for J40 (East) project

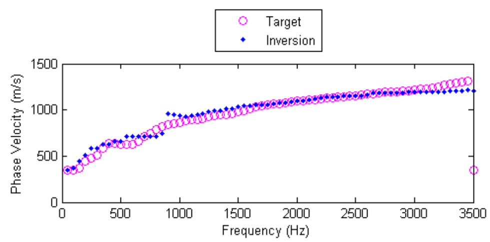
Table 44. Back-calculated shear velocity results for J40 (East) project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1852.81	1933.59	418.75
Station 2	1525.27	1801.29	506.78
Station 3	1841.97	1844.15	773.98

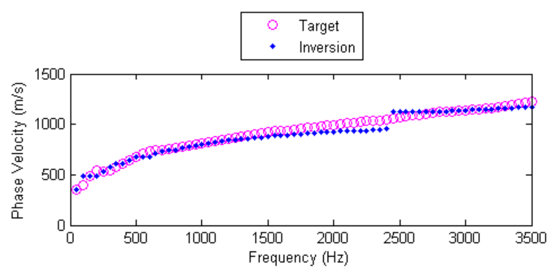
G61 (West)



Station 1 (17 C)



Station 2 (18 C)



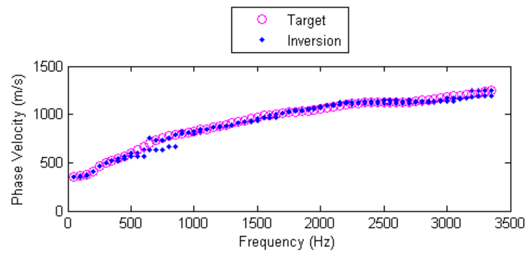
Station 3 (18 C)

Figure 46. Comparison of measured and theoretical dispersion curve for G61 (West) project

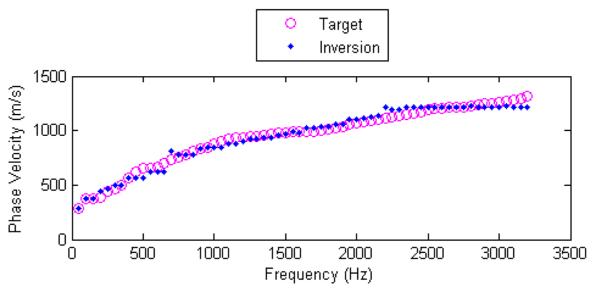
Table 45. Back-calculated shear velocity results for G61 (West) project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	800.55	1806.52	473.63
Station 2	1125.27	1677.15	406.48
Station 3	1041.90	1545.15	473.08

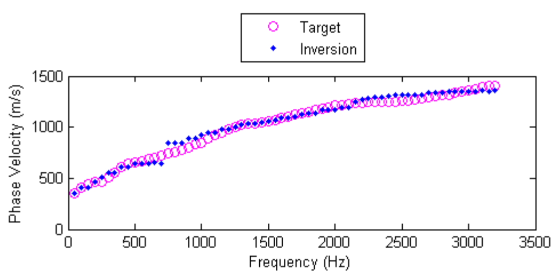
L55



Station 1 (14 C)



Station 2 (13.5 C)



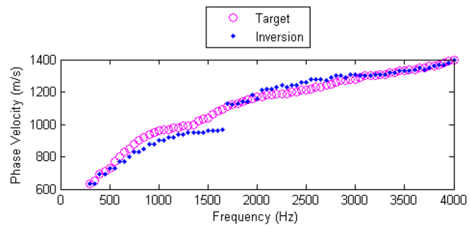
Station 3 (13.5 C)

Figure 47. Comparison of measured and theoretical dispersion curve for L55 project

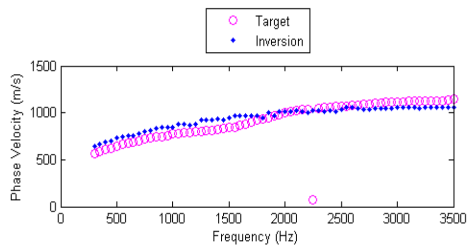
Table 46. Back-calculated shear velocity results for L55 project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1621.95	698.75	258.38
Station 2	1681.93	751.79	277.03
Station 3	1946.22	818.91	297.21

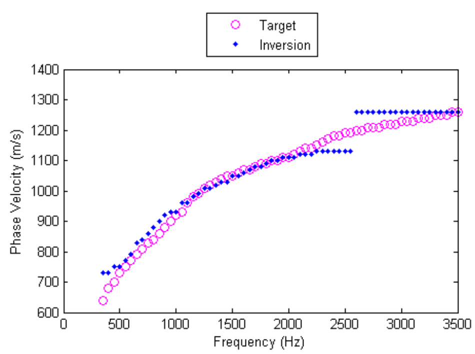
D16



Station 1



Station 2



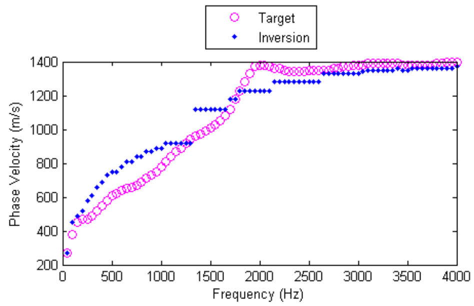
Station 3

Figure 48. Comparison of measured and theoretical dispersion curve for D16 project

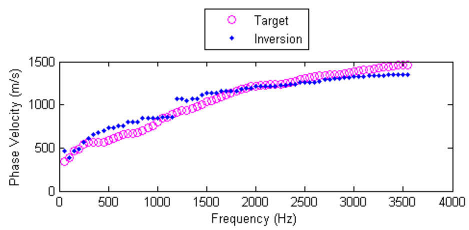
Table 47. Back-calculated shear velocity results for D16 project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1453.50	718.71	278.08
Station 2	1580.03	805.55	357.00
Station 3	1466.12	790.39	377.11

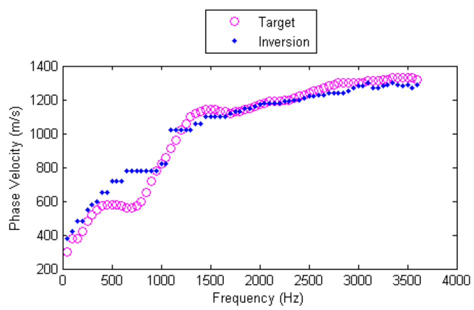
P29 (South)



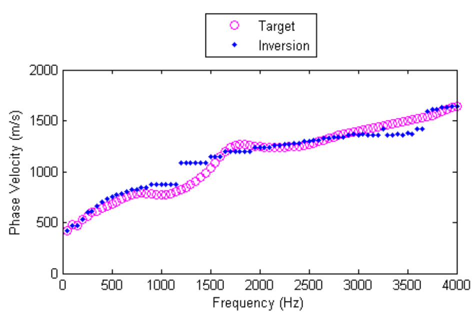
Station 1



Station 2



Station 3



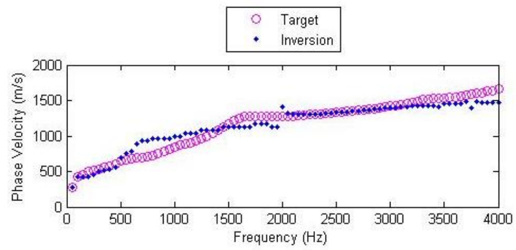
Station 4

Figure 49. Comparison of measured and theoretical dispersion curve for P29 (South) project

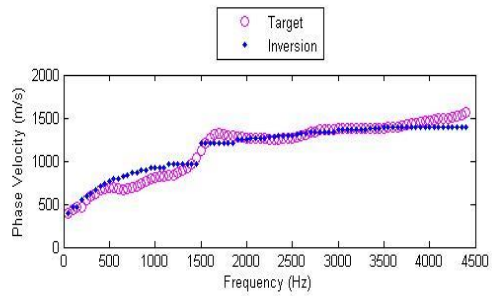
Table 48. Back-calculated shear velocity results for P29 (South) project

	Shear velocity			
	HMA	Rock	Broken PCC	Subgrade
Station 1	1452.51	904.51	1880.36	414.32
Station 2	1608.33	945.04	1583.50	384.89
Station 3	1449.73	986.65	1543.53	364.15
Station 4	1894.33	758.90	1704.9	398.00

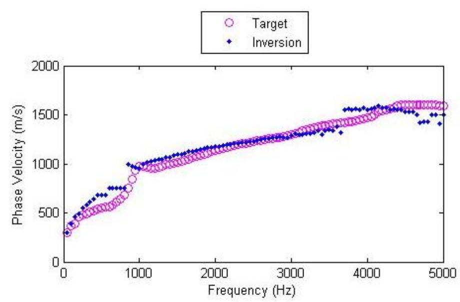
P29 (North)



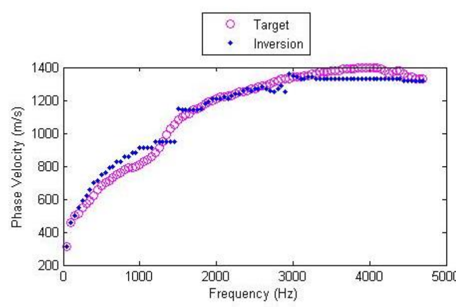
Station 1



Station 2



Station 3



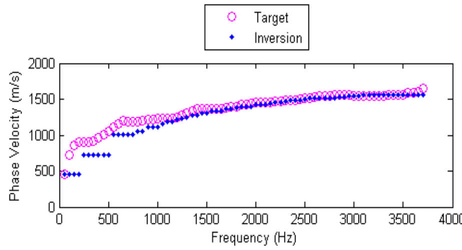
Station 4

Figure 50. Comparison of measured and theoretical dispersion curve for P29 (North) project

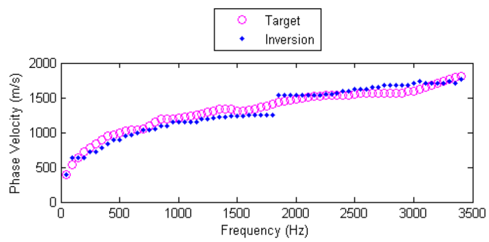
Table 49. Back-calculated shear velocity results for P29 (North) project

	Shear velocity			
	HMA	Rock	Broken PCC	Subgrade
Station 1	1944.97	988.30	1771.24	316.45
Station 2	1478.02	1069.43	1871.72	432.65
Station 3	1761.54	826.37	1620.21	325.12
Station 4	1637.42	850.12	1654.64	411.59

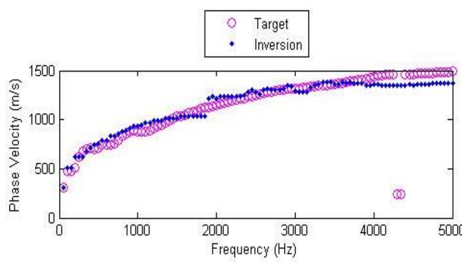
D43 Project



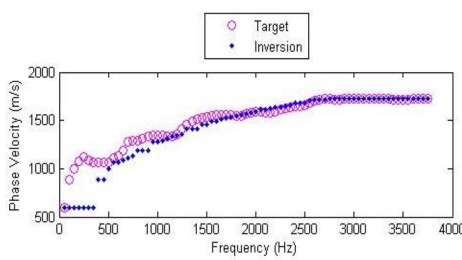
Station 1



Station 2



Station 3



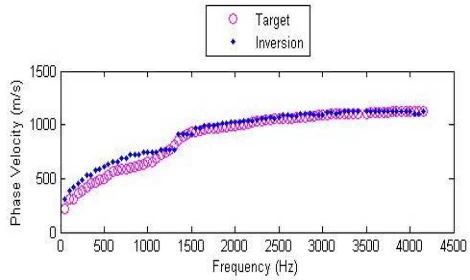
Station 4

Figure 51. Comparison of measured and theoretical dispersion curve for P43 project

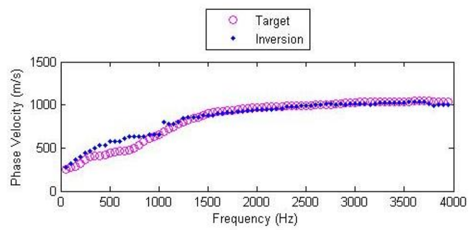
Table 50. Back-calculated shear velocity results for P43 project

	Shear velocity		
	HMA	Broken PCC	Subgrade
Station 1	1406.38	2271.78	423.88
Station 2	1977.00	2107.81	563.36
Station 3	1443.76	1551.60	465.77
Station 4	1765.31	2536.32	391.85

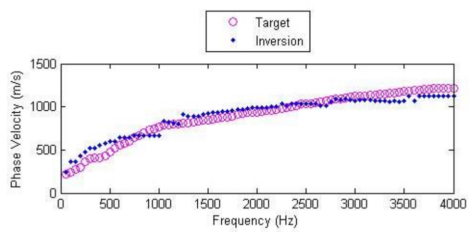
P59 Project



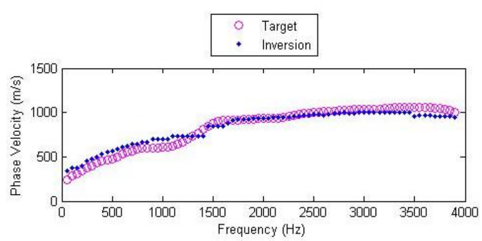
Station 1



Station 2



Station 3



Station 4

Figure 52. Comparison of measured and theoretical dispersion curve for P59 project

Table 51. Back-calculated shear velocity results for P59 project

	Shear velocity			
	HMA	Rock	Broken PCC	Subgrade
Station 1	1077.85	936.56	1363.31	342.05
Station 2	1029.12	920.81	1296.60	290.09
Station 3	1272.65	821.62	1368.98	301.99
Station 4	949.30	905.14	1341.60	326.52

APPENDIX D

Y4E Project

Per 50 meter survey by measuring wheel, it is found that the longitudinal joint crack is 12.8 m, center longitudinal crack is 2 m and 1 middle size thermal transverse cracking. No reflective cracking, but very bad longitudinal crack.

Figure 53. Performance condition on Y4E project

H 14 Project

Reflective/transverse cracking pops out in a few locations meters in medium size and cracks were sealed. Longitudinal cracking is whole along the shoulder.

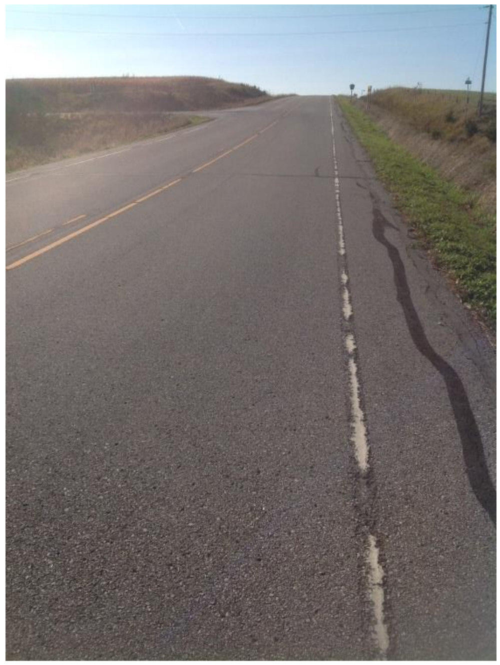


Figure 54. Performance condition on H14 project

H 24 Project

Reflective/transverse cracking pops out in occasionally (50 meter per crack) in medium size.

Figure 55. Performance condition on H24 project

L55 Project

Reflective/transverse cracking pops out every 6 meters in medium size. Some low temperature cracks are also seen.

Figure 56. Performance condition on L55 project

G61 (East) Project

Reflective/transverse cracking pops out every 8 to10 meters in medium size. Cracks were sealed. Have slight rutting along the wheel path.

Figure 57. Performance condition on G61 (East) project