Non-Cementitious Repair Materials Study

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Final Report

WisDOT Project No. 0092-18-02
June 2019

Research & Library Unit

Wisconsin Highway Research Program

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TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
0092-18-02		

4. Title and Subtitle Non-Cementitious Repair Materials Study	**5. Report Date** Final Report **6. Performing Organization Code**
7. Author(s) Prashant V. Ram, Kurt D. Smith, Ayesha Shah, Jan Olek, Tom Van Dam, and Larry Sutter	**8. Performing Organization Report No.** 0092-18-02
9. Performing Organization Name and Address Applied Pavement Technology, Inc. 115 W. Main St., Ste 400 Urbana, IL 61801	**10. Work Unit No.** **11. Contract or Grant No.**
12. Sponsoring Agency Name and Address Wisconsin Department of Transportation Research & Library Unit 4822 Madison Yards Way, Madison, WI 53705	**13. Type of Report and Period Covered** Final Report Oct 2017-Jun 2019 **14. Sponsoring Agency Code**

15. Supplementary Notes
Technical Oversight Committee Chair: Myungook (MK) Kang
Project Oversight Committee Chair: Jed Peters

16. Abstract
The Wisconsin Department of Transportation (WisDOT) is continually looking for state-of the-art technologies, materials, and methodologies to cost-effectively preserve the condition of their pavements so as to extend the service life and delay the need for major rehabilitation or reconstruction. In a search for a more durable and sustainable concrete pavement repair strategy, WisDOT has used non-cementitious repair materials on concrete pavements around the State with varying levels of success. Where poor performance from these repairs has been observed, it has been attributed to either poor workmanship or inappropriate use of the repair material for the prevailing concrete pavement condition. These factors suggest that the non-cementitious materials may have been used as a "band-aid fix" to allow for early opening to traffic rather than selecting and implementing the most suitable repair strategy to effectively address the specific distresses in the existing pavement.

This report presents a summary of an investigation into the use of non-cementitious repair materials. This includes the results of a literature review focused on the laboratory and field performance of non-cementitious repair materials as well as a review of state highway agency practices related to the application of non-cementitious repair materials. A field survey of five different non-cementations repair materials used in partial-depth repair (PDR) applications throughout Wisconsin was performed and the results documented. Additionally, the findings from a limited laboratory testing program conducted to assess the bond and dimensional stability properties of three non-cementitious materials at different testing temperatures are also presented. The report concludes with guidance on the use of non-cementitious repair materials for concrete pavement PDR applications in Wisconsin.

17. Key Words	18. Distribution Statement
Non-cementitious repair materials, repair materials, concrete pavement, partial-depth repair, spall repair, polymer materials, elastomeric concrete, repair performance	No restrictions. This document is available through the National Technical Information Service. 5285 Port Royal Road Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	138	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

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This research was funded through the Wisconsin Highway Research Program by the Wisconsin Department of Transportation and the Federal Highway Administration under Project 0092-18-02. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Wisconsin Department of Transportation or the Federal Highway Administration at the time of publication.

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SI* (RM-3DERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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Chapter 1. INTRODUCTION

Background

The Wisconsin Department of Transportation (WisDOT)—as well as many other state and local highway agencies—are continually looking for state-of the-art technologies, materials, and methodologies to cost-effectively preserve the condition of their pavements so as to extend the service life and delay the need for major rehabilitation or reconstruction. The State's high-priority roadways require repairs to be completed with minimal interruption to traffic flow, with lane closures limited to short durations during off-peak hours (typically 6 to 8 hours of night-time closures). These short closure times severely limit the ability to use many conventional cementitious repair materials and methods on their portland cement concrete (PCC) pavement infrastructure, and as a result WisDOT has typically resorted to using asphalt concrete or rapid-setting cementitious materials for repairs of PCC pavements, neither of which has consistently resulted in durable or long-lasting repairs.

In a search for a more durable and sustainable concrete pavement repair strategy, WisDOT has used non-cementitious repair materials on many of their PCC pavements around the State with varying levels of success. Where poor performance from these repairs has been observed, it has been attributed to either poor workmanship or inappropriate use of the repair material for the prevailing concrete pavement. WisDOT has also noted that a majority of the failures were due to the continued deterioration and the poor condition of the concrete around the repaired area[1]. These factors suggest that the non-cementitious materials may have been used as a "band-aid" fix to address traffic management concerns rather than selecting and implementing the most suitable repair strategy to effectively address the specific distresses in the existing pavement. Given WisDOT's experience with non-cementitious repair materials, there is a need for a more formal evaluation of these materials to determine their applicability and overall performance capabilities.

Project Objectives

The overall goal of this study is to develop recommendations regarding the proper selection and application of <u>non-cementitious</u>[2] repair materials for concrete pavements. Specific project objectives are stated below:

- Evaluate the availability and applicability of non-cementitious repair materials and products.
- Evaluate the processes used to identify the application of non-cementitious repair materials currently installed by WisDOT.
- Identify locations of non-cementitious repair materials and methods.
- Evaluate the performance of currently installed non-cementitious repairs.
- Develop recommendations for WisDOT specifications and manuals.

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[1] WisDOT Non-Cementitious Repair Materials Study RFP

[2] In this study, the term "non-cementitious" refers to non-hydraulic cement-based systems.

Research Approach

The project objectives were accomplished by the completion of the following five work tasks:

1. Conduct a thorough search of available literature focusing on current use and practices of non-cementitious repair materials and products for concrete pavement partial-depth repair (PDR) applications.
2. Develop a work plan and a testing matrix for evaluating the performance of currently installed non-cementitious repairs as well as for guiding the subsequent testing of selected repair materials in the laboratory.
3. Present key findings from the literature review and the proposed work plan to the Technical Oversight Committee (TOC) and update the work plan based on feedback received.
4. (a) Conduct field evaluation of the selected repair materials to document prevailing conditions of repair materials and surrounding concrete pavement.

   (b) Conduct limited coring to visually assess the bond condition between the repair material and the substrate concrete and to assess the quality of underlying substrate concrete.

   (c) Conduct limited nondestructive testing to assess the in situ dynamic elastic modulus of the repair material and the surrounding substrate concrete.

   (d) Conduct laboratory testing to evaluate the bond characteristics and dimensional stability of repair materials at different testing temperatures.
5. Document results of the study in this final report for use by WisDOT to assist in improving policy and specifications related to non-cementitious repair materials.

Report Organization

This report consists of five chapters (including this one), a listing of resources referenced in the report, and six appendices as summarized below:

- Chapter 2. Literature Review
- Chapter 3. Field Condition Evaluation of Repairs
- Chapter 4. Laboratory Testing
- Chapter 5. Conclusions and Recommendations
- References
- Appendix A. Core Photographs
- Appendix B. Repair Material-1 (RM-1) Site Visit Photographs
- Appendix C. Repair Material-2 (RM-2) Site Visit Photographs
- Appendix D. Repair Material-1 (RM-3) Site Visit Photographs
- Appendix E. Repair Material-1 (RM-4) Site Visit Photographs
- Appendix F. Repair Material-1 (RM-5) Site Visit Photographs

With the general background information and motivation for the study provided in this chapter, the following chapter includes a literature review focused on laboratory and field performance of non-cementitious repair materials and state highway agency specifications related to non-cementitious repair materials.

CHAPTER 2. LITERATURE REVIEW

Introduction

This chapter summarizes the literature review pertinent to this study. The literature review focused on current use and practices of non-cementitious repair materials and products for concrete pavement patching and PDR applications.

Literature Review

The literature search focused on work performed in the past 10 years and included a review of the Transportation Research Information Service (TRIS) database, the Transportation Research Board's (TRB's) Research in Progress database, selected state DOT specifications, and several other sources. The search results have been categorized into four distinct topic areas: types of repair materials, repair material selection considerations, laboratory and field performance of non-cementitious repair materials, and state departments of transportation (DOTs) experiences and specifications related to non-cementitious repair materials.

Types of Repair Materials

A wide range of repair materials are available for concrete pavement partial-depth repairs (PDR) applications. The materials can broadly be categorized into six categories (Smith et al. 2014,):

- **Conventional Concrete**: Conventional portland cement concrete is generally accepted to be the most suited material for concrete pavement repair applications. In situations requiring the pavement facility to be open to traffic quickly, Type I cement with an accelerating admixture or Type III cement may be used. Most highway agencies typically have a standard repair material mixture design that is routinely used for PDR applications.
- **Modified Hydraulic Cements**: A variety of hydraulic cement-based binders are available for PDRs. Some of these types of materials include calcium-aluminate cements, gypsum-cement based binders, and other proprietary mixtures. Some of these materials are very rapid-setting materials and allow for opening to traffic as early as 2 hours after placement.
- **Polymer-Based Materials**: Several types of rapid-setting, polymer-based repair materials are commercially available for PDR applications. These materials can either be hot applied or cold applied depending upon their chemical composition, and may use a polymer base or may incorporate polymer resins. Polymer-based materials used for pavement repairs can generally be classified into the following categories (Frentress and Harrington 2012):
  - Epoxy concrete: These are impermeable materials and tend to have good adhesive properties. In order to control heat build-up during placement, these materials are generally extended with coarse aggregates when placed in multiple lifts for deep repairs.
  - Methyl Methacrylate Concrete (MMC): These materials have long working times and exhibit good compressive strength and adhesion and can be placed over a wide range of temperatures [39 to 129 °F (4 to 54 °C)].
  - Polyester-styrene concrete: Though these materials have similar properties as MMC, they exhibit a slower rate of strength gain.

- Polyurethane concrete: These materials consist of a two-part polyurethane resin mixed with aggregate and they set very rapidly. Some types of polyurethanes can also be placed on wet substrates with no detrimental effects.
- Other polymeric materials: There are several other types of polymeric materials that are suitable for partial-depth repair applications. A vast majority of these materials are rapid-setting and highly impermeable. Materials using polymer-modified resins exhibit elastic properties that make them suitable for joint and crack repair without having to re-establish the joint. These types of materials are sometimes referred to as "elastomeric concretes" and are popular in bridge repair applications.

- **Magnesium Phosphate Concrete**: These materials set very rapidly and produce high early strength but are extremely sensitive to moisture and aggregate type (calcareous aggregates are not acceptable). Also, testing by other researchers have shown that these materials exhibit poor freeze-thaw durability (Ram, Olek, and Jain 2013).
- **Conventional Bituminous Materials**: Conventional bituminous concrete is generally used for stopgap repairs on concrete pavements until a more permanent solution can be found. These materials are relatively inexpensive, are easy to place, and can be opened to traffic right after placement.
- **Proprietary and Modified Bituminous Materials**: Several proprietary and polymer-modified bituminous materials are available for pavement repair applications. These materials can be very expensive but have been noted to perform better than conventional bituminous materials.

Commercially available polymer-based repair materials and proprietary bituminous materials, either hot or cold-applied, are the primary focus in this study.

Repair Material Selection Considerations

The selection of an appropriate repair material for an existing pavement structure requires a systematic and a rational approach. An overview of the important steps to be followed in this process is shown in figure 2-1.

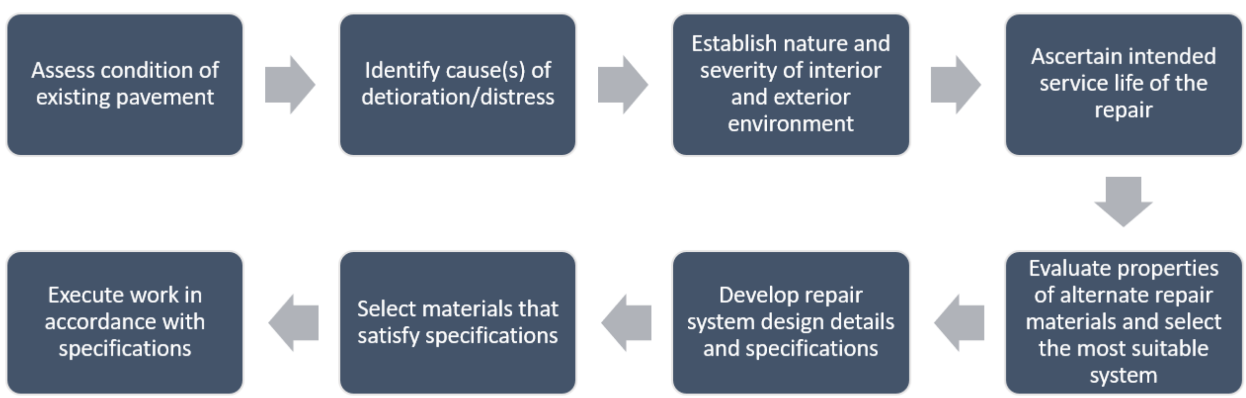

Figure 2-1. Steps in the selection of a repair material (adapted from McDonald et al. 2001 and ACI 2014).

For techniques such as partial- and full-depth repairs, the performance of repair materials used to address deteriorated areas in concrete pavements is dependent on several factors. Some of these factors are specific to the repair material (e.g., workability, rate of strength gain, shrinkage, freeze-thaw resistance) whereas others involve site-specific conditions and workmanship (e.g., condition of the existing concrete pavement being repaired, weather during placement, curing time and method) (Barde et al. 2006; Ram, Olek, and Jain 2013). The typical performance requirements, dictating characteristics, and the influencing material properties for a concrete pavement repair material are summarized in table 2-1.

Table 2-1. Performance requirements, dictating characteristics, and influencing properties for a concrete pavement repair material (Parameswaran 2004).

Repair Material Performance Requirements	Dictating Characteristics	Influencing Properties
Ability to meet the structural requirements associated with the load carrying capacity of the pavement.	• Load-carrying capacity • Bond to substrate concrete	• Compressive, tensile, and bond strength • Elastic modulus
Exhibit good workability and must be easy to mix, place, and finish.	• Flowability and sag characteristics • Turn-around time	• Rate of strength gain • Slump
Exhibit adequate durability when subjected to various exposure conditions such as: temperature and moisture changes, freeze-thaw cycles, and exposure to deicing salts.	• Dimensional stability under temperature and moisture variations. • Resistance to chloride and chemical attack • Freeze-thaw resistance	• Coefficient of thermal expansion • Permeability • Drying Shrinkage • Exotherm during placement and curing
Satisfy functional requirements and should provide a smooth and safe riding surface	• Rideability and smoothness • Skid resistance	• Material density • Nature of surface membrane and texture

Some of the other critical factors that should be considered during the repair material selection process include (Smith et al. 2014; Frentress and Harrington 2012):

- Placement conditions (ambient temperature and moisture levels).
- Repair area dimensions.
- Performance requirements.
- Project size.

Ultimately, there are three major factors that come into play when selecting a PDR material for concrete pavements:

- The opening time (i.e., how quickly the facility can be opened to normal traffic operations), which is often the critical factor that drives the selection of the repair material.

- Compatibility with surrounding concrete, which is affected by the elastic modulus, dimensional stability, shrinkage, coefficient of thermal expansion, and the degree of bonding to the existing concrete pavement and all should be carefully evaluated during the material selection process.

- Long-term durability of the repair material, which is particularly important when the materials are exposed to harsh freezing environments as are routine in Wisconsin.

Laboratory and Field Performance of Non-Cementitious Repair Materials

This section summarizes the laboratory and field performance of a few non-cementitious repair materials reported in the literature.

Investigation of Ultra-Thin Fiber-Reinforced Concrete Overlay, Recycled Concrete Aggregate Slab, and Patching Materials Using Laboratory and Accelerated Performance Tests (Kuo and Armaghani 2001)

This study evaluated three types of commercially available repair materials in the laboratory: cementitious mortar, polymer concrete, and elastomeric concrete. The study also involved the accelerated testing of mock-up repair patches prepared in the laboratory. The testing was conducted using the Circular Accelerated Test Track (CATT) facility developed by the University of Central Florida. The ambient air temperature varied between 60 and 80 °F (15.5 °C and 26.7 °C) during the testing. After a total of 500,000 repetitions using a 10,000 lbf wheel load applied to the repair patches, none of the cementitious mortars or polymer concrete materials exhibited any major signs of distress or deterioration. The main issue observed was the delamination of the elastomeric patching materials from the substrate concrete, which appeared to be more susceptible to debonding when compared to the polymer concretes or cementitious materials.

Investigation of Spall Repair Materials for Concrete Pavement (Markey et al. 2006)

This study investigated the laboratory and field performance of a few commercially available repair materials for concrete pavement spall repairs in Texas. The repair materials used in the study were classified into three general stiffness categories: rigid, semi-rigid, and flexible. Rigid materials included magnesium phosphate and hydraulic cement-based materials; semi-rigid and flexible materials included polymer-based materials. Table 2-2 shows the repair materials evaluated in this study.

Table 2-2. Repair material included in Texas study (Markey et al. 2006).

Repair Material	Material Type	General Category
RSP	Polyurethane Polymer Concrete	Semi-Rigid
Delpatch	Polyurethane Polymer Concrete	Flexible
Wabo Elastopatch	Polyurethane Polymer Concrete	Flexible
FlexPatch (SSI)	Epoxy Polymer Concrete	Semi-Rigid
FlexKrete	Thermosetting Vinyl Polymer Concrete	Semi-Rigid
EucoSpeed MP	Magnesium Polyphosphate	Rigid
MG-Krete	Magnesium Polyphosphate	Rigid
Pavemend 15	Magnesium Polyphosphate	Rigid
Rapid Set	Hydraulic Cement	Rigid
Fibrescreed	Polymer-Modified Bitumen	Flexible

Elastic modulus was considered the key material property indicative of the dimensional stability of the material when used for spall repair applications. For a high-modulus material, small changes in the material volume can generate large stresses in the material and the surrounding concrete that can result in debonding and cracking issues. Figure 2-2 shows a comparison of the elastic modulus values for the different materials [at 70 °F (21 °C)] tested in this study.

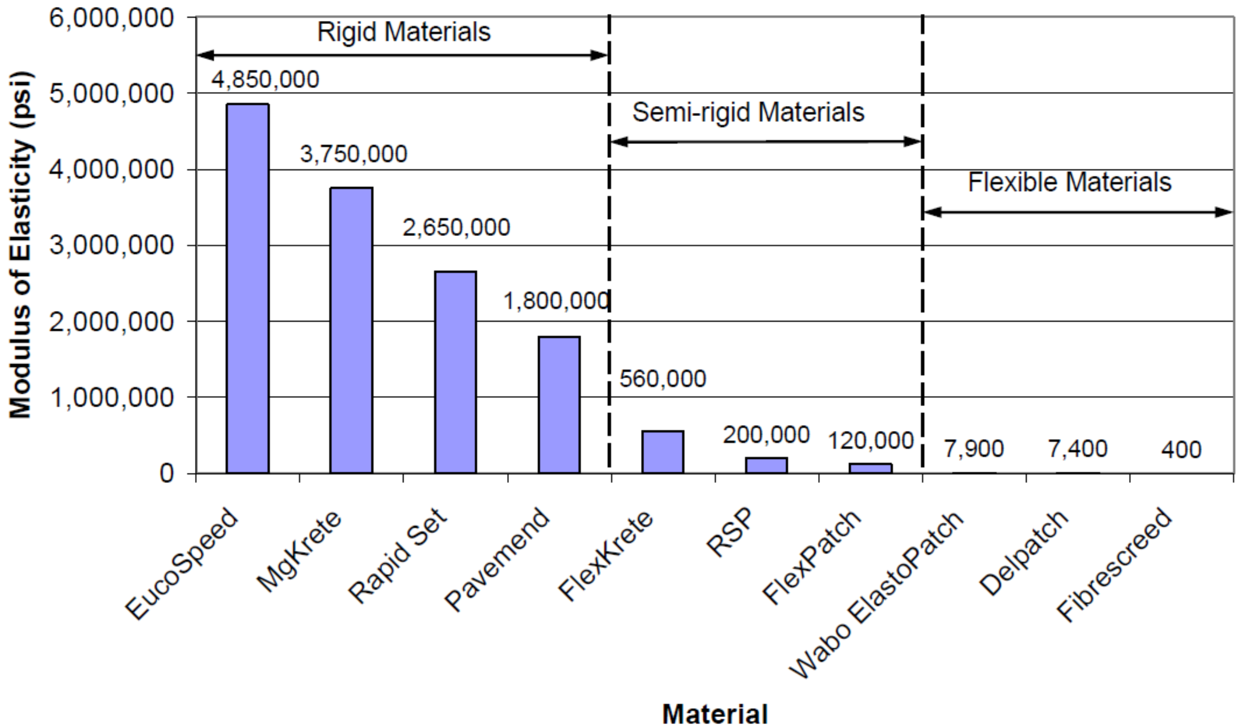

Figure 2-2. Elastic modulus of the repair materials in Texas study (Markey et al. 2006).

The elastic modulus testing showed the rigid materials exhibited modulus values that were 3 to 10 times greater than the polymer-based materials. The modulus values for the semi-rigid materials ranged from 120,000 to 560,000 psi and the flexible materials exhibited modulus values between 400 and 7,900 psi. The study also noted the challenges in conducting the elastic modulus testing for flexible materials due to the non-linear stress-strain behavior of these materials.

The study also evaluated the tensile bond strength between the repair materials and a substrate concrete. A comparison of the bond strength values for the various materials is shown in figure 2-3. General trends showing the impact of temperature on the bond strength could not be discerned.

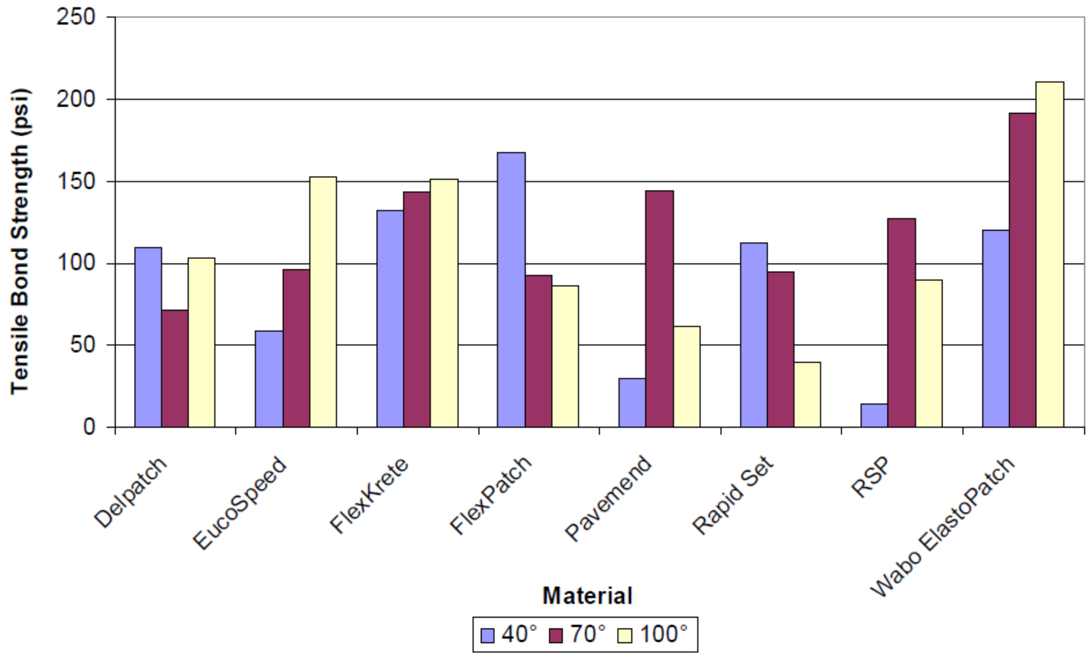

Figure 2-3. Tensile bond strength of the repair materials in Texas study (Markey et al. 2006).

Shrinkage of the materials in the first 18 hours after placement was also studied. The length change in the material was recorded while the material was being cured in controlled environmental chambers maintained at 40 °F (4 °C), 70 °F (21 °C), and 100 °F (38 °C). The maximum shrinkage was calculated as the maximum percent length change during the 18-hour curing period. Figure 2-4 shows a comparison of the initial shrinkage of each of the materials tested. The flexible and semi-rigid materials (RSP, Delpatch, Wabo ElastoPatch, FlexKrete, and FlexPatch) exhibited moderate to high shrinkage.

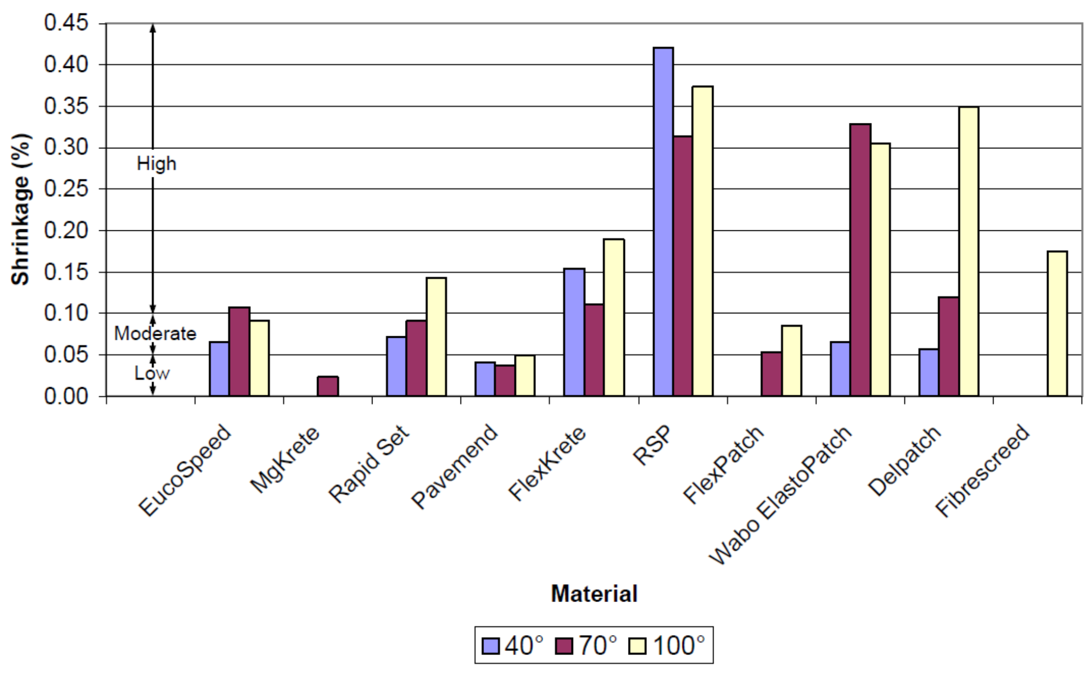

Figure 2-4. Initial shrinkage of the repair materials in Texas study (Markey et al. 2006).

The coefficient of thermal expansion (CTE) was also determined for each of the materials included in the study, with the results shown in figure 2-5. The rigid materials had relatively low CTE values (between 5.7×10^{-6} and 7.5×10^{-6} in/in/°F), which is very comparable to that of normal-weight concrete. In comparison, the semi-rigid and flexible materials exhibited CTE values that were much higher (between 16.4×10^{-6} and 65.1×10^{-6} in/in/°F).

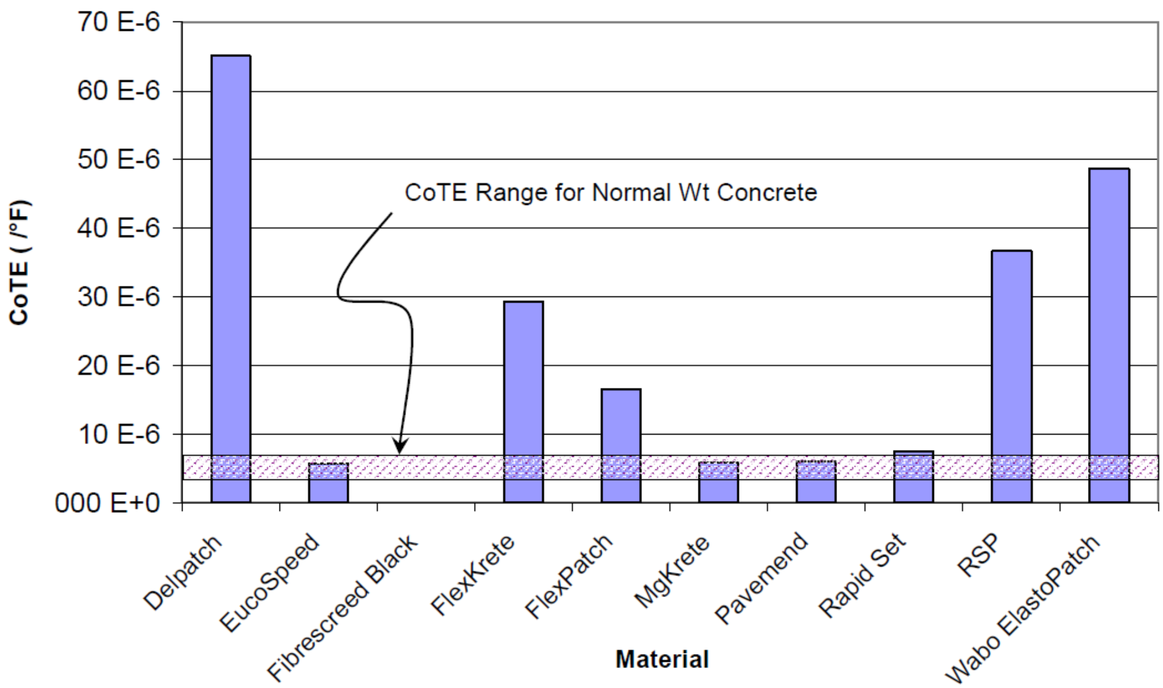

Figure 2-5. CTE of the repair materials in Texas study (Markey et al. 2006).

A few cores were extracted from successful repair jobs in the field and the bond strengths, elastic moduli, and compressive strengths of the cores were compared to the laboratory-prepared specimens. While the field specimens generally exhibited lower values, definitive conclusions could not be made due to the small size of the sample set.

Performance of Various Partial-Depth Repair Materials at the MnROAD Facility (Burnham, Johnson, and Worel 2016)

A recent study by the Minnesota Department of Transportation (MnDOT) evaluated the performance of various partial-depth repair materials at the MnROAD facility. Out of the 22 materials evaluated as a part of the study, two materials were epoxy-based repair systems (Alkona Rapid Patch Pavement Repair and Pro-Poxy 2500) and the rest were either asphalt-based or hydraulic cement-based repair materials. The materials were installed in 2011 and the report describes the 3-year performance of the repairs. The two epoxy-based materials exhibited varying levels of performance based on the installation location with Pro-Poxy 2500 exhibiting better performance than Alkona Rapid Patch.

The condition of the patches were evaluated using a subjective rating scale (a 5-point rating scale was used, with 5 indicating "excellent" condition with no cracking and 0 indicating a completely failed patch that had to be replaced; a condition rating of 3 indicated "good" serviceable condition) that was primarily based on visual observations and sounding using a ball-peen hammer to provide bond condition information. Around 60 percent of the patches (55 of 93 patches) installed remained in good serviceable condition after 3 years of service. The primary distresses noted were random cracking, deterioration along the joints, and loss of bond to the substrate concrete. One interesting observation from the study was the location of the patches (i.e., either on the centerline or near the loaded areas) seemed to have little effect on their overall performance.

Evaluation of High Performance Pavement and Bridge Deck Wearing Surface Repair Materials (Delatte et al. 2016)

This study evaluated the laboratory and field performance of a few proprietary non-cementitious repair materials for concrete pavement and bridge deck patching applications. The primary objective was to provide the Ohio Department of Transportation (ODOT) with materials and procedures that would help reduce bridge and roadway closure times. Table 2-3 shows the materials tested in this study along with some data on the cost and some general characteristics of the repair materials

Table 2-3. Repair material included in the study (Delatte et al. 2016).

Product	Material Category	Cost per ft^3	Primer Needed?	Opening to traffic, hours	Temperature range, °F (°C)
FlexSet	Polymer concrete	$ 235.00	No	0.5	-10 to 140 (-23 to 60)
MG-Krete	Magnesium phosphate	$ 122.22	No	2	Above 14 (above -10)
SR-2000	Polymer concrete	$ 175.00	Yes	2	35 to 120 (2 to 50)
Delpatch	Polyurethane elastomeric	$ 232.43	Yes	1	Above 45 (above 7)
FastSet DOT Mix	Rapid hardening	$ 11.32	No	1.5	Not provided*
Repcon 928	Polymer modified	$ 57.36	No	3	45 to 85 (7 to 29)*

*Use cold mixing for higher temperatures

These materials were installed in bridge decks and on concrete pavements along US 35 near Xenia, OH in 2014 and 2015. After about 2 years of service, all the materials were reported to be performing well. There were a few instances of cracking and other failures, but these were not tied to any particular material. In the few cases where excessive cracking or partial failures were observed, the primary cause appeared to be issues with the substrate concrete. The results of the study were used to develop a repair material specification and a decision matrix (see table 2-4) for the selection of repair materials.

Table 2-4. Decision matrix for repair material selection (Delatte et al. 2016).

Factor	Categories	Recommendation	Example Materials
Traffic Interruption	Low traffic, long closure possible	Conventional cement-based, lower cost repair material	Conventional concrete, Rapid hardening concrete
	High traffic, short daytime closure	Lower cost repair material, or HP repair material, allow bonding agent, open to traffic in 2 hours	Polymer-modified or polyurethane elastomeric concrete
	Very high traffic, short night closure only	HP repair material not requiring bonding agent, rated for traffic opening in 1 hour	Magnesium phosphate, polymer-modified, or polymer concrete
Durability Requirement	Short term solution, facility replacement within 5 years	Rapid hardening or polymer-modified concrete	Short-term solution, facility replacement within 5 years
	Long term solution, 10 to 15 years	Magnesium phosphate or polyurethane elastomeric concrete	Long-term solution, 10 to 15 years
Temperature During Installation	Low (near or below freezing)	Low temperature rated material	Magnesium phosphate or polymer concrete
	Moderate (40 to 70 °F)	Conventional or HP	Rapid hardening, polymer-modified, or polyurethane elastomeric concrete
	High (80 °F and higher)	Conventional, HP only if high temperature rated or with retarder	Rapid hardening or polymer modified concrete
Patch Size	Small, less than about 2 by 2 feet by 3 inches deep (600 by 600 by 75 mm)	Use small batches, do not extent material with pea gravel	Rapid hardening, polymer-modified, or polyurethane elastomeric concrete
	Larger than about 2 by 2 feet by 3 inches deep (600 by 600 by 75 mm)	Use a portable higher capacity mixer, extend with pea gravel	Magnesium phosphate or polymer-modified concrete w/pea gravel
Bridge deck Substrate Condition	Distress limited to top third or half	Surface patch	Based on other criteria
	Distress through full thickness	Cut through and form full depth patch	Based on other criteria
Pavement Type	Asphalt pavement	These materials are not recommended	These materials are not recommended
	Concrete pavement	All materials tested are satisfactory	Based on other criteria

HP: High Performance

State DOT Specifications

Several state DOT standard specifications were reviewed, and it was noted that only a few maintain specifications for the types of non-cementitious repair materials being investigated under this WisDOT study. This section summarizes some of the main provisions in state DOT specifications that are relevant to the scope of this study.

New York

The New York State Department of Transportation (NYSDOT) maintains a specification for elastomeric concrete repair materials. The required mechanical properties for these materials are summarized in table 2-5.

Table 2-5. Required mechanical properties for elastomeric concretes (NYSDOT 2018b).

Test	Procedure	Minimum Requirements
Resilience	ASTM C579	70%
5-Hr. Compressive Strength	ASTM C579 (modified)	500 psi
4-Hr. Compressive Strength	ASTM C579 (modified)	2000 psi
7-Day Tensile	ASTM D638	150 psi
7- Day Tear	ASTM D624	40 lb_f/in
Pot Life	Gardco GT-S Gel Timer	5 minutes

Specifications related to the approval and acceptance of these materials are summarized below.

- **Material approval**: manufacturers are required to submit material detail sheets to NYSDOT for approval and once approved, it will be placed on NYSDOT's approved list. To maintain the material on the approved list, the materials are evaluated every 6 months over a 2-year period from the date of field installation. If the material is performing satisfactorily over the 2-year evaluation period, it will be retained on the approved list.
- **Acceptance**: Acceptance is based on the material appearing on NYSDOT's approved materials list. The material supplier is required to provide all supporting documents (e.g., material detail sheets, safety data sheets) 14 days prior to shipment of the product to the job site.

Texas

The Texas Department of Transportation (TxDOT) classifies polymeric patching materials into two types:

- **Type I**: Flexible material with high resilience properties. This material is not intended for use in areas where an asphalt overlay is expected.
- **Type II**: Semi-rigid material with a high compressive strength. This material is preferred when an asphalt overlay is expected.

Manufacturers must follow the pre-qualification procedure that involves sampling and testing before the material can be placed on the approved materials list. The materials must be qualified every 6 months to remain on the approved materials list. Materials not on the approved list may be allowed for use based on project-specific testing. Some of the key requirements outlined in the TxDOT specifications are summarized in table 2-6.

Table 2-6. TxDOT requirements for polymeric patching materials (TxDOT 2014).

<table>
<tr><th colspan="4">General Requirements</th></tr>
<tr><td colspan="4">• Ability to carry traffic within 3 hours of placement or as directed by the engineer.
• Resistant to weather and abrasion.
• Skid-resistant surface texture.
• Non-reflective finish similar to color tone of concrete.
• Must be placed at substrate temperatures of 10 °C and rising.</td></tr>
<tr><th colspan="4">Chemical Resistance (ASTM D 471, 25 °C after 22 hr.)</th></tr>
<tr><th colspan="2">Chemical</th><th colspan="2">Effects</th></tr>
<tr><td colspan="2">Deicers</td><td colspan="2">None</td></tr>
<tr><td colspan="2">Motor Oil</td><td colspan="2">None</td></tr>
<tr><td colspan="2">Sodium Chloride Solution (5%)</td><td colspan="2">None</td></tr>
<tr><td colspan="2">Hydraulic Brake Fluid</td><td colspan="2">None</td></tr>
<tr><th colspan="4">Physical Requirements: Type I Material</th></tr>
<tr><th>Test</th><th>Procedure</th><th>Requirements for Type I Material</th><th>Requirements for Type II Material</th></tr>
<tr><td>Gel time, min</td><td>Tex-614-J</td><td>5 min, 60 max.</td><td>1 min, 60 max.</td></tr>
<tr><td>Wet Bond Strength to Concrete, psi</td><td>Tex-618-J</td><td>100 (min.)</td><td>250 (min.)</td></tr>
<tr><td>Compressive Strength, 24 hr. psi</td><td>ASTM C 579, Method B</td><td>200 (min.)</td><td>2,000 (min.)</td></tr>
<tr><td>Compressive Stress @ 0.1 in., 7 days, psi</td><td>Tex-618-J</td><td>200 (min.)</td><td>2,000 (min.)</td></tr>
<tr><td>Resilience, %</td><td>Tex-618-J</td><td>90 (min.)</td><td>65 (min.)</td></tr>
<tr><td>Thermal Compatibility One cycle is 8 hrs. @ 60 °C, followed by 16 hrs. @ -21°C Determine results after 9 cycles</td><td>ASTM C884/ C884M with modifications</td><td>No delamination or cracking</td><td>No delamination or cracking</td></tr>
</table>

North Carolina

The North Carolina DOT (NCDOT) allows the use of elastomeric concretes for repairing transverse joints. The performance requirements for elastomeric concretes are summarized in table 2-7. In addition to the requirements summarized in table 2-7, the NCDOT also requires elastomeric concretes to resist water, chemical, ultraviolet and ozone exposures, and to withstand extreme temperature changes. A manufacturer's representative is required to be present on-site during the installation of elastomeric concretes until the DOT crew is experienced in working with the material.

Table 2-7. Performance requirements for elastomeric concretes (NCDOT 2016).

Concrete Properties		
Property	**Procedure**	**Minimum Requirements**
Bond Strength	ASTM D638	450 psi
Brittleness by Impact	Ball Drop	7 ft-lb
Compressive Strength	ASTM D695	2,800
Binder Properties		
Tensile Strength	ASTM D638	800 psi
Ultimate Elongation	ASTM D638	150%
Tear Resistance	ASTM D624	90 lb/in.

Virginia

The Virginia DOT (VDOT) has developed a specification for elastomeric concretes for use in bridge applications. The performance requirements are summarized in table 2-8.

Table 2-8. Performance requirements for elastomeric concretes (Balakumaran et al. 2016).

Property*	**Minimum Requirements**
Elastic Strain at Yield	10% (compression and tension)
Elastic Modulus (E)	5,000 psi (0 to 120 °F)
Compressive Strength	Larger of 500 psi or E/10
Tensile Strength	E/5 (at all values of E, 0 to 120 °F)
Shear Strength	Larger of E/9 or 500 psi
Bond Strength	250 psi
Other Requirements	
• Ability to bond to itself • Maximum set time of 6 hours, longer durations accepted if material is exceptionally durable • Meet skid resistance requirements	

*test method not specified

Other State Highway Agency Experiences

The New York State DOT (NYSDOT) has had experience with using non-cementitious repair materials and the four non-cementitious materials on their approved list of materials are: EucoSpeed MP, Pavemend SLQ, MasterEmaco T 545, and MasterEmaco T 545 HT (NYSDOT 2018a). For bridge deck and joint header repairs, elastomeric concretes are the preferred option because of their improved workability and the quick set times. Some of the elastomeric concretes that NYSDOT has used with great success include: Concrete Welder Gray and Polyflex DS Gray (from Roklin Systems Inc.), Liquid Ply-Krete and Ply-Krete (from Polyset Company, Inc.), PF-60 Rapid Surface Repair (from Five Star Products, Inc.), Silspec 900 PNS (from Silicone Specialties, Inc), Silspec 2000 (from C.S. Behler, Inc.), and Wabocrete II (from Watson Bowman Acme Corporation) [Jennifer Hawkins, NYSDOT, personal communication, November 2017].

The California DOT (Caltrans) requires the use of polyester concrete exclusively for its partial-depth repairs. Although polyester concretes are more expensive than traditional cementitious materials or commercially available rapid-setting materials, they exhibit better performance over a wider range of conditions and develop very good bond strengths. Polyester concretes also cure rapidly and the rate of strength gain is fast enough for early opening to traffic (Caltrans 2015).

The Louisiana DOT has had a long history of using RM-4 and RM-1, which both have exhibited good field performance (Donmeyer 2016). These materials are hot-applied synthetic polymer-modified resinous materials.

Summary

Non-cementitious materials are not as commonly used as cement-based materials for concrete pavement repair applications and the available literature on laboratory and field performance of these materials is limited. A few state highway agencies have developed specifications for the acceptance and use of polymer-based non-cementitious materials. The following general conclusions can be drawn from the review of the available literature:

- Polymer materials exhibit high CTE and significant initial shrinkage.
- The elastic modulus of repair materials at various temperatures and the bonding strength to the substrate concrete may be considered to be key indicators of their potential field performance.
- Consideration of future overlay activities should be given due consideration during the material selection process as polymer-based, non-cementitious materials may not bond well to a concrete overlay.
- The performance of the repair material is dependent on the condition of the surrounding concrete pavement.

As WisDOT moves towards the development of specifications for the application of non-cementitious repair materials, lessons learned from the experiences of other highway agencies that use these materials will help establish the protocols for the successful implementations of repairs using non-cementitious, polymer-based materials.

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CHAPTER 3. FIELD CONDITION EVALUATION OF REPAIRS

Introduction

This chapter presents the details on the field condition evaluation of five selected non-cementitious repair materials. The final testing matrix used for field and laboratory evaluations, as well as the field performance of the various repairs evaluated in the field, are discussed.

Final Testing Matrix

Several factors influence the performance of pavement repairs. Some of these factors are specific to the repair material (e.g., workability, rate of strength gain, shrinkage, bond strength, freeze-thaw resistance) and some involve site-specific conditions and workmanship (e.g., condition of the existing concrete pavement being repaired, weather during placement, curing time and method). With this large number of variables, no single study can be designed to provide statistically valid results that will be universally applicable to all conditions. Instead, the experimental plan proposed for use in this project was designed to make use of the significant amount of information that already exists regarding most of these factors to evaluate the performance of existing repairs and also to develop recommendations regarding the use of non-cementitious repair materials in partial-depth repair of concrete pavements.

Five repair materials were evaluated in this study as shown in table 3-1. As indicated in the table, some materials were evaluated both in the field and the laboratory (RM-2, RM-4, RM-5) and some materials were evaluated only in the field (RM-1 and RM-3).

Table 3-1. Final testing matrix.

Repair Material	Material Type	Field Evaluation	Laboratory Evaluation
RM-1	Hot-applied flexible material formulated with polymer-modified resins, fiberglass, mineral fillers, and high-quality aggregates.	Yes	No
RM-2	Polyester polymer concrete.	Yes (only bridge decks)	Yes
RM-3	Hot-applied polymer-modified asphalt repair mastic.	Yes	No
RM-4	Hot-applied polymer-modified synthetic resin with fibers, fillers, fines, and high-quality aggregate.	Yes	Yes
RM-5	Hot-applied polymer-modified asphalt with engineered aggregates and modifiers.	Yes	Yes

Note: Information on the material was obtained from the product data sheets posted on the material producer’s websites.

Field Condition Evaluation Procedures

Field evaluations focused on collecting data and identifying factors that could potentially affect the performance of the repairs. Field evaluations included visual investigations, nondestructive testing, and coring. Since traffic control was not available for the visual investigations, condition assessments were performed from the shoulder. It should be noted that the field evaluations did not involve a comprehensive survey but instead focused on the performance of the repair materials and the general condition of the surrounding concrete pavement.

Visual Investigations

The visual investigations consisted of an evaluation of the general condition of the repaired area and the adjacent concrete pavement. A subjective rating scale was used to evaluate the condition of the repairs (see table 3-2). In addition, geo-referenced photographs of the repaired area and the surrounding concrete were captured to document the prevailing conditions. At each site visited, all areas repaired using the materials being studied in this project were evaluated to arrive at a general overall rating (summarized in table 3-3).

Table 3-2. Condition rating scale used to evaluate repair patches (adapted from Burnham, Johnson, and Worel 2016).

Rating	Condition of Repair
Excellent	No random cracking or failures observed
Good	Small number of tight cracks, no material missing
Fair	Multiple cracks and some material missing
Poor	Substantial material missing, portions replaced/refilled
Failed	Completely failed repair

Coring

In order to investigate the quality of the bond between the repair material and the substrate concrete, a limited amount of pavement coring was conducted in areas that appeared to be sound based on the condition of the repaired surface. Nominal 4-inch diameter cores were extracted, and the core holes were filled with cold patch material. In addition to visual examination of the cores, ultrasonic pulse velocity through the core specimens of the repair materials was also conducted (discussed in Chapter 4).

Nondestructive Testing

The project team also conducted a limited amount of nondestructive evaluation using a portable seismic pavement analyzer (PSPA) to assess the in situ dynamic elastic modulus of the repair material and the surrounding substrate concrete (see figure 3-1). The testing was only conducted at locations where the coring was performed.

Figure 3-1. Portable seismic pavement analyzer (PSPA).

Repair Material Performance

With the help of WisDOT maintenance personnel and material manufacturer representatives, the project team was able to identify several locations for field condition evaluations (see figure 3-2). Table 3-3 presents a summary of the field sites visited and the results of visual investigations. Table 3-4 summarizes the results of the coring and PSPA testing.

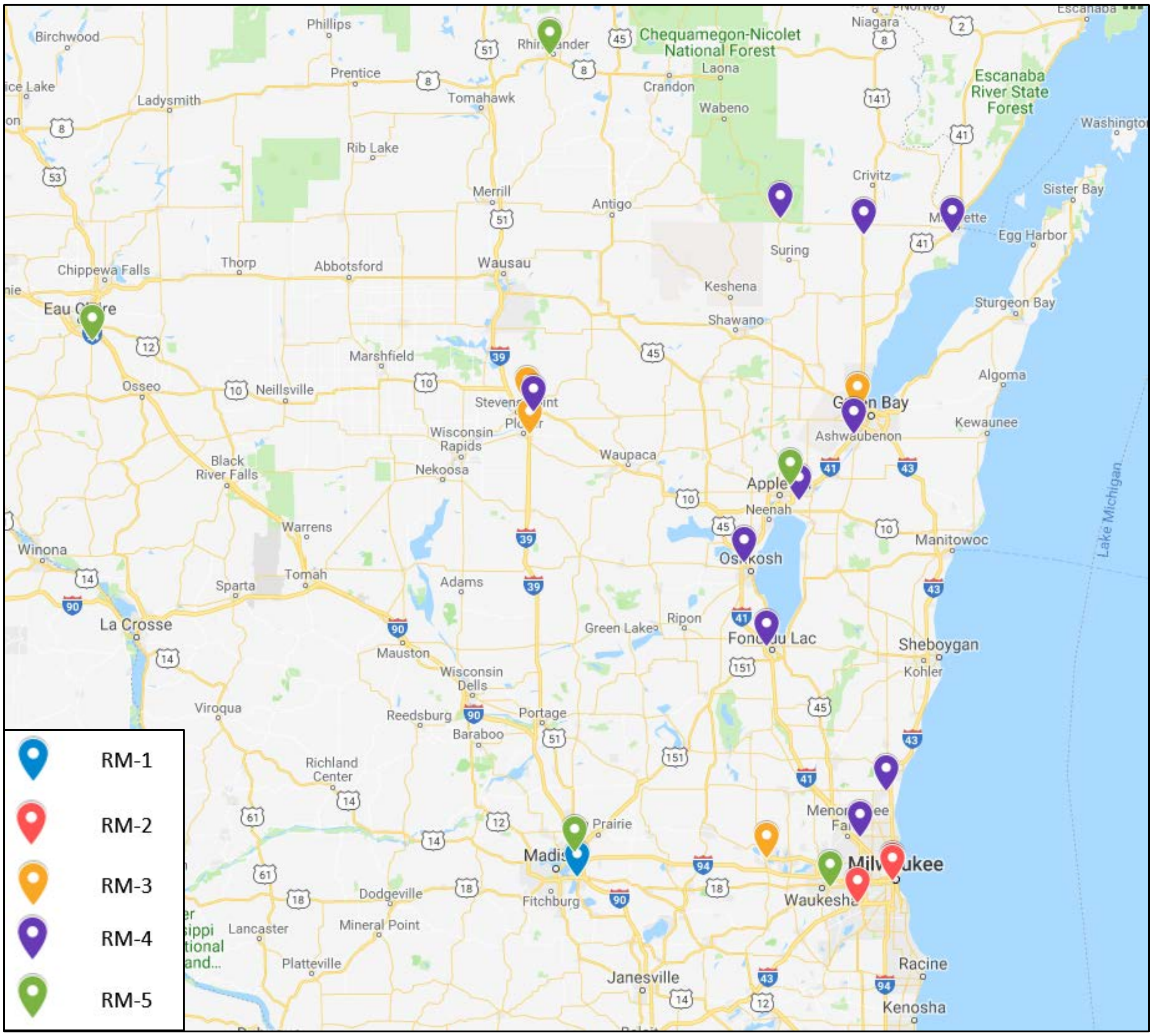


Figure 3-2. Sites for field investigations (image source: Google Maps).

Repair Area Preparation

While the procedures used to prepare the repair areas for the partial-depth repairs (PDRs) is not documented, personal communication with contractors and repair material manufacturers indicated that the repair areas were generally not demarcated using sawcuts (except for most of the RM-4 repairs and some RM-1 repairs). The unsound concrete (as determined through rough on-site sounding) was removed using jack hammers and the repair area was cleaned with compressed air before the material was placed. When specified by the material manufacturer, a bonding agent was applied to the repair area prior to the placement of the repair material.

Table 3-3. Field sites visited and summary of visual investigations.

Site ID	Location			Evaluation Date	Repair Age (years)**	No. of Cores Extracted	Condition of Repair Material	Condition of Surrounding PCC Pavement
	Route	County	City					
RM-1								
1	US 12/18 (Madison Beltline) EB	Dane	Madison	7/12/2018	3 to 4	3	**Fair to Excellent.** Several repairs were evaluated at this site and a vast majority of them exhibited no sign of surface distresses. Even though some repairs appeared to be sound on the surface, they could easily be removed from the pavement using a pry bar.	The surrounding concrete exhibited moderate amounts of distresses that included joint failures, spalling, transverse cracking, and corner breaks. Around many of the repaired areas, existing PCC continued to deteriorate.
RM-2								
2	I-43 Marquette Interchange*	Milwaukee	Milwaukee	10/2/2018	1	--	**Very Good to Excellent.** This site was a bridge deck location at the Marquette Interchange in Milwaukee. The material was used to place a 0.75-inch overlay on a bridge deck that exhibited moderate amounts of cracking and spalling. One isolated area (< 2 ft^2) exhibited shrinkage cracking.	N/A Entire deck has been overlaid using RM-2.
3	I-94 over Menomonee River*	Milwaukee	Milwaukee	10/2/2018	1	--	**Excellent (travel lanes), Poor (shoulder areas)**. Another bridge deck site on I-94 where the material was used to place a 0.75-inch overlay on a 10-year old deck. While the travel lanes exhibited no distress, the top layer (~0.25 inches) of the overlay in the shoulder area had delaminated. Upon discussions with the contractor, it was learnt that this issue was due to an inconsistent mix and the issue has been rectified since by complete removal and replacement of the overlay in the distressed areas.	N/A Entire deck has been overlaid using RM-2.

Table 3-3. Field sites visited and summary of visual investigations (continued).

Site ID	Location: Route	Location: County	Location: City	Evaluation Date	Repair Age (years)**	No. of Cores Extracted	Condition of Repair Material	Condition of Surrounding PCC Pavement
RM-2								
4	I-43 over W. Beloit Rd.*	Milwaukee	New Berlin	10/2/2018	1	--	**Excellent**. Material used for a 0.75-inch overlay on a 10- to 15-year-old bridge deck on I-43. After 1 year of service, material showed no signs of deterioration.	N/A Entire deck has been overlaid using RM-2.
RM-3								
5	W. Wisconsin Ave.	Waukesha	Oconomowoc	10/1/2018	3 to 4	1	**Fair to Excellent.** Material has been used to repair severely distressed joints at this location. Surface of the repaired area showed no signs of cracking. In a few areas, the material has chipped along the edges. The material has sunk into the repair hole (~0.75 inch) along some repair edges.	Surrounding concrete pavement has received significant amounts of joint repairs using RM-3.
6	Plover Rd.WB	Portage	Plover	10/3/2018	3 to 4	--	**Good to Excellent.** Material has been used to repair spalled joints at this location. The material itself appears to be holding up but the concrete pavement around the repaired area appears to be deteriorating.	Surrounding concrete pavement exhibited a fair amount of cracking and spalling. Joints were in poor condition.
7	S. Taylor St.	Brown	Green Bay	10/5/2018	1 to 2	--	**Excellent.** RM-3 has been used to repair spalled joints at this location. Material surface showed no signs of cracking.	Surrounding concrete pavement exhibited transverse cracking, spalling, and joint damage.
8	State Route 66 SB	Portage	Stevens Point	11/13/2018	2 to 3	1	**Fair.** RM-3 has been used to repair severely spalled joints (due to D Cracking) and deteriorated areas in the interior of the slab (due to freeze-thaw and scaling). The material shows some amount of surface wear. The pavement around the repaired area has continued to deteriorate significantly.	Surrounding concrete was in poor condition with significant amounts of spalling and D Cracking.

Table 3-3. Field sites visited and summary of visual investigations (continued).

Site ID	Location			Evaluation Date	Repair Age (years)**	No. of Cores Extracted	Condition of Repair Material	Condition of Surrounding PCC Pavement
	Route	County	City					
RM-4								
9	Washington St. EB/WB	Ozaukee	Grafton	9/26/2018	3 to 4	1	**Fair to Excellent**. Several small PDRs are located at this site. A vast majority of the PDRs exhibit little to no distress. Some edge deterioration and random cracking was observed on a few PDRs.	Surrounding concrete pavement exhibited moderate amounts of transverse cracking, spalling, corner breaks, and joint damage.
10	State Route 145 SB	Waukesha	Menomonee Falls	9/26/2018	3 to 4	--	**Poor to Good**. Two PDRs were evaluated at this site. One had a lot of missing material and the other was in relatively good condition.	Surrounding concrete exhibited significant amount of joint spalling.
11	US 10 EB	Portage	Stevens Point	10/3/2018	1	1	**Good to Excellent**. Several PDRs were evaluated at this location and most of them were in excellent condition. However, it should be noted that these PDRs have been in service for only just one year. The maintenance personnel mentioned three failures (of the several hundred PDRs installed in 2017) where the material had completely popped out.	Surrounding concrete exhibited moderate amounts of spalling and corner breaks.
12	US 41 SB	Marinette	Marinette	10/5/2018	2	--	**Good to Excellent**. Several joint and corner spalls have been repaired at this location using RM-4. The repairs have performed well over two winter seasons.	Surround concrete appears to be very old (>30-40 years). the pavement has received diamond grinding and dowel bar retrofitting. Moderate amounts of transverse cracking, spalling, and joint damage were noted.

Table 3-3. Field sites visited and summary of visual investigations (continued).

Site ID	Location			Evaluation Date	Repair Age (years)**	No. of Cores Extracted	Condition of Repair Material	Condition of Surrounding PCC Pavement
	Route	County	City					
RM-4								
13	State Route 32 NB/ SB*	Oconto	Mountain	10/5/2018	6	--	**Good to Excellent**. Ten partial-depth bridge deck PDRs were inspected at this location. The PDRs have been in service for around 6 years and appear to be performing very well. Minor surface abrasion was noted and a four PDRs exhibited a map cracking pattern on the surface.	Other than the areas repaired using RM-4, no surface cracking or distresses were noted on the bridge deck.
14	US 141 SB*	Marinette	Pound	10/5/2018	4 to 5	--	**Good to Excellent**. Two bridge deck approach slabs (SB direction) have been repaired at this location. The repairs are fairly substantial in size (full lane width, with average length of 3 ft.). After 4 to 5 years, of service under heavy traffic, the PDRs appear to be performing well. Minor surface map-cracking was noted on one PDR, but this defect does not appear to be compromising the structural integrity of the repair.	The surrounding concrete pavement appears to be in good condition and no distresses were noted. The repair at this location was isolated to the bridge deck approach slab.
15	Calumet St. EB/WB	Outagamie	Appleton	10/5/2018	< 1	--	**Excellent**. Several small spalls have been repaired at this location. The repairs are less than a year old and no failures have been noted. In some locations, the existing pavement has continued to deteriorate around the repaired area.	Moderate amount of joint spalling was noted in the surrounding concrete pavement.

Table 3-3. Field sites visited and summary of visual investigations (continued).

Site ID	Location			Evaluation Date	Repair Age (years)**	No. of Cores Extracted	Condition of Repair Material	Condition of Surrounding PCC Pavement
	Route	County	City					
16	W. Main Ave.*	Brown	Ashwaubenon	10/5/2018	7 to 8	--	**Fair to Good**. Several partial-depth deck repairs have been performed at this location. Some of the RM-4 PDRs are darker in color at this location and the material manufacturer attributed to the color anomaly to the poor clearing of the mixing equipment. Some PDRs exhibit a map-cracking pattern on the surface. These repairs have been in service for over 7 years.	The existing deck is in reasonably good condition. In a couple of isolated locations, the deck is showing some signs of deterioration around the PDR edges.
17	W. Johnson St.	Fond du Lac	Fond du Lac	10/5/2018	3	--	**Excellent**. RM-4 was used to repair a corner break in one isolated location. The repair shows no signs of deterioration after 3 years of service.	The existing PCC pavement is in very good condition and shows little to no distress.
18	I 41 SB*	Winnebago	Oshkosh	10/5/2018	1 to 2	--	**Good to Excellent**. Two large partial-depth repairs (12 ft x 3 ft) have been performed at this location on bridge deck approach slabs. While one PDR (right most lane) shows some surface wear, the other PDR (center lane) shows no sign of deterioration.	Other than the RM-4 PDRs, no other distress was noted in the surrounding concrete pavement.
RM-5								
19	State Route 93 NB	Eau Claire	Eau Claire	10/3/2018	1 to 2	--	**Good**. A large PDR (12 ft. x 2.5 ft.) was inspected at this location. Other than the surface wear, the PDR showed no other defects.	Surrounding PCC pavement was in generally good condition. Isolated areas of spalling and transverse cracking were noted.

Table 3-3. Field sites visited and summary of visual investigations (continued).

Site ID	Location			Evaluation Date	Repair Age (years)**	No. of Cores Extracted	Condition of Repair Material	Condition of Surrounding PCC Pavement
	Route	County	City					
RM-5								
20	N. Stoughton Rd. NB	Dane	Madison	10/3/2018	1 to 2	1	**Good**. A large PDR was performed on a slab that appeared to be severely distressed. Minor surface wear was noted.	The surrounding PCC pavement exhibits a fair amount of joint damage, spalling, and transverse cracking. Concrete around the edges of the PDR are beginning to deteriorate.
21	State Route 59 SB	Waukesha	Waukesha	10/3/2018	1 to 2	1	**Fair to Excellent**. At this location, RM-5 slabs with corner breaks and severe spalling. Other than the minor surface wear noted, the PDRs appear to be performing well.	The PCC pavement around the PDR edges are beginning to show some signs of deterioration. In general, the surrounding PCC pavement exhibits a fair amount of spalling and joint damage.
22	N. Ballard Rd. NB	Outagamie	Appleton	10/4/2018	1 to 2	--	**Good to Excellent**. RM-5 has been used to repair one transverse joint and one longitudinal joint at this location. Both repairs show no signs of damage other than minor surface wear.	The surrounding PCC pavement is in poor condition and almost all the joints exhibit some amounts of spalling.
23	W. Kemp St.	Oneida	Rhinelander	10/4/2018	1 to 2	--	**Good to Excellent**. RM-5 has been used to repair severe joint spalling at this location. Other than minor surface wear, the repair appears to be performing well.	The surrounding PCC pavement is in poor condition with severe joint spalling on almost every joint.

*Bridge Deck Sites

**Estimated age at date of inspection provided by material manufacturer representative

Table 3-4. Coring and PSPA testing summary.

Site ID	Core #	Repair Material	Average Ambient Temp, °F (°C)	Coring Summary			PSPA Testing Summary	
				Repair Material Thickness (inch)	PCC Thickness (inch)	Bond Condition	RM Dynamic Modulus (ksi)**	PCC# Dynamic Modulus (ksi)**
1	1	RM-1	87 (31)	2.50	8.50	Debonded	2,600	5,338
1	2	RM-1	88 (31)	2.25	7.75	Debonded	1,853	3,940
1	3	RM-1	83 (28)	3.00	N/A*	Debonded	2,280	4,808
5	1	RM-3	26 (-3)	3.00	N/A*	Debonded	NR*	4,633
8	1	RM-3	13 (-11)	1.50	N/A*	Debonded	1,670	4,010
9	1	RM-4	28 (-2)	2.50	7.25	Debonded	1,947	5,993
11	1	RM-4	12 (-11)	4.0	N/A*	Debonded	650	5,143
20	1	RM-5	23 (-5)	3.25	N/A*	Debonded	450	4,580
21	1	RM-5	24 (-4)	4.00	N/A*	Debonded	730	5,233

*Underlying PCC pavement was deteriorated; core could not be extracted
**Average of three readings recorded from one location
#Measurement performed on PCC pavement close to the PDR

As seen from table 3-4, in all the cores extracted, the repair material had debonded from the substrate concrete. The thickness of the PDRs varied from 1.5 to 4.0 inches. In most of the coring locations, the substrate concrete had deteriorated to such an extent that only loose material was found beneath the repair material. This is potentially indicative of the fact that non-cementitious materials had been placed in heavily deteriorated areas.

During the coring operation, the heat generated by the drill melted the binder (in all the repair materials), which gummed the core drill. Hence more water had to be used to cool down the material and the operation was much slower than a typical asphalt concrete/PCC coring process. Appendix A includes photographs of all the cores extracted.

The in situ dynamic elastic modulus of the concrete pavement around the repaired areas was estimated in the 4 to 6 million psi range (28 to 41 RM-5a). It should be noted that the quality of the concrete pavement in the areas where the PSPA testing was conducted could potentially be very different from the substrate concrete in the repaired areas. Since the substrate concrete was deteriorated in many of the coring locations, the modulus values may be significantly lower than the values reported in table 3-4.

The repair materials exhibited vastly different dynamic elastic modulus values as observed from table 3-4. RM-1 exhibited significantly higher modulus values [average value in excess of 2,200 psi (15 RM-5a)] when compared to the rest of the materials. It should be noted that the temperature of testing significantly impacts the modulus values for non-cementitious materials. These materials are viscoelastic in nature and are very flexible at warmer temperatures and tend to exhibit brittle behavior at colder temperatures. The modulus values documented in table 3-4 correspond to the testing temperature reported. Since these materials are expected to exhibit substantially different behavior at different temperatures, modulus testing (static and dynamic) was conducted in the laboratory to further study the dimensional stability of these materials at various temperatures (see Chapter 4). The following sections presents more information on the field performance of the various repair materials.

Repair Material-1 (RM-1)

RM-1 was used in about 8,000 partial-depth repairs (PDRs) on the Madison Beltline (USH 12/18) from Fish Hatchery Road to I-39 in 2014 and 2015. In 2014, 192,999 lbs. of the material were used to complete 3,206 PDRs and additional repairs were performed in 2015, consisting of 363,293 lbs. of material to complete another 4,644 PDRs. Initially, the product was noted to be performing extremely well, with almost no failures in the winter of 2014-2015. In January 2016, however, several instances of sudden failures were noted that led to vehicle damage and lane closures for emergency repairs. In order to investigate the potential causes of the failures observed, WisDOT extracted core samples from the eastbound left and center lanes on January 30, 2016. In many cases, the underlying concrete pavement beneath the cores were found to be in a deteriorated condition (Layton 2016).

Another round of failures occurred in January 2018. After the second occurrence of these failures within a span of 2 years, WisDOT decided to remove and replace all the 8,000 RM-1 PDRs with conventional PCC full-depth repairs.

Prior to the replacement of the RM-1 PDRs, the project team had the opportunity to conduct a field visit to document the condition of the PDRs and extract three cores in July 2018. The first core was extracted from an apparently sound PDR on the left most lane, close to a transverse joint. The thickness of the underlying pavement in this location was approximately 8.5 inches and the thickness of the PDR was around 2.5 inches (see figure 3-3).

Figure 3-3. RM-1 core location #1 (left) and extracted core #1 (right).

The PDR was debonded from the substrate concrete, but upon examination of the bottom of the debonded repair material (see figure 3-4), it was evident that the failure had occurred in the concrete and not in the repair material.

Figure 3-4. Close-up view of the bottom (left) and top (right) of RM-1 core #1.

The second core location was also in the left most lane, close to the longitudinal joint. The PDR showed no signs of distress on the surface. The thickness of the RM-1 PDR at this location was around 2.25 inches and the concrete pavement was approximately 7.75 inches thick (see figure 3-5). The RM-1 PDR was debonded from the substrate concrete. Similar to core #1, the debonded core had significant amounts of substrate concrete still bonded to it (see figure 3-6).

Figure 3-5. Coring operation in progress for core #2 (left) and the extracted core #2 (right).

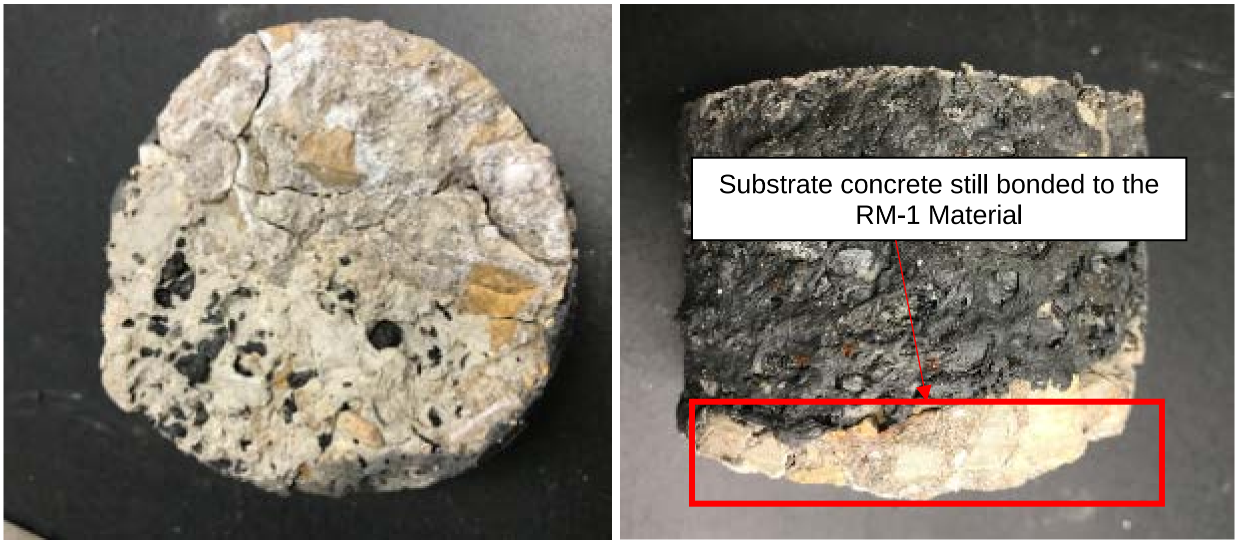


Figure 3-6. Close-up view of the bottom (left) and side (right) of the debonded portion from core #2.

A closer examination of the substrate concrete revealed multiple parallel horizontal cracks through the entire depth of the core (see figure 3-7) and these cracks extend through the coarse aggregates. This potentially suggests freeze-thaw-related damage, however, since petrographic analysis was not performed on the extracted cores, this speculation cannot be confirmed.

Figure 3-7. Multiple parallel horizontal cracks extending through the entire depth of core #2.

A third core was extracted from the same lane, on a PDR close to the transverse joint (see figure 3-8). The PDR did not exhibit any visual signs of deterioration. The core extracted from this location was completely debonded from the substrate concrete. The underlying concrete appeared to be completely deteriorated and only loose material was found in the core hole. The thickness of the PDR at this location was approximately 3 inches.

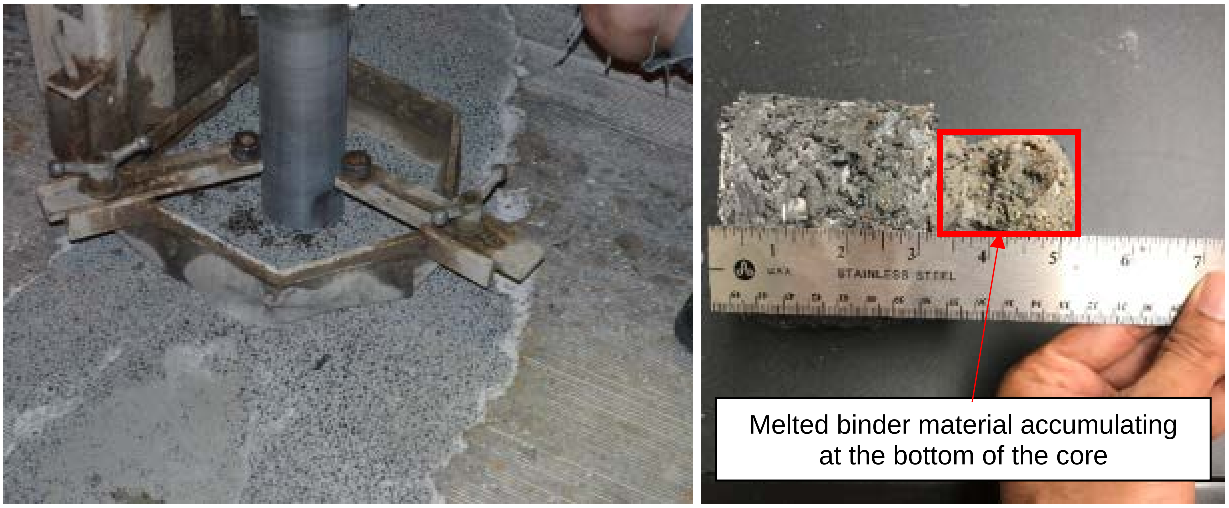


Figure 3-8. Location for core #3 (left), and extracted core (right).

Several other RM-1 PDRs were visually inspected at this site (site #1). On the surface, most of the PDRs showed no signs of damage (see figure 3-9) and the surface condition was noted to be Good to Excellent for most of the PDRs. However, some of the thinner PDRs simply peeled off from the surface (see figure 3-10) when a hydraulic pavement breaker was used to remove the RM-1 PDRs and prepare the repair areas for full-death repairs using conventional concrete material. Significant amount of substrate concrete material was still bonded to the bottom of the RM-1 PDRs that peeled off. Appendix B includes additional photos from the RM-1 field investigations.

Figure 3-9. RM-1 PDRs on the Madison Beltline.

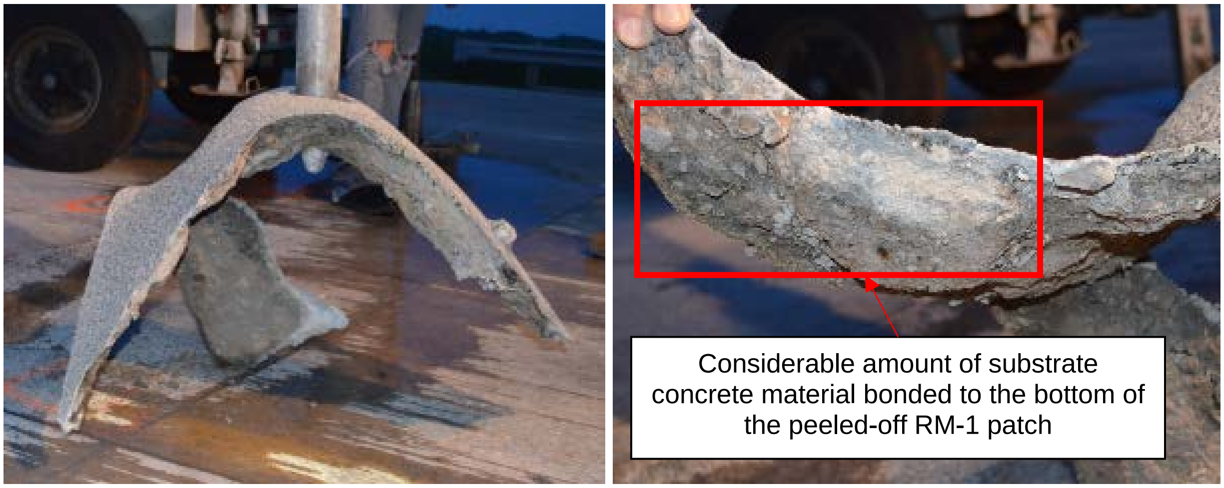


Figure 3-10. RM-1 PDRs peeling off from the concrete pavement.

The observations from the RM-1 site investigations seem to indicate that the reason for the failures may potentially due to the one of more of the following issues: (a) failure to remove all the unsound concrete prior to the placement of the PDRs, (b) continued deterioration of the substrate concrete material due to freeze-thaw cycles, (c) presence of residual deicing chemicals in the repair areas that could compromise bond quality, and (d) carbonation of the substrate concrete in the repair areas. All the issues listed are simply speculations at this point and additional field and laboratory investigations are needed to test these theories.

Repair Material-2 (RM-2)

WisDOT has not used RM-2 in concrete pavement PDR applications. However, the material has recently been used in a few bridge deck overlay projects near Milwaukee (see figure 3-11). The overlays are typically 0.75-inches thick and used to resurface bridge decks that exhibited moderate amounts of cracking and spalling (typical National Bridge Inventory rating of 5). Additional photographs from the RM-2 field investigations are available in Appendix C.

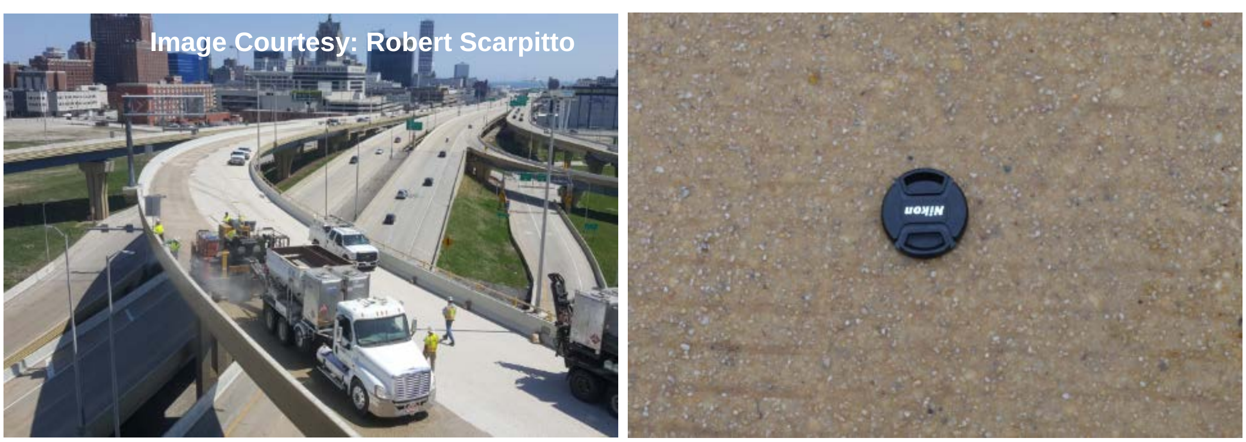


Figure 3-11. Bridge deck overlay using RM-2 in progress (left) and close-up of the RM-2 surface (right).

The RM-2 overlay was generally in Excellent condition on all of the bridge deck sites visited. The only exception was Site #3, where the top 0.25 inches of the overlay was peeling off from the deck. This deterioration was isolated to the edge of the decks (see figure 3-12). The material manufacturer has investigated the failure and deduced that the material was either under-catalyzed or the presence of moisture in the mix aggregates resulted in the failure noted. The contractor had agreed to remove and replace the deteriorated areas.

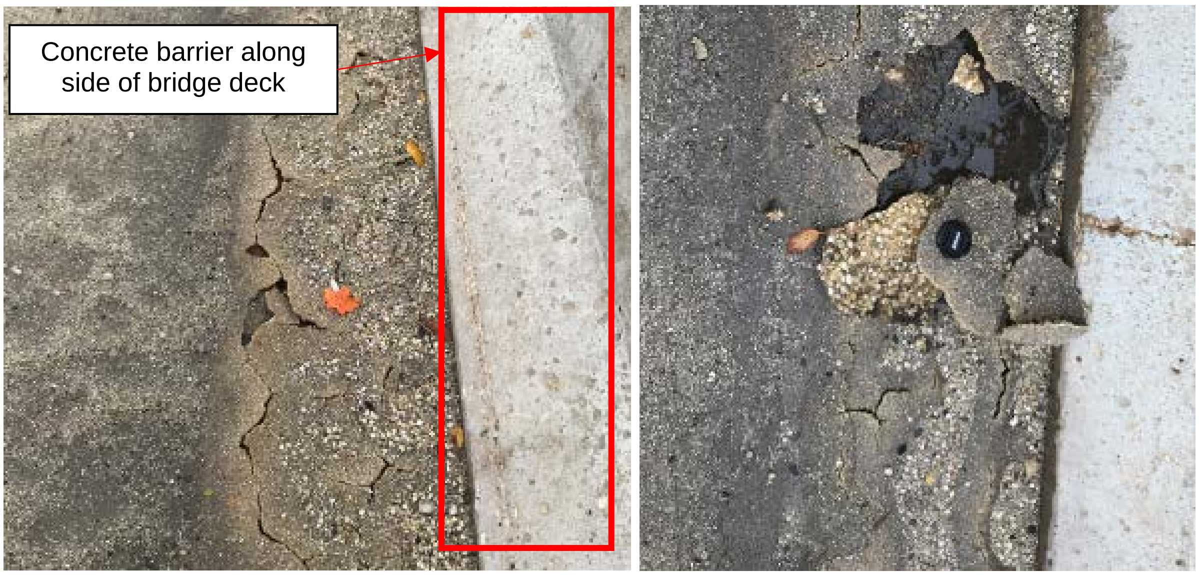


Figure 3-12. Top one-third of the RM-2 overlay peeling off along the deck edge (Site #3).

Another potential cause for the failures noted in figure 3-12 could be due to the accumulation of excess moisture (and potentially deicing chemicals) along the side barriers and exposure to freeze-thaw cycling, which can accelerate damage.

While the material has not been used in any partial-depth repair application in Wisconsin, the material manufacturer noted that Indiana and other states have used it in repair applications on both pavements and bridge decks.

Repair Material-3 (RM-3)

RM-3 appears to be a very popular repair material for asphalt pavements in Wisconsin and has been widely used in the South Eastern region. While the use on concrete pavements is not widespread, a number of WisDOT maintenance personnel noted good performance in concrete pavement joint and spall repair applications. In most of the sites visited, the repairs using RM-3 appear to be more of a stopgap fix until funding is available to perform a more permanent fix. Figure 3-13 presents some photographs of RM-3 used in concrete pavement PDR applications. Additional photographs from the RM-3 field investigations are available in Appendix D.

Figure 3-13. RM-3 used in concrete pavement PDR applications.

The field condition of the repaired areas was observed to be Fair to Excellent. The existing concrete pavement at Site #8 was heavily distressed. Severe joint spalling and scaling was widespread on this pavement. Also, many of the spalls and cracks noted on the pavement exhibited white exudate, which could be signs of other types of materials-related distresses Considering the poor condition of the areas that have been chosen as the candidates for RM-3 applications (particularly on Site #8), the material has exhibited reasonably good performance.

In areas where the cores were extracted (Sites #5 and #8), the underlying concrete pavement was deteriorated to such an extent that only loose material was found beneath the repair material. At Site #5, the extracted repair material deteriorated into multiple pieces even through the repaired area looked apparently sound on the surface. Photographs of the RM-3 Cores extracted are shown in figures 3-14 and 3-15.

Figure 3-14. RM-3 core from Site #5 (left) and
remains of underlying concrete pavement extracted from the core hole (right).

Figure 3-15. RM-3 core from Site #8 (left) and
pieces of underlying concrete pavement extracted from the core hole (right).

Repair Material-4 (RM-4)

RM-4 is the most popular non-cementitious repair material used in concrete pavement PDR applications in Wisconsin. The use of this material is particularly widespread in the North Central and North East regions of the state. Figure 3-16 presents some examples of RM-4 used in concrete pavement PDR applications. Additional photographs from the RM-4 field investigations are available in Appendix E.

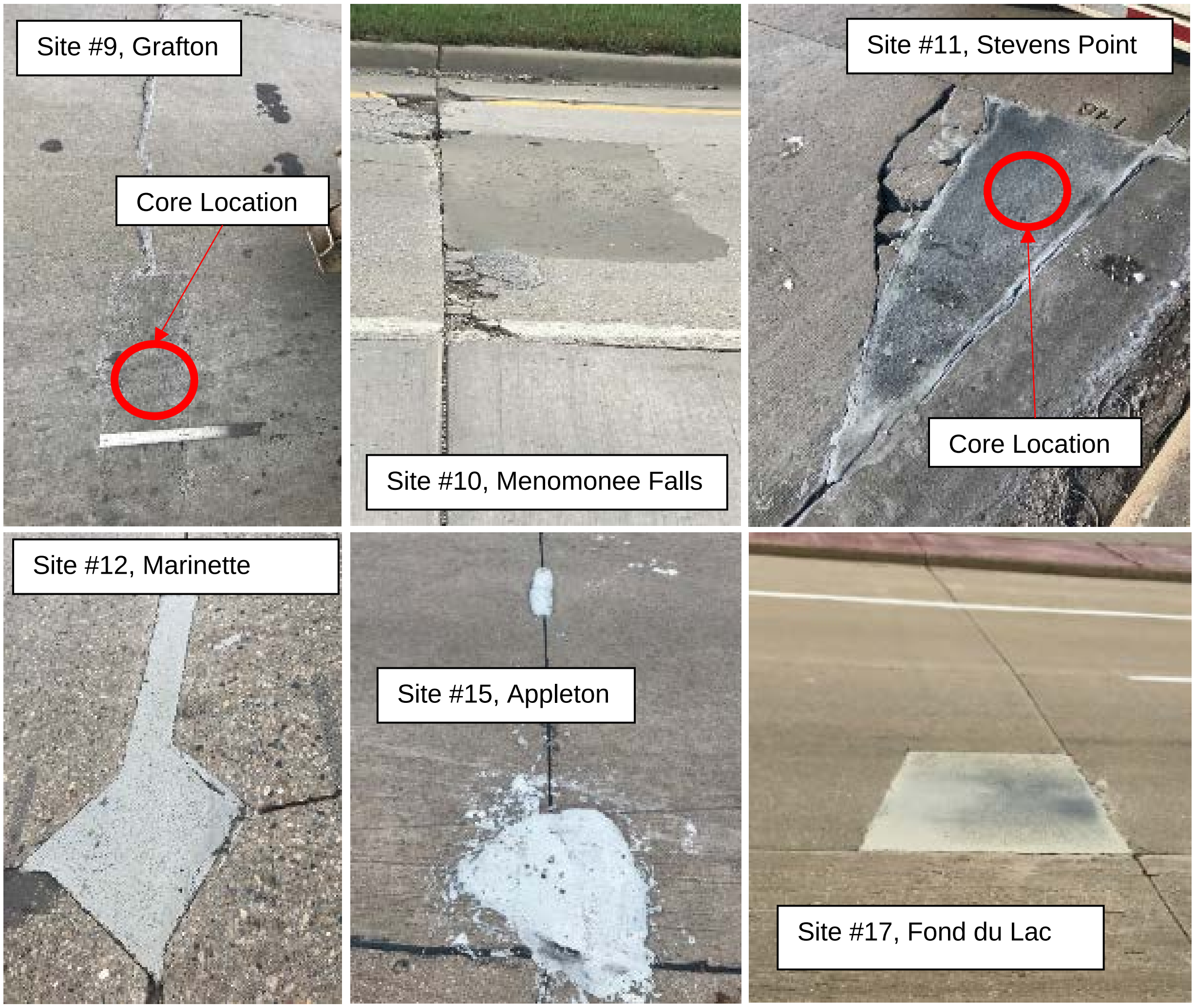


Figure 3-16. RM-4 used in concrete pavement PDR applications.

The most recent contract project for PDRs using RM-4 was performed in Stevens Point (Site #11). Several transverse cracks, spalls, corner breaks, and deteriorated joints along a 1.4 mile stretch of USH-10 were repaired in August 2017. In addition to the PDRs using RM-4, the same project also included a number of full-depth repairs using conventional concrete mixtures to address large areas that were severely deteriorated. The majority of the RM-4 PDRs at this location were still intact after one winter season. The local WisDOT maintenance personnel noted that three small PDRs (out of the several hundred PDRs installed in 2017) had failed and had to be filled with cold patch material as a stopgap fix.

At all the sites visited, the vast majority of the RM-4 repairs on concrete pavements were observed to be in satisfactory condition. In some repaired areas, the existing concrete pavement around the RM-4 PDR continued to deteriorate, however, no cracks or failures were noted in the repair material itself. Also, RM-4 appears to be the only material for which the repair boundaries were demarcated using saw cuts. Minimal preparation of the repair area appears to have been performed for the other repair materials evaluated in this study.

Two cores were extracted, one each from Sites #9 and #11 (core locations are shown in figure 3-16). RM-4 was used to repair a wide transverse crack at Site #9 and a corner break at Site #11.

The existing concrete around the repaired area at Site #11 has continued to deteriorate (see figure 3-16, top right). Figures 3-17 and 3-18 show photographs of the extracted cores.

Figure 3-17. RM-4 core from Site #9 (left) and
pieces of underlying concrete pavement extracted from the core hole (right).

Figure 3-18. RM-4 core from Site #11 (left) and bottom of the core showing
substrate concrete material bonded to the repair material (right).

RM-4 has also been used in several bridge applications in Wisconsin and figure 3-19 presents some examples.

Figure 3-19. RM-4 used in bridge applications.

The RM-4 repairs on bridge decks appear to be performing satisfactorily. In fact, some of the bridge deck PDRs have been in service for over 5 years without any failures. The surface of the PDRs on Site #16 (see figure 3-19, bottom left) have very different tints and appear to have been performed at different times and the repair boundaries have been sealed with a flexible sealant material. The material manufacturer claims that all the repairs on Site #16 were performed using RM-4 and attributed the difference in color to contaminants from the mixing equipment. The same mixing equipment (see figure 3-20) is used for both RM-3 and RM-4. The material manufacturer noted that if the equipment is not cleaned properly after using it to mix RM-3, the RM-4 mix produced using the same equipment is expected to have a darker shade.

Figure 3-20. Mixing equipment used for RM-3 and RM-4.

The material with the greyish tint has developed a map cracking pattern on the surface (see figure 3-21). The cracks are tight (no loose pieces found in the PDR) and resemble a shrinkage cracking pattern often seen on newly placed concrete.

Figure 3-21. Surface map cracking pattern observed on some RM-4 PDRs on Site #16.

Repair Material-5 (RM-5)

RM-5 has only been used in experimental demonstrations in Wisconsin at five sites and the project team was able to visit all these sites and document the prevailing condition of the repairs performed using this material. Figure 3-22 presents some examples of RM-5 used in concrete pavement PDR applications. Additional photographs from the field investigations are available in Appendix F.

Figure 3-22. RM-5 used in concrete pavement PDR applications.

The repaired areas were observed to be in generally Good condition. While the PDRs exhibited some surface wear from traffic, none of them developed cracking. In a few locations, a small amount of material was missing along the edge of the repaired areas, primarily due to the deterioration of the surrounding concrete pavement.

One core each was extracted from the Madison (Site #20) and Waukesha (Site #23) sites. At Site #20, the material was used to repair a heavily distressed transverse joint with severe spalling and cracking along the slab edges. Site #20 is located on US-51/N. Stoughton Road (Northbound lanes, see figure 3-23). Almost every single transverse joint is severely deteriorated at this location (especially the right lane) and has been repaired with either cold patch material or conventional hot mix asphalt. Considering the fact that the material was used to repair a severely distressed area, the performance after two winter seasons has been Good.

Figure 3-23. Overview of US 51/N. Stoughton Rd northbound lanes(top) and satellite imagery showing severely distressed condition of the north bound lanes.

Figures 3-24 show photographs of the extracted cores. At both the sites, the underlying concrete pavement was completely deteriorated and only loose material was found beneath the RM-5 material.

Figure 3-24. RM-5 cores from Site #20 (left) and Site #23 (right).

Summary

Key observations and findings from the field investigations are summarized below.

- In Wisconsin, non-cementitious repair materials are typically used to repair heavily distressed areas in concrete pavements, particularly spalled joints, transverse cracks, and corner breaks.
- For a vast majority of the repairs performed, the repair boundaries are not demarcated using saw cuts. The unsound concrete (as determined through on-site sounding) is removed using jack hammers and the repair area is cleaned with compressed air before the material is placed. RM-4 appears to be the only material for which many of the repair boundaries were cut before placement of the material.
- All the repair materials investigated were relatively flexible even at low temperatures [< 20 °F (<-7 °C)]; application of a small amount of force produced an impression on the material's surface. Coring of these materials was found to be challenging since the binder in these materials melts due to the heat generated from the friction between the core drill and the repair material. The melted binder gums up the coring drill and the whole operation required more water and was slower than a conventional coring operation on either asphalt or concrete materials. The "material melting" issue is a potential concern when performing operations like diamond grinding where the melted binder can "gum-up" the diamond-head blades. This issue has been acknowledged by the manufacturer of RM-4 and the following guidance is provided on addressing these issues:
  - Ensure that the surface of the material is covered with surfacing aggregates.
  - Reduce weight and time of the grinding operations. Heavy downward load applied by the grinding machine may remove too much material and this is to be avoided. The grinding head is to be floated over the surface of the repairs so that only the surface material is removed without creating excessive fins.
  - Perform the grinding operations during the coolest temperatures possible.
  - Maintain the grinding head as cool as possible.
- The abrupt failures of some RM-1 PDRs (where the entire patch popped out of the repair area) could potentially be attributed to the following factors:

- **Failure to remove all of the unsound concrete from the repair area**. In many cores extracted, a significant amount of substrate concrete material still appeared to be well-bonded to the bottom of the RM-1 cores. This may be an indication that the underlying deterioration or delamination in the existing concrete pavement was not adequately or completely removed prior to repair material placement.
- **Continued deterioration of substrate concrete**. The substrate concrete has continued to deteriorate, likely due to the factors that caused the original failures that required repairs using PDRs. The potential failure mechanisms include: (a) freeze-thaw damage, (b) damage due to excessive use of deicing chemicals (physical and/or chemical), and (c) carbonation of the substrate concrete. All these issues are likely to compromise the integrity of the quality of bond between the repair material and the substrate concrete
- **Inconsistency in the mixes produced on site**. For RM-1, bulking stone (included to increase the volume of the material) is added on site. If the proper field control is not exercised, it is possible to end up with an insufficient amount of binder at the bonding interface, a condition that could potentially compromise the bond between the substrate concrete and the repair material.

  The potential failure mechanisms noted above are merely speculative at this point and would require additional field and laboratory investigations for confirmation.

- RM-2 has not been used in any concrete pavement repair applications in Wisconsin, but several bridge decks in the Milwaukee area have been overlaid using this material. The overlays are generally distress free with the exception of one isolated location where the top one-third of the overlay has delaminated from the deck along the edges. This was attributed to improper mix design and/or presence of moisture in the aggregates.
- RM-3 appears to be a relatively popular repair material for asphalt pavements. The material is used to repair wide cracks and deteriorated longitudinal joints in asphalt pavements. In the few locations where it has been used for concrete pavement repair applications, WisDOT maintenance personnel noted better performance from this material when compared to conventional cold patch products.
- RM-4 appears to be the most popular non-cementitious repair material used in Wisconsin on concrete pavements. The material has seen widespread use in the North Central and North East regions of the state. In addition to PDRs on concrete pavements, RM-4 has also seen a good amount of use for bridge deck repair applications. Most of the RM-4 repairs were observed to be exhibiting Good performance.
- RM-5 has only been used in experimental demonstrations in Wisconsin. The material has been used in heavily distressed areas and has not experienced any failures after two winter seasons.

In summary, the field investigations indicate that majority of the failures observed are due to the deterioration of the substrate concrete and not the failure of the repair materials themselves.

CHAPTER 4. LABORATORY TESTING

Introduction

This chapter presents the details on the limited laboratory testing conducted to study the bond and dimensional stability aspects of three repair materials: RM-2, RM-4, and RM-5. The primary intent of the laboratory testing was to determine if the material exhibits significantly different behavior at different testing temperatures and if that can potentially be linked to the expected performance in the field.

Material Mixing

Prior to the commencement of the laboratory testing, a face-to-face meeting was conducted with each of the material manufacturer representatives to review the material handling and mixing procedures to be adopted in the laboratory. The details on the mixing procedures for the three materials evaluated in the laboratory are summarized in the following sections.

Repair Material-2 (RM-2)

Four components were required to prepare the RM-2 material: (a) binder resin, (b) catalyst, (c) accelerator, and (d) sand (see figure 4-1). All the materials were proprietary products, pre-packaged by the manufacturer.

Figure 4-1. RM-2 mix components (right)

A bucket and a handheld drill mixer (see figure 4-2) were used for the RM-2 material to prepare small batches using the following mix proportions:

- 84 oz. binder
- 2.5 oz. catalyst
- ¼ teaspoon accelerator
- 50 lb sand (one whole bag)

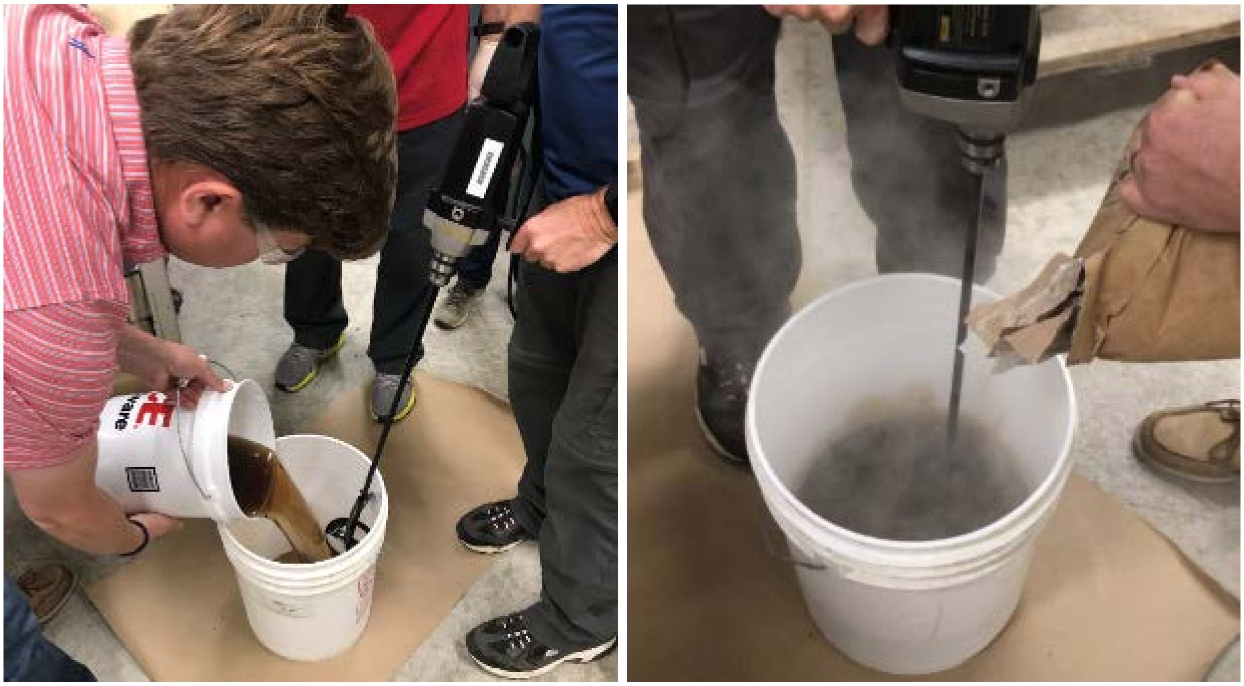


Figure 4-2. RM-2 mixing in progress.

Repair Material-4 (RM-4)

RM-4 arrives as a pre-packaged material (in 55-lb polyethene bags) in two formulations: (a) RM-4-R, which contains binder and fine aggregates for use in narrow (up to 4 inches) shallow repairs (up to 0.75 inches), and (b) RM-4-TBR, which contains binder and larger aggregate used in wider and deeper (up to 8 inches) repairs. All the field investigations were conducted on the TBR formulation and the same material was also used in the laboratory.

This material could not be mixed using a handheld drill mixer or other equipment typically used for asphalt materials. Hence, the material manufacturer furnished mixing equipment used in the field and assisted with the material mixing and specimen preparation process (see figure 4-3).

Figure 4-3. RM-4 mixing equipment (left) and specimen preparation (right).

Since RM-4 arrives as a pre-mixed product, no mixture proportioning is required. The entire bag of material is dumped into the mixing kettle and the material is continuously sheared using the mixing blades inside the kettle. The material is heated to a temperature of 375 to 400 °F (190 to 204 °C) and when the material reaches the desired level of consistency (should have no clumps and flow easily out of the kettle), it can be used for the desired application.

Repair Material-5 (RM-5)

RM-5 is another pre-packaged material delivered in a polystyrene package (see figure 4-4). The package includes both binder and aggregate. Due to its unique packaging, RM-5 was not suitable for mixing in the laboratory.

Figure 4-4. RM-5 material package (left) with
binder on one side (center), and aggregates on the other (right).

The material manufacturer representative brought special mixing equipment to the laboratory and assisted in preparing the mixtures (see figure 4-5). As with RM-4, no mixture proportioning was required since the RM-5 arrives as a pre-packaged product. The entire polystyrene package is dumped in to the mixer during the mixing process. As recommended by the manufacturer, the mixed material was stored in 10-gallon steel pails, which was then reheated and agitated to produce a consistent, flowable mixture prior to specimen preparation.

Figure 4-5. RM-5 mixing equipment (left) and
prepared mixtures stored in steel pails for specimen preparation (right).

Bond Testing

For bond testing, the pull-off method (as specified in ASTM C1583, *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)*) was used on repair material specimens cast in the laboratory. The bond testing was performed after the specimens were conditioned at temperatures of 23 ˚C, -23 ˚C, and -10 ˚C for 24 hours using environmental chambers. Testing at -23 ˚C is expected to represent the “worst-case” scenario because actual repair patches in the field would require prolonged exposure to such extreme temperatures before the materials reach that condition. Once the repair materials were cast onto the substrate concrete, they were allowed to air cure for 7 days before the temperature conditioning commenced.

Substrate concrete beams (3×6×20 inch) were first cast using typical concrete pavement mixture designs used in Wisconsin (see figure 4-6). The beams were moist-cured for 28 days. Once cured, the beams were air dried before the repair materials were installed on top of it. Each material manufacturer provided a proprietary bonding agent to use before placement of the repair material on the substrate concrete. The bonding agent was applied to the dry concrete substrate (which did not receive any surface preparations before or after curing) surface before the repair material was cast over it.

Figure 4-6. Substrate concrete beams prepared for bond testing (left) and repair material cast on top of substrate concrete beams (right)

Once the repair material on top of the substrate concrete beams had cured for 7 days, a 2-inch core drill was used to drill through the repair material to a depth of 1 inch into the substrate concrete beam. A 2-inch steel diameter steel disc with a screw-in type nut on the top was glued to the top of the repair material cores to facilitate the application of the tensile load. An additional period of 24 hours was allowed for the epoxy to cure before the temperature conditioning commenced.

All the testing was conducted inside the environmental chamber where the specimens were stored. The Proceq DY-225 automated pull-off tester (see figure 4-7) was be used to conduct the test. This equipment is capable of applying a pulling force of 5,620 lbf and can measure bond strengths in the working range of 185 to 1,847 psi. The tensile load was applied until failure and the maximum load as well as the location of the failure plane was noted.

Figure 4-7. RM-2 specimen prepared for bond testing.

There were a number of issues encountered during the bond testing process, and at the end the bond testing could successfully be performed only on the RM-2 specimens. Among some of the issues that were encountered include:

- At -23 °C, the glue used to stick the steel disc to the surface of the repair material failed to hold up during the bond-testing process and the bond between the steel disc and the material failed. Three different cold-temperature resistant glues were tried without any success. After several unsuccessful attempts (on 10 different specimens), eventually one successful test was performed.
- For RM-4 and RM-5, it was not possible to drill the 2-inch core hole successfully. As discussed in the previous chapter, the repair material melted during the coring process. When the core drill was removed from the specimen, the melted material filled up the hole and it was not possible to achieve smooth, uniform top surface which is essential for bond testing process (see figure 4-8).

Figure 4-8. Unsuccessful specimen preparation effort for bond testing.

- An alternate approach was attempted to circumvent the issues encountered above. Instead of attempting to core through the specimen to produce a circular surface for the pull-off bond testing, a series of saw cuts were performed through the cross-section of the specimen to produce a square surface at the top. Even this approach proved to be challenging. The heat from the saw-cutting process was melting the material. To address this issue, the specimen had to be conditioned to a cold temperature (by placing the specimen in an environmental chamber conditioned to -10 °C to arrest the melting after each saw cut was effected). Although relatively laborious, this approach was a success in terms of producing a specimen suitable for bond testing (see figure 4-9).

Figure 4-9. Alternate approach used to prepare specimens for bond testing.

- The bond testing on the RM-4 and the RM-5 specimens using the modified approach described had a number of other issues:
  - At a testing temperature of 23 °C, the material was just too soft and was unable to support the reaction forces during pull-out test (see figure 4-10).

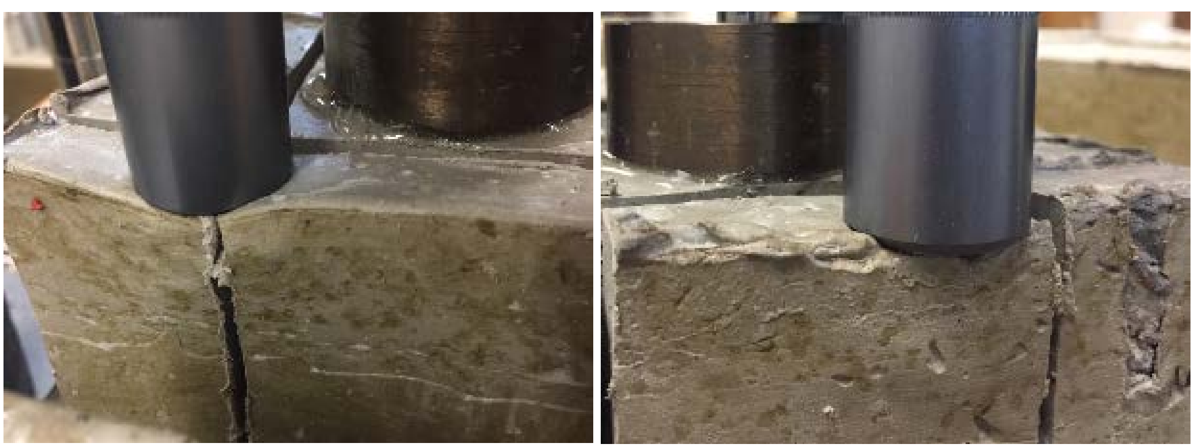


Figure 4-10. RM-4 specimens being crushed by the bond testing equipment at 23°C.

  - Since the materials become stiffer at colder temperatures, the research team was hopeful that the material would be able to provide adequate support for the bond testing equipment. Another set of specimens were prepared using the modified approach for testing at -10 °C and -23 °C. At the colder temperatures, even though the material was strong enough to support the testing equipment, the bond between the material and the steel disc failed first. Even the use of different cold-temperature resistant glues (as attempted for the RM-2 specimens) did not solve this issue.

- The research team attempted one last specimen prepared approach where the coring was performed from the concrete side of specimen (see figure 4-11). This approach was successful in terms of preparing the specimens for testing. The tests at 23 °C were not successful since the RM-4 and RM-5 specimens exhibit excessive creep at this temperature. The tests at -23 °C were not successful due to glue failures. Successful tests could only be performed at -10 °C and the results are summarized in table 4-1.

Figure 4-11. Specimens for bond testing prepared by coring through the concrete.

Due to the number of issues documented above, the bond testing effort was largely unsuccessful. Table 4-1 presents a summary of the test results.

Table 4-1. Bond strength test results.

Material	**Average Pull-off Bond Strength (psi)**		
	23 °C	**-10 °C**	**-23 °C**
RM-2	172*	291*	197**
RM-4	NR	159#	NR
RM-5	NR	126##	NR

NR: No Result, test was unsuccessful
*Average value of three specimens tested, failure within repair material at 23 °C and -10 °C
** Only one specimen resulted in a successful test and failure was observed at interface b/w PCC and repair material, bond between specimen and steel disc failed for other specimens tested
#Average value of two specimens tested (219 psi and 99 psi); pull-off test performed from concrete side.
##Only one specimen tested; pull-off test performed from concrete side.

For RM-2, the failure was observed within the repair material and not at the bonding interface for the testing temperatures of 23 °C and -10 °C (see figure 4-12). The bond between RM-2 and the substrate concrete appears to be stronger than the tensile strength of the material at these temperatures. While the failure plane was very close to the surface at a testing temperature of 23 °C, a comparatively deeper failure was observed at -10 °C. At -23 °C, only one successful test could be performed.

Figure 4-12. Failures observed after pull-off testing at
23 °C (left, shallow failures) and -10 °C (right, deep failures).

Static Elastic Modulus Testing

Static elastic modulus testing was performed in accordance with ASTM C 469, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete*. The same temperature conditioning regimen (23 ˚C, -23 ˚C, and -10 ˚C) used for the bond testing (discussed in the previous section) was also adopted for the static elastic modulus testing. Cylinder specimens (4-inch diameter by 6.9 inches tall) were fixed in a standard compressometer with an LVDT (linear variable differential transducer) and the specimen was loaded at a rate of 1 mm/min (0.04 inches/min). A continuous stress-strain response was captured.

Figure 4-13 presents the stress-strain response for RM-2 at the three testing temperatures and the average static elastic modulus values are summarized below:

- 9,840 MPa (1.4 million psi) at 23 °C
- 24,973 MPa (3.3 million psi) at -10 °C
- 24,662 MPa (3.6 million psi) at -23 °C

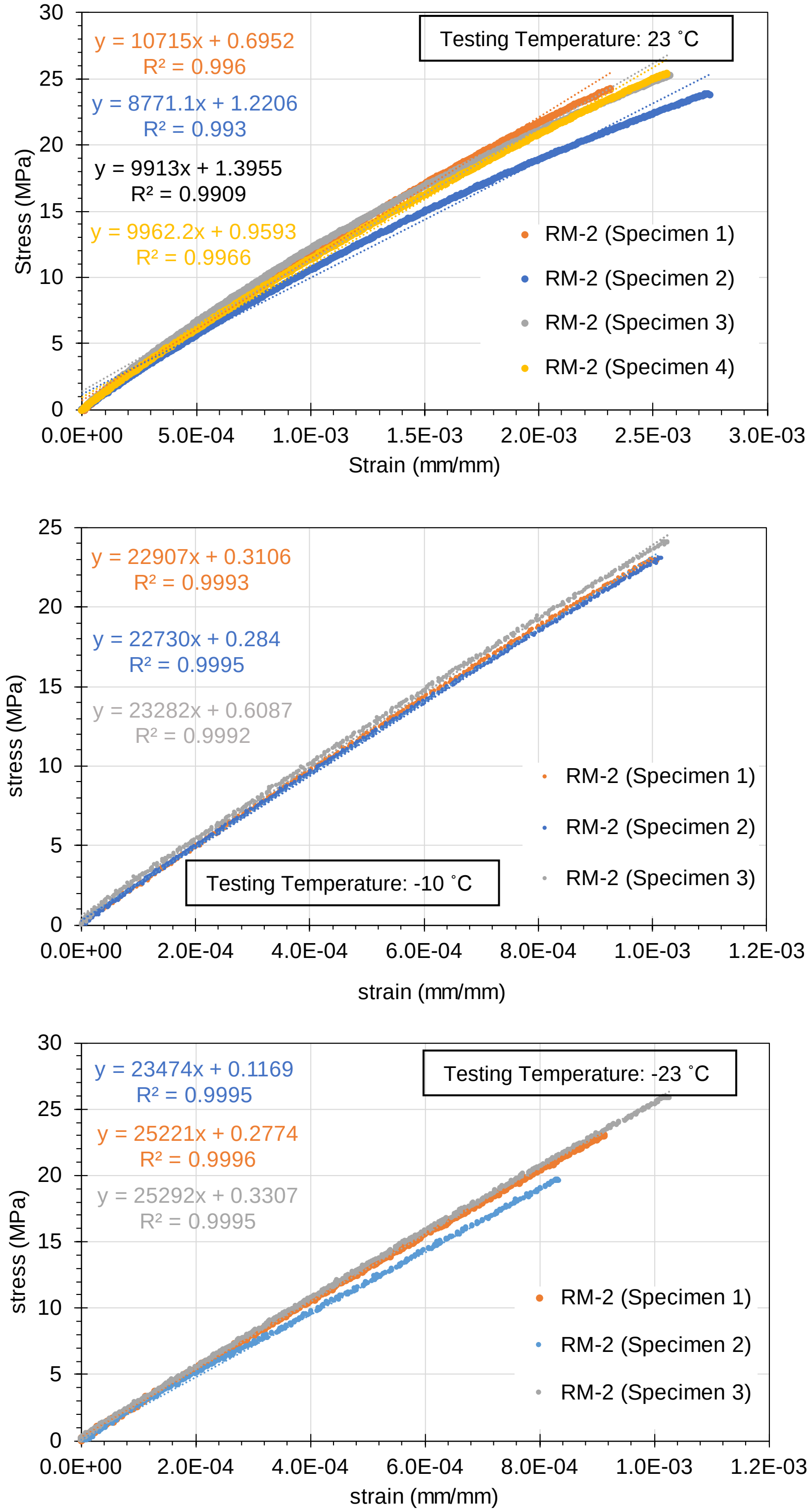


Figure 4-13. Stress-strain response curves for RM-2.

As seen in figure 4-13, RM-2 is much stiffer at the colder testing temperatures and the stress-strain response is linear. A slight non-linear response was observed at the higher testing temperature (23 °C). Dynamic modulus testing (discussed in the next section) was used to study the viscoelastic response of this material at various temperatures and loading frequencies.

Figures 4-14 and 4-15 present the stress-strain response curves for RM-4 and RM-5, respectively.

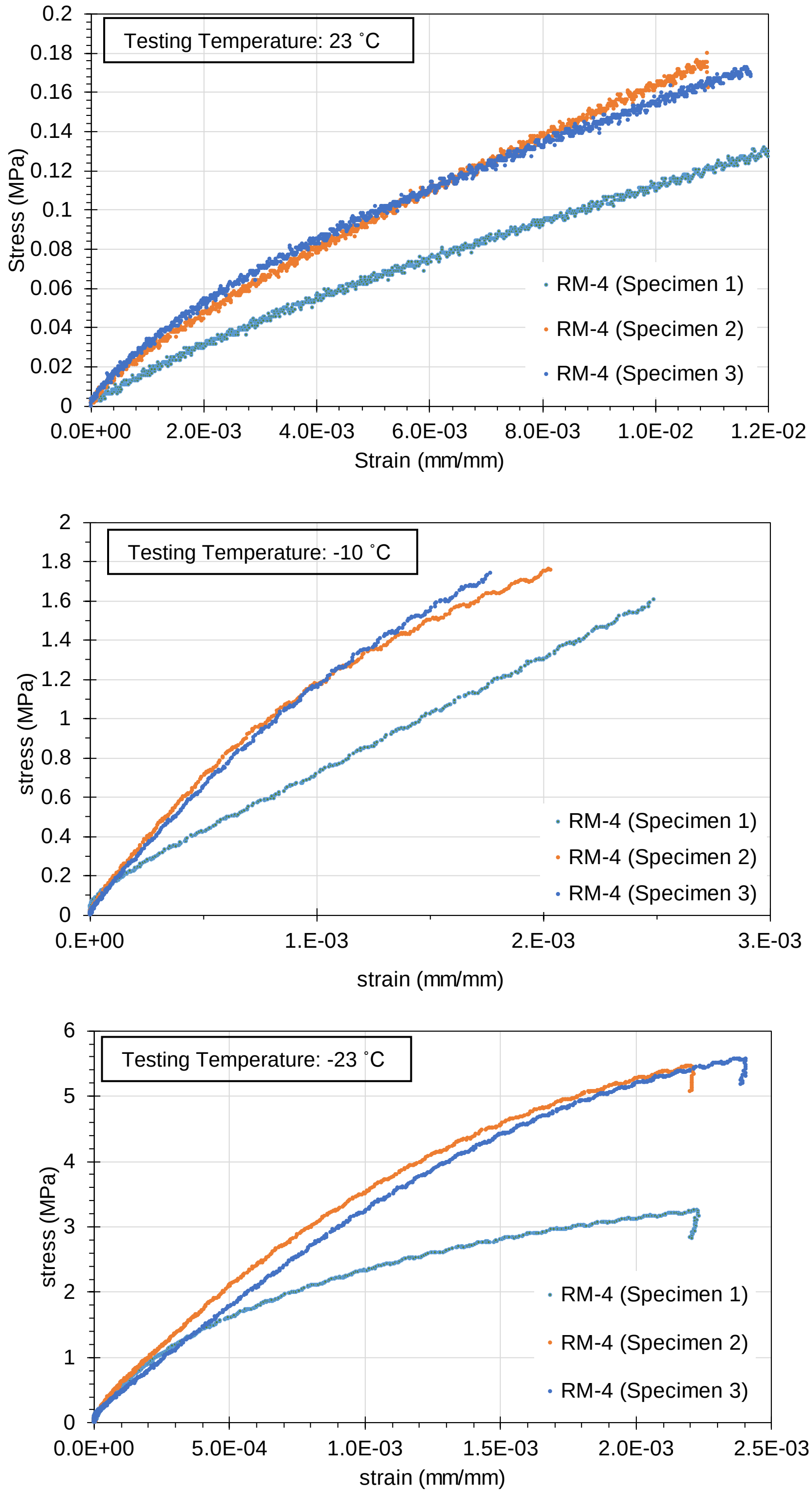
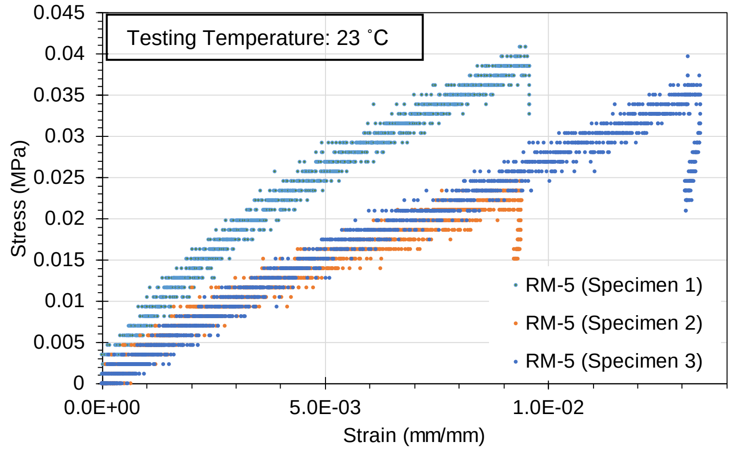
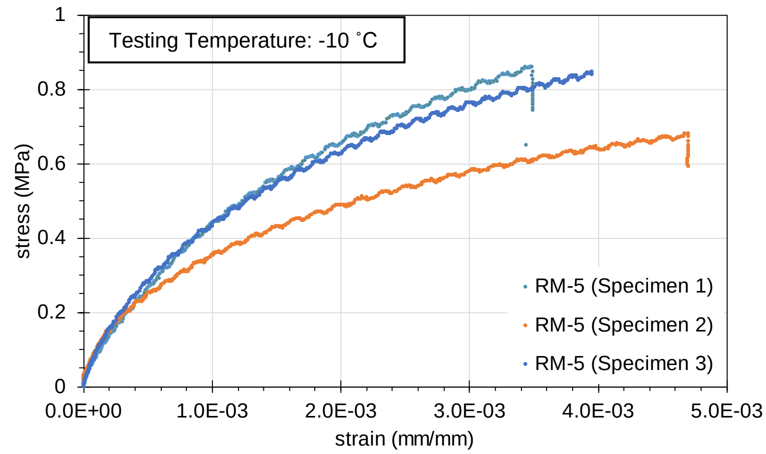


Figure 4-14. Stress-strain response curves for RM-4.





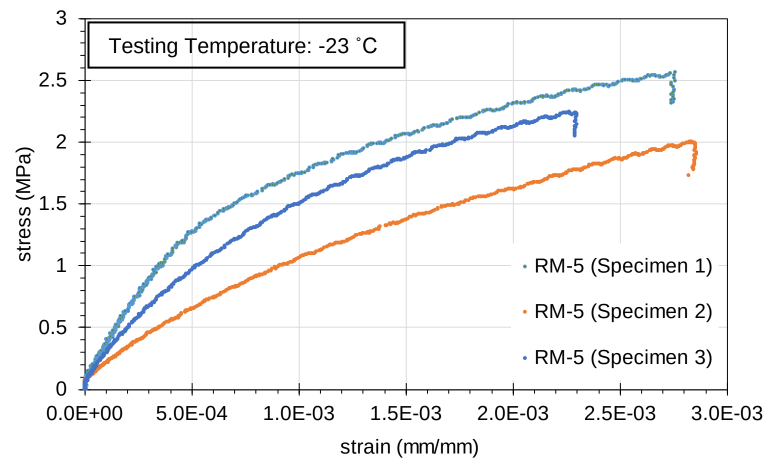


Figure 4-15. Stress-strain response curves for RM-5.

As seen from figures 4-14 and 4-15, both RM-4 and RM-5 exhibit non-linear stress strain response at all the testing temperatures. The test specimens were simply being “squished” under the compressive loading and then when the load was removed, they almost returned back to the original shape/dimensions. Hence, the static elastic modulus test is deemed to be unsuitable for characterizing the fundamental stress-strain response for these materials. The dynamic modulus test (typical performed on asphalt concrete specimens) was hypothesized to be a more suitable test for these materials (discussed in the next section).

Dynamic Elastic Modulus Testing

The dynamic elastic modulus (|E*|) test is typically performed on materials exhibiting viscoelastic behavior (such as polymers and asphalt concrete). The stiffness of these materials tends to vary with time, temperature, and loading frequency. To obtain the stiffness properties over a range of typical operating (application) temperatures and loading frequencies (vehicular traffic), this test is conducted in accordance with AASHTO T342. A sinusoidal compressive stress is applied to the specimen and the corresponding axial strain is measured at each test temperature and frequency. A lag in the strain response gives a measure of the viscoelastic nature of the material, as defined by a phase angle (δ). A low value of phase angle indicates quick response or more elastic behavior, whereas a high value of phase angle indicates more viscous behavior. Then, using time-temperature superposition, the stress-strain response at each temperature and frequency are combined to generate a “mastercurve”, which represents the complete response of the material over a range of temperatures and frequencies.

For this study, 100-mm diameter (~4 inch) x ~170-mm (~6.8 inch) height cylindrical specimens were prepared. In the case of RM-2 samples, the test temperatures were -10, 4, 21 and 37 °C. RM-5 and RM-4, being significantly softer, could only be tested at -10, 4 and 21°C. Figures 4-16 through 4-18 show the dynamic modulus versus the testing frequency at different testing temperatures for RM-2, RM-4, and RM-5, respectively.

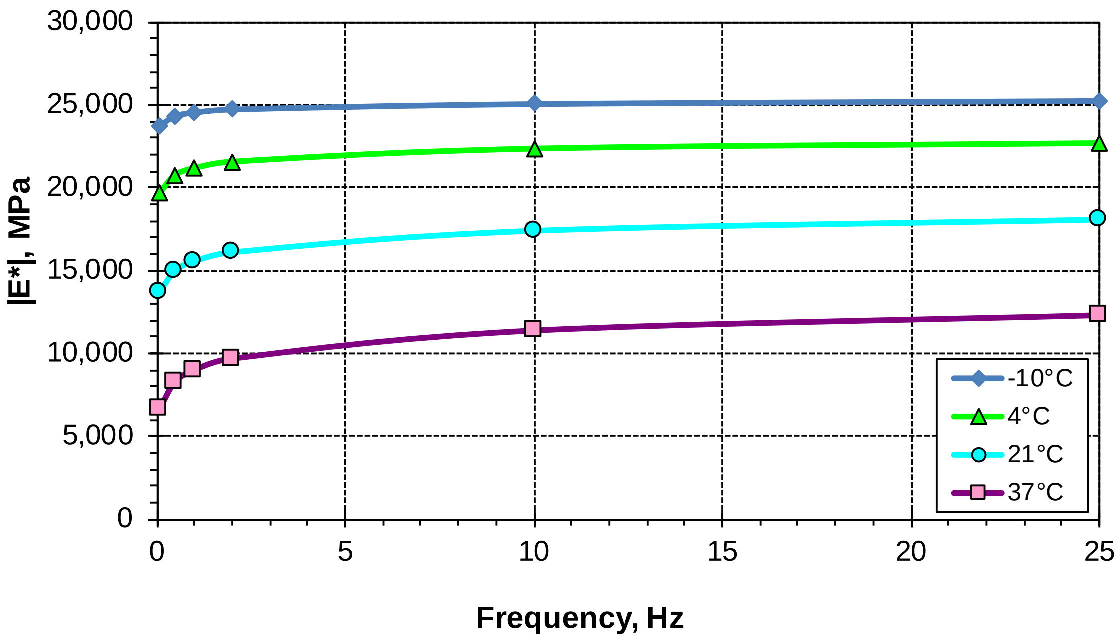


Figure 4-16. Dynamic modulus vs. testing frequency for RM-2.

The very low curvature of the plot at -10 °C in figure 4-16 indicates that RM-2 shows almost rigid, non-viscous behavior at subzero temperatures. The slight curvature of the plots at the other testing temperatures indicates that its viscous component is being activated.

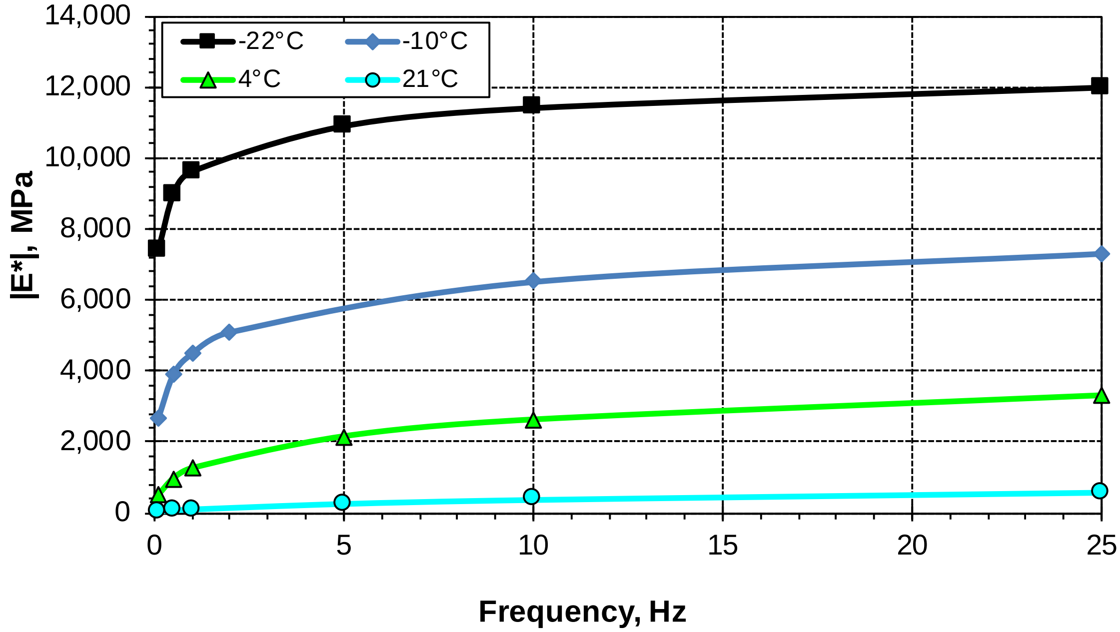


Figure 4-17. Dynamic modulus vs. testing frequency for RM-4.

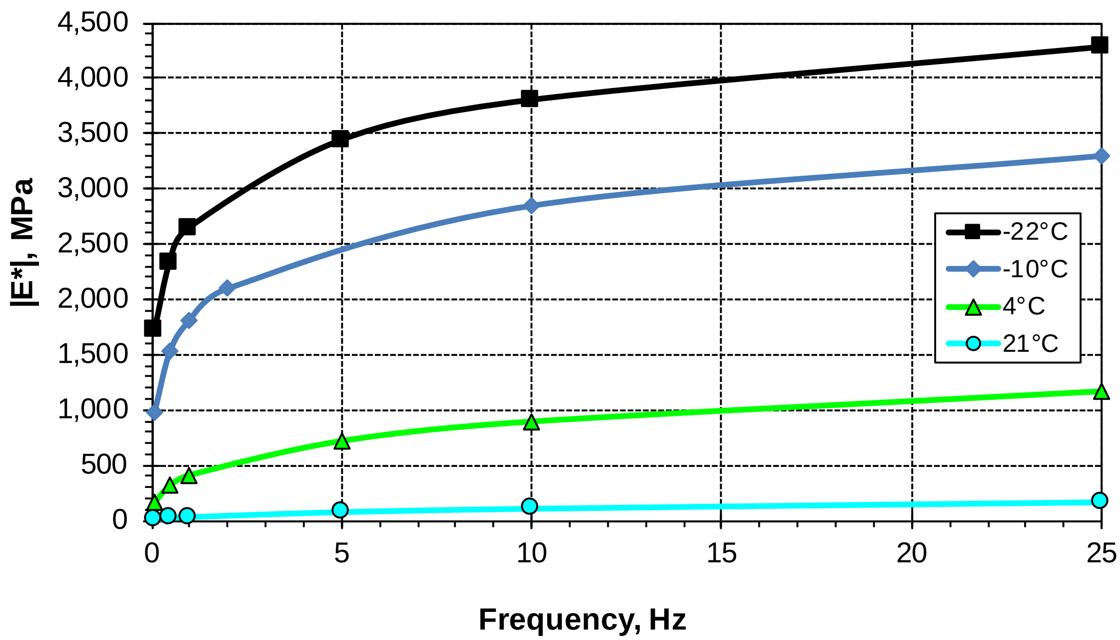


Figure 4-18. Dynamic modulus vs. testing frequency for RM-5.

As seen in figures 4-17 and 4-18, the dynamic modulus of RM-4 and RM-5 varies significantly with temperature and loading frequency. The dynamic modulus of RM-5 is significantly lower than RM-4 even at the colder testing temperatures when these materials become relatively stiffer.

Figure 4-19 shows the mastercurves of the three materials tested along with that of a typical hot-mix asphalt mixture (9.5 mm nominal maximum aggregate size surface mixture).

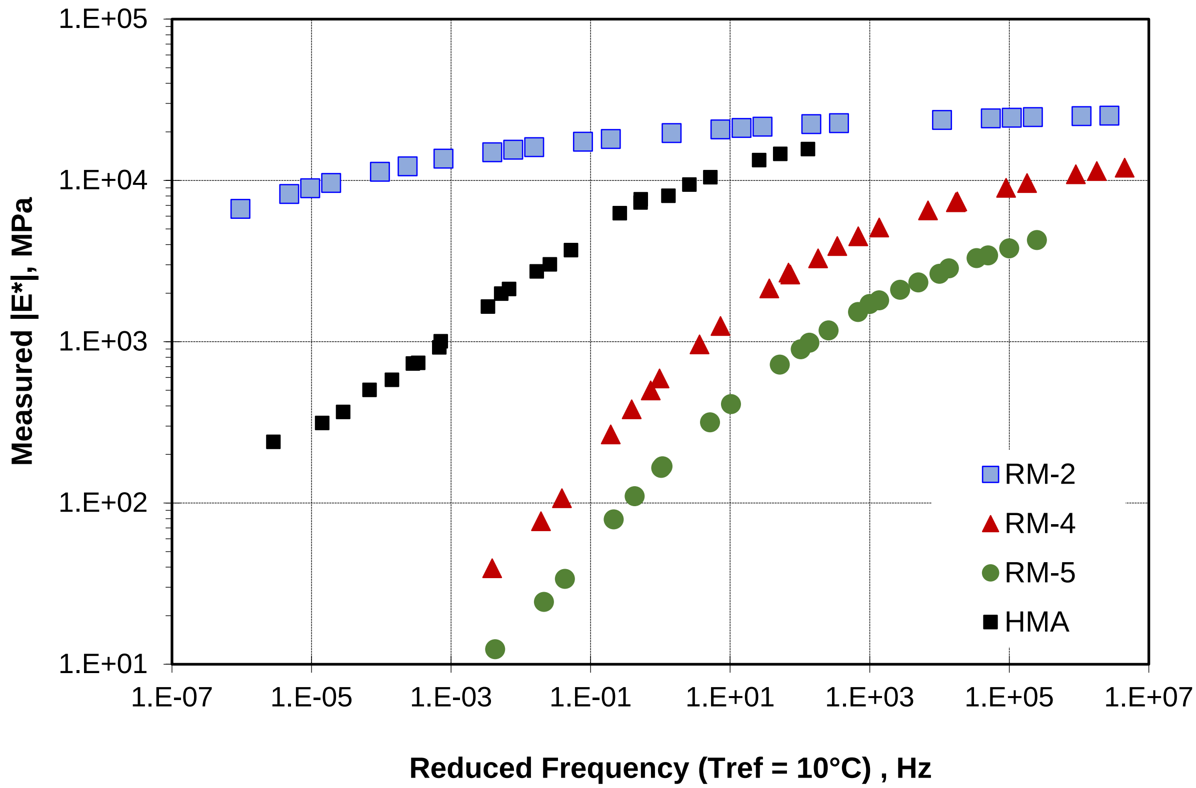


Figure 4-19. Mastercurves developed from dynamic modulus testing.

The modulus of RM-2 at lower temperatures (i.e., higher frequencies) is comparable to that of asphalt, while that of RM-5 and RM-4 is significantly lower. At lower temperatures, all the materials become relatively stiffer, however, the modulus values are still lower than conventional HMA. RM-5 and RM-4 appear to be very sensitive to changes in temperature and loading rate. The low modulus of these two materials indicates that they may be prone to rutting/permanent deformation if exposed to very high temperatures. At very low temperatures, the modulus of RM-4 and RM-5 starts to reach an asymptotic value, running parallel to that to RM-2 and HMA but at a lower modulus value. The lower modulus of RM-4 and RM-5 when compared to RM-2, suggests higher flexibility, and thereby improved ability to withstand thermal stresses at lower temperatures.

Table 4-2 summarizes the dynamic modulus data for all the materials tested and table 4-3 provides a summary of the single factor Analysis of Variance (ANOVA) testing to test whether the results are significantly different from one another. Based on the ANOVA results (summarized in table 4-3), it can be concluded that the three materials evaluated exhibit significantly different dynamic modulus properties.

Table 4-2. Summary of dynamic modulus test results.

Test Condition	**25 Hz and -22°C**		**10 Hz and -22°C**	
Material	**RM-4**	**RM-5**	**RM-4**	**RM-5**
|E*|, MPa	13,204	3,428	12,685	2,983
	11,433	4,801	10,846	4,291
	11,377	4,601	10,742	4,118
Mean	***12,005***	***4,277***	***11,424***	***3,797***
Std. Dev.	***1,039***	***7,42***	***1,093***	***711***

Test Condition	**25 Hz and -10°C**			**10 Hz and -10°C**		
Material	RM-2	RM-4	RM-5	RM-2	RM-4	RM-5
|E*|, MPa	24,681	7,292	3,497	24,406	6,477	3,019
	24,892	7,207	2,944	24,664	6,425	2,524
	26,139	7,463	3,467	26,074	6,674	3,011
Mean	***2,5237***	***7,321***	***3,303***	***25,048***	***6,525***	***2,851***
Std. Dev.	***788***	***130***	***311***	***898***	***131***	***284***

Test Condition	**25 Hz and 4°C**			**10 Hz and 4°C**		
Material	RM-2	RM-4	RM-5	RM-2	RM-4	RM-5
|E*|, MPa	22,337	3,557	1,306	22,040	2,835	1,004
	22,132	3,217	946	21,800	2,556	713
	23,607	3,097	1,274	23,237	2,443	983
Mean	***22,692***	***3,290***	***1,175***	***22,359***	***2,611***	***900***
Std. Dev.	***799***	***239***	***199***	***770***	***202***	***162***

Test Condition	**25 Hz and 21°C**			**10 Hz and 21°C**		
Material	RM-2	RM-4	RM-5	RM-2	RM-4	RM-5
|E*|, MPa	17,441	692	140	16,789	450	91
	17,467	548	178	16,772	351	118
	19,350	536	189	18,643	341	122
Mean	***18,086***	***592***	***169***	***17,401***	***381***	***110***
Std. Dev.	***1,095***	***87***	***25***	***1,075***	***60***	***17***

Table 4-3. Single factor ANOVA results ($\alpha = 0.05$).

Frequency	Temperature	Null Hypothesis	p-value	Conclusion
25 Hz	-22°C	$\mu_{RM-4} = \mu_{RM-5}$	0.0005	Stat significant
	-10°C	$\mu_{RM-2} = \mu_{RM-4} = \mu_{RM-5}$	5.76e-09	Stat. significant
	4°C	$\mu_{RM-2} = \mu_{RM-4} = \mu_{RM-5}$	5.26e-09	Stat. significant
	21°C	$\mu_{RM-2} = \mu_{RM-4} = \mu_{RM-5}$	5.63e-08	Stat. significant
10 Hz	-22°C	$\mu_{RM-4} = \mu_{RM-5}$	0.0005	Stat. significant
	-10°C	$\mu_{RM-2} = \mu_{RM-4} = \mu_{RM-5}$	9.58e-09	Stat. significant
	4°C	$\mu_{RM-2} = \mu_{RM-4} = \mu_{RM-5}$	3.68e-09	Stat. significant
	21°C	$\mu_{RM-2} = \mu_{RM-4} = \mu_{RM-5}$	6.05e-08	Stat. significant

Ultrasonic Pulse Velocity Testing

Ultrasonic pulse velocity (UPV) measurements through the hardened repair material was performed after the specimens were conditioned at temperatures of 23 ˚C, -23 ˚C, and -10˚C. Three specimens were tested in accordance with ASTM *C, 597 Standard Test Method for Pulse Velocity through Concrete* at each temperature using 4-inch (diameter) × 7-inch (length) cylinder specimens. In addition to the UPV testing on the repair materials, the same test was also be conducted on "base" concrete specimens that were prepared using typical paving mixtures used in Wisconsin. These specimens were moist-cured for 28 days before the testing was performed and can be used as a baseline for comparison with the UPV values for the repair materials. The base specimens were cast as 3-inch x 4-inch x 16-inch prisms for the UPV testing on the concrete specimens. Table 4-4 provides the results of the UPV testing, presented with the static elastic modulus (SEM) and dynamic modulus (DM) values.

Table 4-4. Summary of UPV and modulus testing results.

Testing Temperature	RM-2			RM-4		RM-5		PCC
	UPV (m/s)	DM (MPa)	SEM (MPa)	UPV (m/s)	DM (MPa)	UPV (m/s)	DM (MPa)	UPV (m/s)
23 °C	3,378	17,401	9,840	2,730	381	2,488	110	4,745
-10 °C	3,589	25,048	22,973	3,189	6,525	2,865	2,851	4,734
-23 °C	3,619	N/A	24,662	3,381	11,424	3,105	3,797	4,647

DM: Dynamic Modulus at 10 Hz
SEM: Static Elastic Modulus; reported only for RM-2 since the test was not suitable for RM-4 and RM-5; test not performed on PCC

The UPV values for each of the three repair materials increased with decreasing temperature. This trend is expected since the modulus testing showed that the material becomes stiffer at colder temperatures. The microstructure is expected to become denser, which will naturally facilitate the faster movement of the ultrasonic pulse waves through the material's mass. The pulse velocity for the concrete specimens remains fairly constant at all the testing temperatures and this observation is also expected.

Figure 4-20 shows a plot of the UPV values versus the testing temperature. A strong correlation is observed for each of the three repair materials. The UPV values also showed strong correlation with the modulus values measured at each of testing temperatures (see figure 4-21).

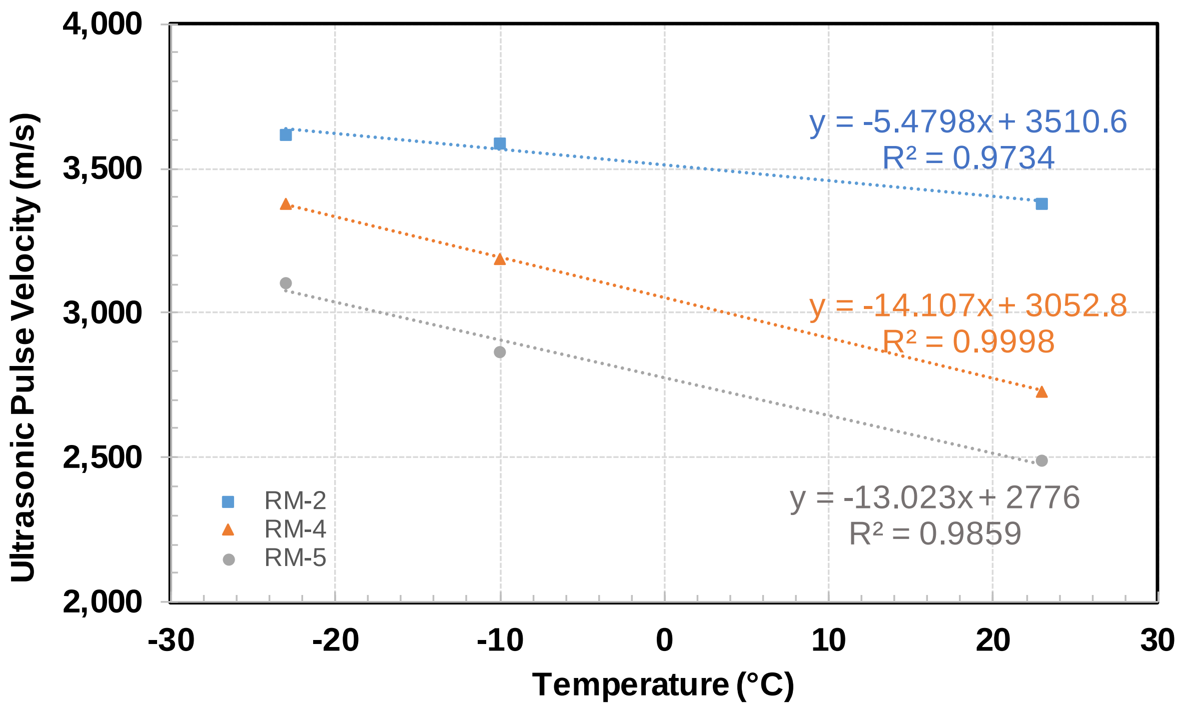


Figure 4-20. UPV vs. testing temperature.

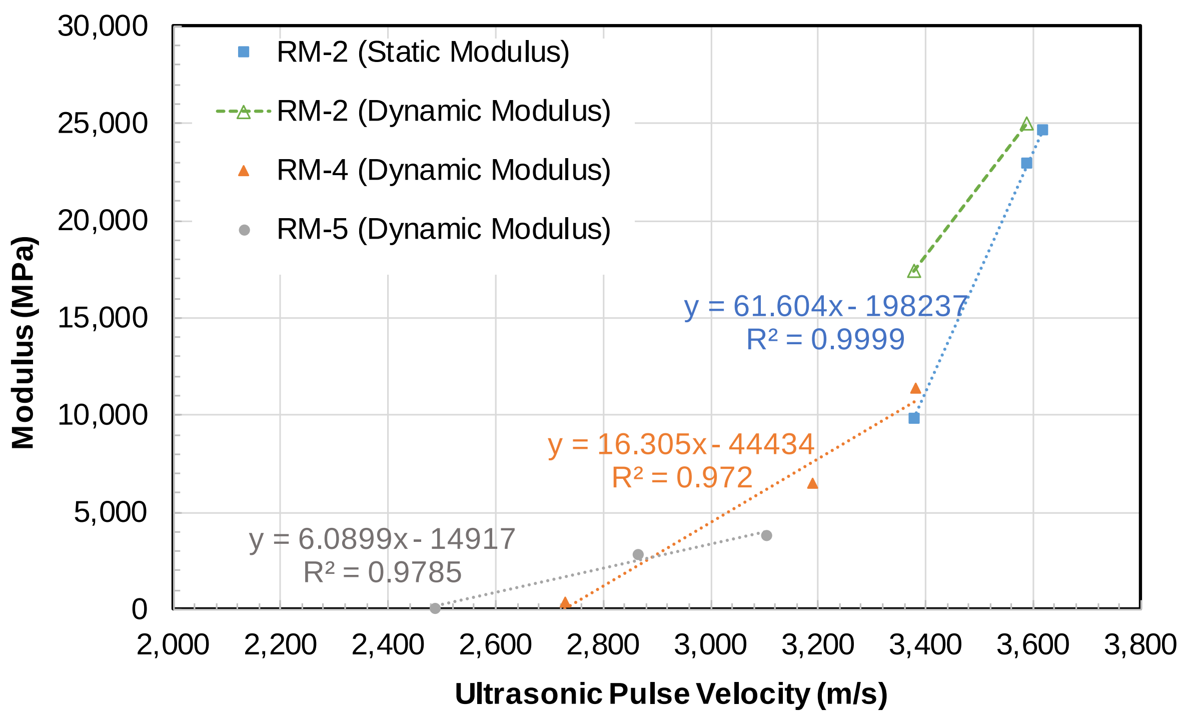


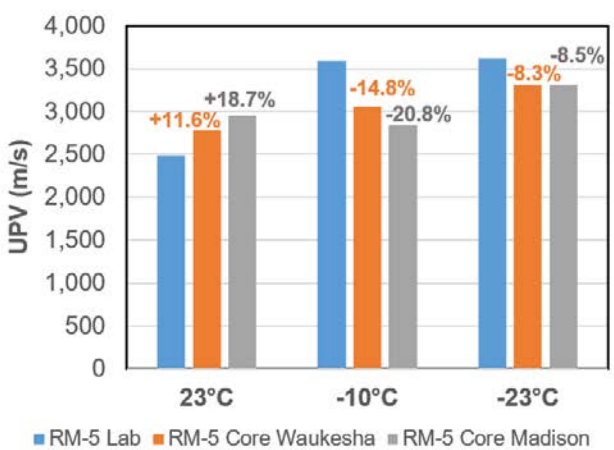
Figure 4-21. UPV vs modulus.

UPV testing was also attempted on the intact cores extracted during the field evaluations. Some of the cores were deteriorated beyond the point where any successful UPV measurements could be performed. The successful UPV measurements on the core specimens are summarized in table 4-5.

Table 4-5. UPV testing summary.

Site ID	Core #	Location	Material	UPV (m/s)		
				23 °C	-10 °C	-23 °C
1	1	Madison	RM-1	NR	NR	NR
1	1	Madison	PCC	4,699	4,562	4,671
1	2	Madison	RM-1	2,966	3,185	3,233
1	2	Madison	PCC	1,345	1,307	Core Broke
1	3	Madison	RM-1	2,857	3,070	3,438
5	1	Oconomowoc	RM-3	NR	NR	NR
8	1	Stevens Point	RM-3	NR	NR	NR
9	1	Grafton	RM-4	NR	NR	NR
11	1	Stevens Point	RM-4	3,012	3,505	3,589
20	1	Madison	RM-5	2,955	2,842	3,312
21	1	Waukesha	RM-5	2,778	3,057	3,318
NR: No Result						

It is worth noting the very low UPV value for the substrate concrete core specimen extracted from Site #1 (Core #2). This measurement was on the concrete core that exhibited parallel cracking throughout the entire depth (see figure 3-7.) Interestingly, this core specimen could not withstand the -23 °C temperature and disintegrated before the UPV measurement could be made. This observation further supports the findings from the field investigations that the PDRs are often performed in areas where the substrate concrete is already weak and freeze-thaw cycling during the winter season likely contributes to further deterioration of the substrate concrete and eventually result in the failure of the PDR. Figure 4-22 compares the UPV values measured on the laboratory and core specimens.



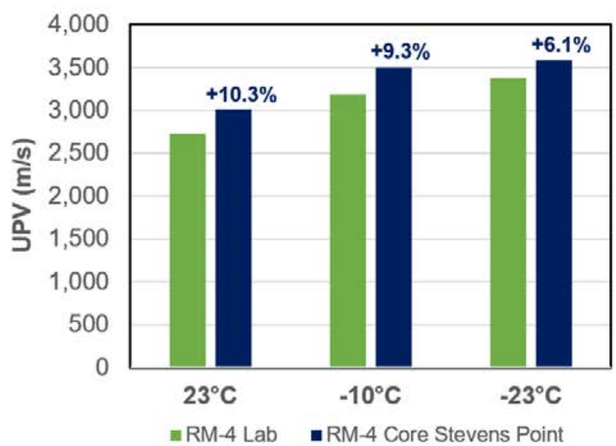


Figure 4-22. UPV comparisons—lab vs. core specimens.

The percentage values shown in the charts are the percentage change in the UPV values when compared to the laboratory specimens. While the UPV values through the RM-5 core specimens were higher than the UPV values measured through the laboratory specimens at 23 °C, an opposite trend was observed at the colder testing temperatures. For RM-4, the UPV values measured through the core specimens were slightly higher than the UPV values measured through the laboratory specimens. In general, significant differences were not observed in the UPV values between the laboratory and field specimens.

Summary

Key observations and findings from the laboratory testing are summarized below.

- **Bond Testing**. For the RM-2 specimens, the pull-off testing was performed successfully at testing temperatures of 23 °C and -10 °C and all of the failures noted were within the repair material. The glue failed to hold up at -23 °C for all but one test specimen. For the lone successful test at -23 °C, the failure plane was at the interface between the repair material and the substrate concrete. The pull-off bond testing effort was largely unsuccessful for the RM-4 and RM-5 specimens due to the following reasons:
  - Material melting during the course of specimen preparation.
  - Material being too soft to provide a suitable platform for the pull-off tester to apply a uniform tensile load.
  - Failure of the glue used to affix the steel disc to the test surface at cold temperatures.
- **Static Elastic Modulus**. The RM-2 material exhibits a linear stress-strain response for the most part. Under higher loading at 23 °C, a slight non-linear response was observed. The static elastic modulus test (ASTM C 469) is not a suitable test for flexible repair materials like RM-4 and RM-5 since they exhibit hysteresis effect.
- **Dynamic Modulus**. The modulus of RM-2 at lower temperatures (i.e., higher frequencies) is comparable to that of typical asphalt paving mixtures, while that of RM-5 and RM-4 is significantly lower. At lower temperatures, all the materials become relatively stiffer, however, the modulus values are still lower than conventional HMA. RM-5 and RM-4 appear to be very sensitive to changes in temperature and loading rate. The low modulus of these two materials indicates that they may be prone to rutting/permanent deformation if exposed to very high temperatures.
- **Ultrasonic Pulse Velocity**. The UPV values of the repair materials are a function of the testing temperature. The UPV values exhibit good correlations with the modulus values measured at different temperatures.

Dynamic modulus testing indicated that all non-cementitious materials become stiffer as the temperature decreases. Still, even at a temperature of -10°F (-23 °C), the materials do not become as stiff as conventional PCC. Modulus variations with temperature is not expected to adversely impact the bond between repair material and substrate concrete that is in sound condition. However, if all of the unsound concrete is not meticulously removed before the placement of the non-cementitious repair materials, bond failures are very likely to occur in the winter seasons as the pavement experiences freeze-thaw cycling. Hence, proper care must be exercised during the repair area preparation process when using non-cementitious repair materials.

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

Concluding Remarks

This project investigated the field performance of five commercially available, non-cementitious repair materials used in partial-depth repair applications for concrete pavements and bridge decks. Twenty-three different sites were visited as a part of the field investigations to document the condition of the selected repair materials and the surrounding concrete pavements in which they were installed. Limited laboratory testing was also conducted to characterize the bond and dimensional stability aspects of some of these materials at different testing temperatures. The significant findings from this study are summarized below.

- In Wisconsin, non-cementitious repair materials are typically used to repair heavily distressed areas in concrete pavements, particularly spalled joints, transverse cracks, and corner breaks. For a vast majority of the repairs performed, the repair boundaries are not demarcated using saw cuts. The unsound concrete (as determined through on-site sounding) is removed using jack hammers and the repair area is cleaned with compressed air before the material is placed.
- All the repair materials investigated were relatively flexible even at low temperatures (<20 °F). Coring of these materials was found to be more challenging since the binder in these materials melts under the heat generated in the coring process. The melted binder gums up the coring drill and as a result the whole operation required more water and time than a coring operation on typical asphalt or concrete materials. The “material melting” issue could be a potential concern when performing diamond grinding operations on pavements with these types of repairs where the melted binder can “gum-up” the diamond-head blades.
- The sudden failures of some RM-1 PDRs observed in 2016 and 2018 (where the entire patch popped out of the repair area) could potentially be attributed to the following factors:
    - **Failure to remove all the unsound concrete from the repair area**. In many cores extracted, a significant amount of substrate concrete material still appeared to be well-bonded to the bottom of the RM-1 cores. This may be an indication that the underlying deterioration or delamination in the existing concrete pavement was not adequately or completely removed prior to repair material placement.

      One of the core specimens extracted (see figure 3-7; Site #1, Core #2, Madison Beltline) exhibited multiple parallel horizontal cracks through the entire depth of the sample. When this specimen was conditioned at -23 °C for UPV testing, it completely disintegrated. Extremely cold temperatures during the winter seasons could potentially result in deterioration of the substrate concrete in areas where it is already in poor condition. This situation will surely compromise the bond between the repair material and the underlying concrete and can cause the material to pop out of the repair area.
    - **Continued deterioration of substrate concrete**. The substrate concrete has continued to deteriorate, likely due to the factors that caused the original failures which required repairs using PDRs. The potential failure mechanisms include: (a) freeze-thaw damage, (b) damage due to excessive use of deicing chemicals (physical and/or chemical), and (c) carbonation of the substrate concrete. All these issues are

likely to compromise the integrity of the quality of bond between the repair material and the substrate concrete. The deterioration process is likely to be accelerated by the increased saturation in the substrate concrete and the potential for excess moisture to be trapped between the impervious repair material and the underlying concrete pavement.

- **Inconsistency in the mixes produced on site**. For the RM-1 material, bulking stone is added on site and if the proper field control is not exercised, it is possible that inadequate amounts of binder may exist at the bonding interface, which could potentially result in a poor bond between the substrate concrete and the repair material.

The potential failure mechanisms noted above are merely speculative at this point and would require additional field and laboratory investigations for confirmation.

- As expected, dynamic modulus testing indicated that all non-cementitious materials become stiffer as the temperature decreases. Still, even at a temperature of -10 °F (-23 °C), the materials do not become as stiff as conventional PCC. Modulus variations with temperature is not expected to adversely impact the bond between repair material and substrate concrete that is in sound condition. However, if all of the unsound concrete is not meticulously removed before the placement of the non-cementitious repair materials, bond failures are very likely to occur in the winter seasons as the pavement experiences freeze-thaw cycling. The failures observed in the RM-1 material are most likely due to this situation. Hence, proper care must be exercised during the repair area preparation process. The existing concrete pavement must be thoroughly sounded and all the unsound material must be removed prior to the installation of the repair materials. The prepared repair area must be dry and free of any debris. The bonding agent (if any) recommended by the material manufacturer should be applied to bonding surfaces prior to the placement of the repair material. Further, if the concrete substrate continues to deteriorate in-service, no amount of preparation will ensure bonding of the repair to the substrate.
- The non-cementitious repair materials evaluated in this study (particularly the more flexible ones) generally do not crack. As observed with the RM-1 failures, if a bond loss with the substrate concrete occurs, these repair patches have the propensity to pop out in larger chunks, which can be present a significant safety issue to the traveling public. Minimizing the size of the repairs (particularly when the material is used to repair severely distressed areas), will help minimize the risk of catastrophic failures.

Recommendations for WisDOT Manuals

While many of the guidelines and specifications for PCC repair materials still apply to non-cementitious systems, some special considerations for non-cementitious materials are summarized below.

- **Removal of Unsound Concrete**. Good sounding practices should be followed, and greater care must be exercised to ensure that all the unsound concrete along the bonding surfaces of the concrete pavement is removed. Typical sounding approaches include: striking concrete surface with a steel rod or ball-peen hammer, or by dragging a chain along the pavement surface. A “dull response” indicates deteriorated concrete and a clear ring indicates sound concrete.

- **Coring**. Occasional coring is recommended to determine the extent of deterioration in the existing concrete pavement.
- **Surface Preparation.** The prepared surface must be clean and dry. The bonding agent (if any) specified by the material manufacturer must be applied to the surface prior to the placement of the repair material following the guidelines specified by the material manufacturer.
- **Mixing and Placement Temperatures**. Since the properties of non-cementitious materials are very sensitive to temperature changes, material manufacturer recommendations regarding mixing and placement temperatures must be strictly observed.
- **Surfacing**. The final layer of the placed repair material may be covered with a surfacing aggregate (per material manufacturer guidelines) to completely cover the repaired area.
- **Precautions for Hot-Applied Materials**: Many non-cementitious materials are hot-applied with mixing and placement temperatures in excess of 350 °F (177 °C), meaning that contact with skin will cause burns. In addition, some of these materials emit harmful fumes and over exposure to these fumes can potentially result in some health concerns for construction personnel and to the general public if exposed to the fumes. Consequently, proper precautions must be exercised to avoid contact with the material and inhalation of the fumes. Suggested precautionary measures include:
  - Protective clothing for construction personnel capable of withstanding high temperatures.
  - Use of appropriate tools designed to endure high temperatures.
  - Traffic management strategies that meet or exceed MURM-4D requirements.
  - Use of proper installation procedures that minimizes material wastage.
  - Proper clean-up and disposal of waste materials and spills from the job site.
- **Diamond Grinding**. When viscoelastic/flexible non-cementitious repair materials are used to repair areas in a concrete pavement that is scheduled to be diamond ground, the repairs must be performed at least 24 hours prior to the diamond grinding operation. The top layer of the repaired area that is expected to be diamond ground should be fortified with structural surfacing aggregated (as specified by the material manufacturer). The surfacing aggregate must be free of moisture.

  Key considerations for diamond grinding of a concrete pavement with large areas repaired with viscoelastic/flexible non-cementitious repair materials are summarized below.

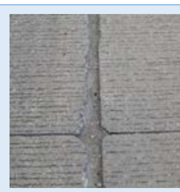
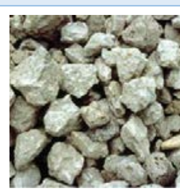
  - The loading and time of the grinding operations should be reduced to the extent possible. Heavy downward load applied by the grinding machine may remove too much material and this should be avoided. If the diamond-head blades sink too deep into the repair material, it will “gum-up” the blades and can potentially cause material to be sucked into the vacuum pumps. Proper care must be exercised to avoid this situation.
  - Grinding operations should be avoided when the ambient temperatures are high (temperatures when the material can become excessively soft).
  - The grinding head must be kept as cool as possible.

- The repair areas need to be relatively small, as large repairs with these materials will gum up the diamond bladed grinding heads.
- As an alternative, the installation of flexible repair materials could be performed after the grinding operation to avoid the issues altogether.

- **Evaluation of Existing Concrete**. For long-term repairs and if the repair material is expected to be used in large quantities, the existing concrete substrate should be evaluated to ensure that it is not susceptible to freeze-thaw damage. This will require conducting petrographic analysis in accordance with ASTM C856, evaluation of the entrained air-void system in accordance with ASTM C457, and possibly testing of extracted cores in accordance with ASTM C666. If freeze-thaw damage occurs in the substrate concrete after application of the repair material, debonding will occur.

A decision matrix for determining the appropriate treatment type and timing based on the observed joint condition is shown in table 5-1 (Weiss et al. 2016).

Table 5-1. Guidance on treatment selection based on joint condition (Weiss et al. 2016)

Condition	Illustration	Actions
Condition #1: Shadowing adjacent to joint		• Drain joint via subdrains • Remove backer rod • Minimize use of calcium chloride and magnesium chloride deicers
Condition #2: Spalling up to 1 inch from face of joint or no more than 2 inch total width		• Same as Actions Listed under #1 • Take cores to determine depth of deterioration • Temporarily filled spalled areas with sealant or asphalt patch material • Program PDR in two years
Condition #3: Spalling up to 2 inches form face of joint or no more than 4 inch total width		• Same as Actions Listed under #1 • Complete PDR • If cores show evidence of flaking, complete petrographic analysis to determine air voids and spacing; if poor air system, use full-depth repair • If pavement < 7 years old, use full-depth repair
Condition #4: Joint spalling > 4 inches form joint face		• Take cores to determine depth of deterioration • If depth < T/2, perform PDR • If deterioration > T/2 perform full-depth repair • If nearly every joint has severe spalling > 6 inches width, see options below.
Condition #5: Severe deterioration at every joint, no-mid panel deterioration, minimal vertical restrictions		• Mill each joint, remove loose material, and backfill with mortar • Construct unbonded concrete overlay
Condition #6: Severe joint deterioration at every joint, with vertical restrictions and/or where milling could be problematic		• Recycle pavement and construct new pavement

Additional guidance on partial-depth repairs of concrete pavements is available in the *Guide for Partial-Depth Repair of Concrete Pavements* (Frentress and Harrington 2012). Three general types of PDRs for cracks, joints, and spalls are defined in the guide (see figure 5-1).

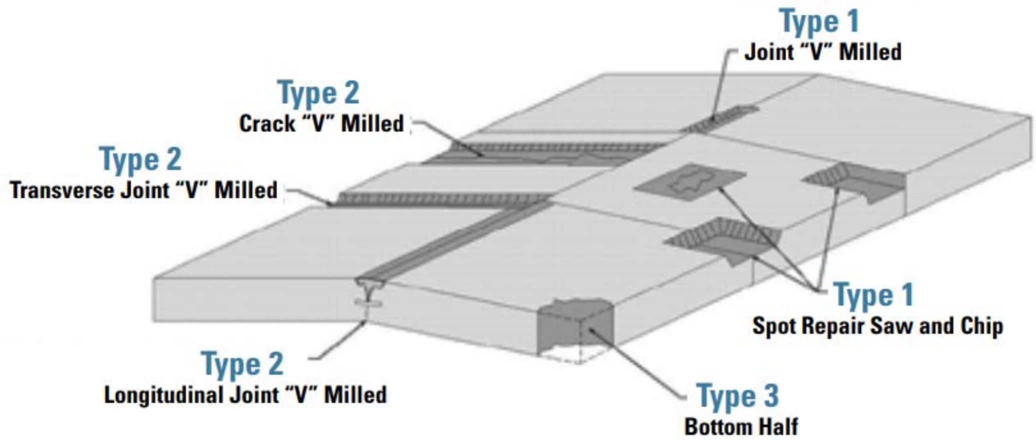


Figure 5-1. Types of PDRs (Fentress and Harrington 2012).

Non-cementitious repair materials can be used for Type 1 and Type 2 repairs and brief summaries on these repair types are provided below (Fentress and Harrington 2012):

- **Type 1: Spot Repairs of Cracks, Joints, and Spalls**.
  - Typically between 15 inches and 6 ft. long, and generally performed to address localized deterioration. Not recommended for long, continuous repairs.
  - Can be used to address joint spalling, mid-slab surface spalling or cracking, severe surface scaling, and joint reservoir issues.
  - Deteriorated concrete removed by either sawing around repair perimeter and removing unsound concrete with light jackhammers or small milling machines. The repair area should be slightly angled out (around 30 to 60 degrees) at edges to improve bond between repair material and substrate concrete. Typical repair details are shown in figure 5-2.
- **Type 2: Spot Repairs of Cracks, Joints, and Spalls**.
  - Repairs of extended length (> 6 ft.) and depths up to one-half of slab depth to address deteriorated longitudinal or transverse joint (Type 2A) or crack (Type 2B).
  - Compression relief constructed differently for Types 2A and 2B repairs. For Type 2A, the joint is re-established, typically through sawing. For Type 2B repairs, a preformed construction material is installed in the crack. Typical repair details are shown in figure 5-3.
  - When performing Type 2 repairs, sawing to re-establish the joint and the provision of compression relief is administered for the full thickness of the repair, plus an additional 0.25 to 1 inch. General construction procedures are the same as those for Type 1 repairs, with the exception of providing compression relief.
  - Some flexible non-cementitious repair materials may not require joint re-establishment, so follow material manufacturer guidelines as appropriate.

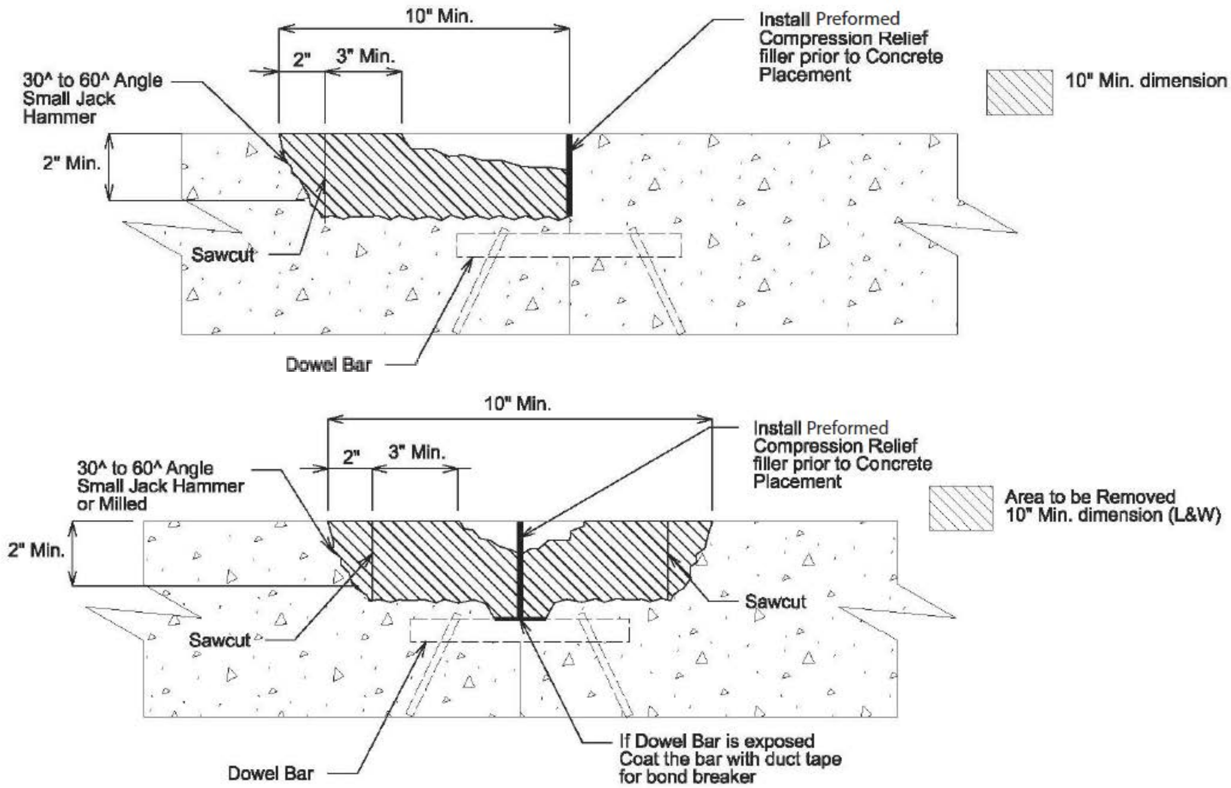


Figure 5-2. Typical details for Type 1 repairs: saw and chip (top) and milled (bottom) (Frentress and Harrington 2012).

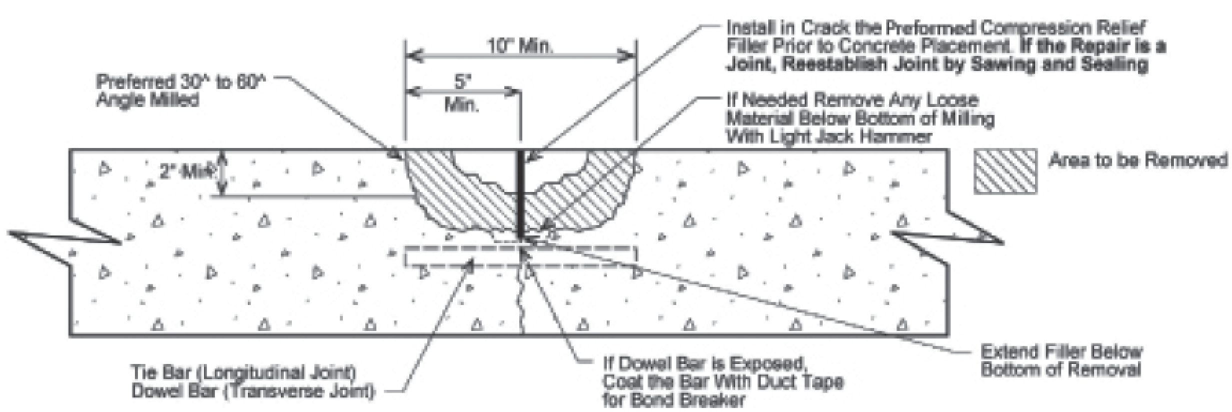


Figure 5-3. Typical details for Type 2 repairs (Frentress and Harrington 2012).

WisDOT maintains a *Standardized Special Provision* (STSP) for PDRs: *Concrete Pavement Partial Depth Repair—STSP 416-015*. This document covers joint repair, crack repairs, surface repair, and edge repair, and focuses exclusively on concrete materials. It is recommended that WisDOT add another section to this special provision to focus on non-cementitious repair systems. Table 5-2 provides a summary of guidance on the selection and use of non-cementitious repair materials for PDR applications that can be incorporated into WisDOT's STSP 416-015.

Table 5-2. Guidance on selection and use of non-cementitious materials for PDRs.

Factor	Categories	Guidance/Recommendation
General Considerations for PDRs using Non-Cementitious Materials	Evaluation of Existing Concrete Pavement	• If repair material is to be used in large quantities and for long-term repairs, conduct petrographic evaluation of existing concrete (in accordance with ASTM C856), evaluate entrained air-void system (in accordance with ASTM C457), and test freeze-thaw susceptibility of extracted cores (in accordance with ASTM C666).
	Repair Area Preparation	• Remove deteriorated concrete by either sawing around repair perimeter and removing all unsound concrete with light jackhammers (for spot repairs) or by milling the deteriorated area using small milling machines (for joint repairs). Repair area may be slightly angled out (~30-60 degrees) at edges to improve bond between repair material and substrate concrete. • Ensure that prepared area is clean and surface dry, particularly when using polyester polymer concretes (such as RM-2) or elastomeric concrete materials (such as RM-1 and RM-4). Asphalt-based materials (such as RM-3 and RM-5) are tolerant of moisture. • As specified by material manufacturer, apply bonding agent to surface prior to repair material placement.
	Material Mixing and Placement, and Curing	• Strictly adhere to mixing and placement temperatures specified by the material manufacturer. • For deeper repairs (> 2 inches), non-cementitious materials [particularly elastomeric concretes such as RM-1 and RM-4] should be placed in multiple lifts (~1 to 2 inches thick). • Non-cementitious materials are typically air-cured and require no special curing techniques. Follow material manufacturer guidance on curing.
	Mixing Equipment	• Some materials may require specialized equipment for material mixing and placement. Use equipment recommended by material manufacturer. • RM-1, RM-4, RM-3, and RM-5 require specialized high-temperature mixing kettle. • RM-2 can be mixed using drum or mortar mixers typically used for mixing conventional concrete materials.
	Joint Re-Establishment	• For relatively rigid non-cementitious materials (such as RM-2), re-establish joint by sawing and sealing. • Flexible materials (such as RM-1, RM-4, RM-3, and RM-5) typically do not require joint re-establishment; follow manufacturer guidelines as appropriate.
	Opening to Traffic	• Non-cementitious materials can typically be opened to traffic within 2 to 6 hours of placement, depending on placement and curing temperatures. • Follow material manufacturer guidance on opening to traffic.
	Safety Precautions for Hot Applied Materials	• Use protective clothing capable of withstanding high temperatures (>350 °F). • Use appropriate tools designed for high temperatures. • Some materials may emit strong fumes which can cause skin and/or eye irritation. Exercise proper precautions to avoid contact with material and inhalation of fumes.

Table 5-2. Guidance on selection and use of non-cementitious materials for PDRs (continued).

Factor	Categories	Guidance/Recommendation
Ambient Temperature During Installation	Low (< 32 °F)	• Follow material manufacturer recommendations.
	Moderate (32-80 °F)	• Follow material manufacturer recommendations.
	High (>80 °F)	• Follow material manufacturer recommendations.
Desired Service Life	Short-term (< 3 years)	• Use HMA-based patching material or cold patch material. Non-cementitious materials are **not recommended**.
	Medium to Long-term (≥ 3 years)	• Use commercially available non-cementitious materials only when existing pavement does not exhibit any sign of materials-related distress.
Existing Concrete Pavement Distress/Condition	Spalling up to 2 inches perpendicular from face of joint (longitudinal or transverse)	• Fill spalled areas with sealant, cold patch material, or HMA. • Extract cores to determine depth of deterioration. • Program PDR using non-cementitious material within 2 years. • Recommended PDR material from study: RM-3.
	Spalling between 2 to 4 inches perpendicular from face of joint (longitudinal or transverse)	• Extract cores to determine depth of deterioration. Conduct petrographic analysis to evaluate air-void system parameters and conduct other tests as appropriate to evaluate susceptibility to other materials-related distresses. o If air void system is deemed to be poor, then perform full-depth repairs (FDR) using conventional concrete. o If air void system is adequate, perform PDR using non-cementitious materials. Limit PDR width from joint face to a maximum of 6 inches. • Recommended PDR Materials from study: RM-4, RM-5
	Spalling > 4 inches perpendicular from joint face (longitudinal or transverse)	• Extract cores to determine depth of delamination. Conduct petrographic analysis to evaluate air-void system parameters and conduct other tests as appropriate to evaluate susceptibility to materials-related distresses. o If depth of deterioration ≤ T/2, perform PDR o If depth of deterioration > T/2 perform FDR • Limit PDR width from joint face to a maximum of 12 inches (along longitudinal joint) and 6 inches (along transverse joint). • If nearly every joint exhibits severe spalling (> 4 inches perpendicular from joint face), consider major rehabilitation activities such as unbonded concrete overlay or complete reconstruction. • Recommended PDR materials from study: RM-2, RM-4, RM-5
	Mid-slab surface spalling or cracking, severe surface scaling (typically 15 inches to 3 ft. long and deteriorated area ≤T/2)	• Extract cores to determine depth of delamination. Conduct petrographic analysis to evaluate air-void system parameters and conduct other tests as appropriate to evaluate susceptibility to materials-related distresses. o If depth of deterioration ≤ T/2, perform PDR o If depth of deterioration > T/2 perform FDR • Limit repair area to 4 sq. ft. • Recommended PDR materials from study: RM-2, RM-4, RM-5

Table 5-2. Guidance on selection and use of non-cementitious materials for PDRs (continued).

Factor	Categories	Guidance/Recommendation
Existing Concrete Pavement Distress/Condition	Severe D-Cracking, ASR or other materials-related distresses that has caused significant amounts of cracking or spalling in existing pavement	• **<u>Do not use</u>** commercially available non-cementitious repair materials. • Use HMA or cold patch material as a stop-gap solution to address safety concerns. • If distress is localized to a few slabs, replace distressed slabs. • If distress is widespread over the entire length of the pavement segment, consider major rehabilitation activities such as unbonded concrete overlay or complete reconstruction.
Patch Size	Repairs along longitudinal joints	• Limit PDR width (perpendicular from longitudinal joint face) to a maximum of 12 inches.
	Repairs along transverse joints	• Limit PDR width (perpendicular from transverse joint face) to a maximum of 6 inches.
	Mid-slab repairs or repairs at slab corners	• Limit patch area to 4 sq. ft.
Considerations for Other Treatments Associated with PDRs using Non-Cementitious materials	Diamond Grinding	• When elastomeric concretes (such as RM-1 and RM-4) are used to repair areas scheduled to be diamond ground, repairs must be performed at least 24 hours prior to diamond grinding operation. • Alternatively, the elastomeric PDR can be performed after diamond grinding. This may be the preferred option for larger sized repairs. • Reduce time and loading of grinding operation to the extent possible. • Avoid grinding at high ambient temperatures (>90 °F). • Keep grinding head as cool as possible.
	Overlays (asphalt or concrete)	• Placement of thin overlays over areas repaired using non-cementitious materials may cause delamination issues. • For areas that are likely to be overlaid in the near future, **<u>do not use</u>** non-cementitious materials for PDRs. • Existing non-cementitious repairs need to be removed and replaced with conventional concrete or HMA-based repair materials prior to overlay placement.
Note: Much of the guidance/recommendations provided in this table are based on anecdotal evidence. Additional research as recommended in the last section of this chapter should be considered to refine these recommendations.		

Suggestions for Future Research

Based on the results of this project, recommendations for future work activities are presented below.

- **Controlled Field Study**. Field investigations were performed on repairs that were: (a) of different ages, (b) installed by different contractors, (c) installed using varying surface preparation techniques, (d) used to repair different types of distresses, (e) used on concrete pavements that varied in condition, and (f) placed on an existing pavement whose condition was not well documented at the time repairs were performed. In order to truly characterize the field performance repair materials that WisDOT is interested in pursuing further, a controlled field experiment is recommended in which the prevailing conditions during the installation of the repairs are well documented. Furthermore, the repairs should be periodically inspected (approximately once every 6 months) and the conditions should be documented over at least a 5-year period (or until the failure/replacement of the material).

- **UPV and Dynamic Modulus Testing**. UPV values exhibit very good correlations with the testing temperatures and dynamic modulus values. Once a material is approved for use by WisDOT, it is recommended that the relationship between UPV, testing temperature, and dynamic modulus be established for repair materials that exhibit a viscoelastic response. Once the correlation is established, UPV testing can be used as a quick way of estimating the dynamic modulus at various temperatures. This may lead to the development of an assessment protocol to judge the quality of the concrete to be repaired.
- **Bond Durability Study**. The pull-off test method was noted to be largely unsuccessful in this study due to a number of factors. Since cold temperature bond-performance remains a concern with these materials, it is proposed that a direct shear bond test (similar to the Iowa shear test [Iowa DOT 2000]) be used to evaluate the bond durability of these materials when subjected to freeze-thaw cycling (25, 50, 100, 150, 200, 250, and 300 cycles) with and without use of deicing chemicals in a saturated environment where temperature cycles can be customized to the harshest free-thaw cycles expected in Wisconsin. Additional test parameters could include:
  - Substrate concrete mix design (particularly, w/cm, paste content, air void system).
  - Substrate concrete surface carbonation.
  - Surface preparation techniques.

  This test can potentially uncover the failure mechanisms related to bond between substrate concrete and repair material.
- **Benefit-Cost Analysis**. In order to quantify the effectiveness of non-cementitious repair materials, a benefit-cost analysis should to be conducted. Costs can be quantified in terms of the material cost per unit volume and the service life of the repair can be approximated as the benefit obtained. The benefit-cost ratios of non-cementitious repair materials should be compared to asphalt, cold patch, conventional concrete, and other commercially available rapid-setting repair materials. Other important factors to consider would be production rate and construction downtime, material availability, ease of material mixing and placement, availability of qualified contractors for material placement, ease of removal and disposal, and environmental and social impacts (for example, some materials may contain chemicals that are harmful to human health and ecosystems).

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APPENDIX A. CORE PHOTOGRAPHS

Figure A-1. Site #1, RM-1 Core #1
(US 12/18, Madison Beltline).

Figure A-2. Site #1, RM-1 Core #1 side view
(US 12/18, Madison Beltline).

Figure A-3. Site #1, RM-1 Core #1 top view
(US 12/18, Madison Beltline).

Figure A-4. Site #1, RM-1 Core #1—Bottom view of RM-1 portion
(US 12/18, Madison Beltline).

Figure A-5. Site #1, RM-1 Core #2
(US 12/18, Madison Beltline).

Figure A-6. Site #1, RM-1 Core #2—Closeup of PCC core
(US 12/18, Madison Beltline).

Figure A-7. Site #1, RM-1 Core #2—sideview of RM-1 portion
(US 12/18, Madison Beltline).

Figure A-8. Site #1, RM-1 Core #2—Bottom view of RM-1 portion
(US 12/18, Madison Beltline).

Figure A-9. Site #1, RM-1 Core #3—sideview of RM-1 portion (US 12/18, Madison Beltline).

Figure A-10. Site #1, RM-1 Core #3—Closeup of melted material (US 12/18, Madison Beltline).

Figure A-11. Site #5, RM-3 Core #1
(W. Wisconsin Ave, Oconomowoc).

Figure A-12. Site #5, RM-3 Core #1 close-up
(W. Wisconsin Ave, Oconomowoc).

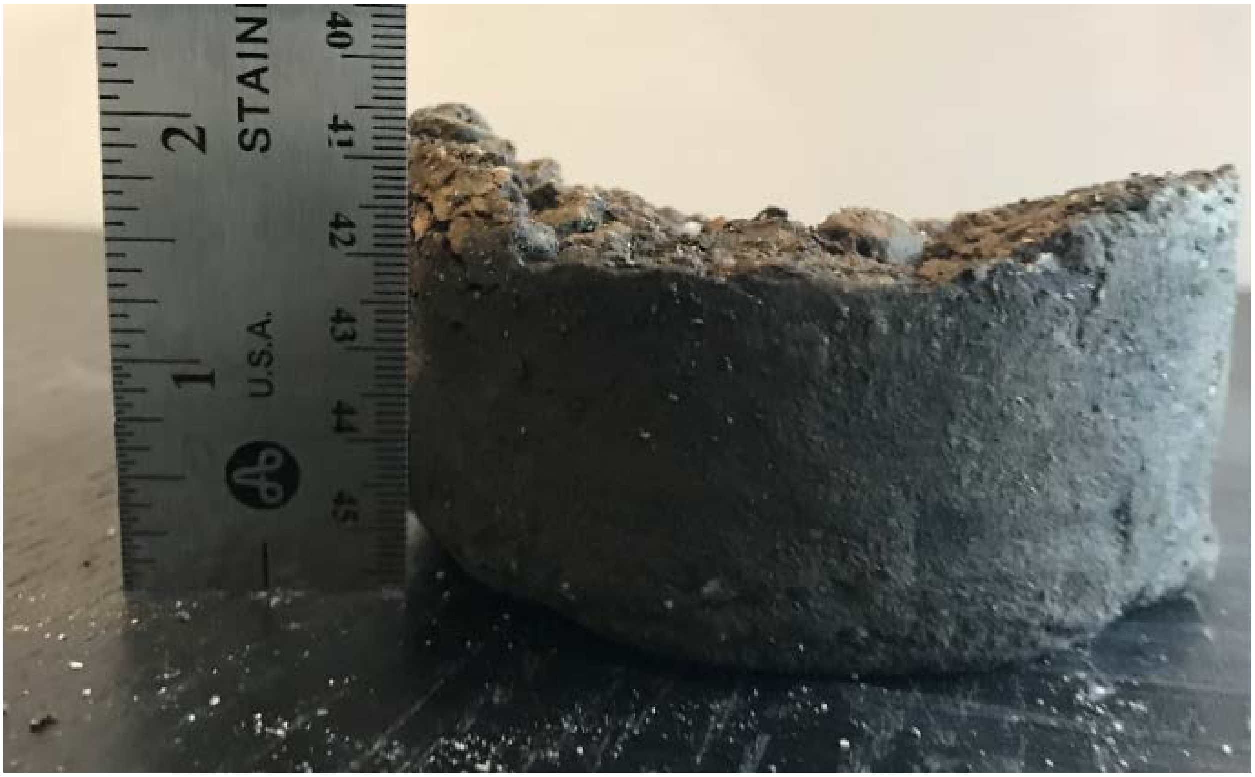

Figure A-13. Site #8, RM-3 Core #1
(State Route 66 SB, Stevens Point).

Figure A-14. Site #8, RM-3 Core #1—concrete pavement core pieces
(State Route 66 SB, Stevens Point).

Figure A-15. Site #9, RM-4 Core #1
(Washington St., Grafton).

Figure A-16. Site #9, RM-4 Core #1—concrete portion
(Washington St., Grafton).

Figure A-17. Site #11, RM-4 Core #1
(US 10 EB, Stevens Point).

Figure A-18. Site #20, RM-5 Core #1
(N. Stoughton Rd., Madison).

Figure A-19. Site #21, RM-5 Core #1
(State Rd. 59, Waukesha).

APPENDIX B. RM-1 SITE VISIT PHOTOGRAPHS

Figure B-1. RM-1 PDR (1)
(Site #1, US 12/18, Madison Beltline).

Figure B-2. RM-1 PDR (2)
(Site #1, US 12/18, Madison Beltline).

Figure B-3. RM-1 PDR (3)
(Site #1, US 12/18, Madison Beltline).

Figure B-4. RM-1 PDR (4)
(Site #1, US 12/18, Madison Beltline).

Figure B-5. RM-1 PDR (5)
(Site #1, US 12/18, Madison Beltline).

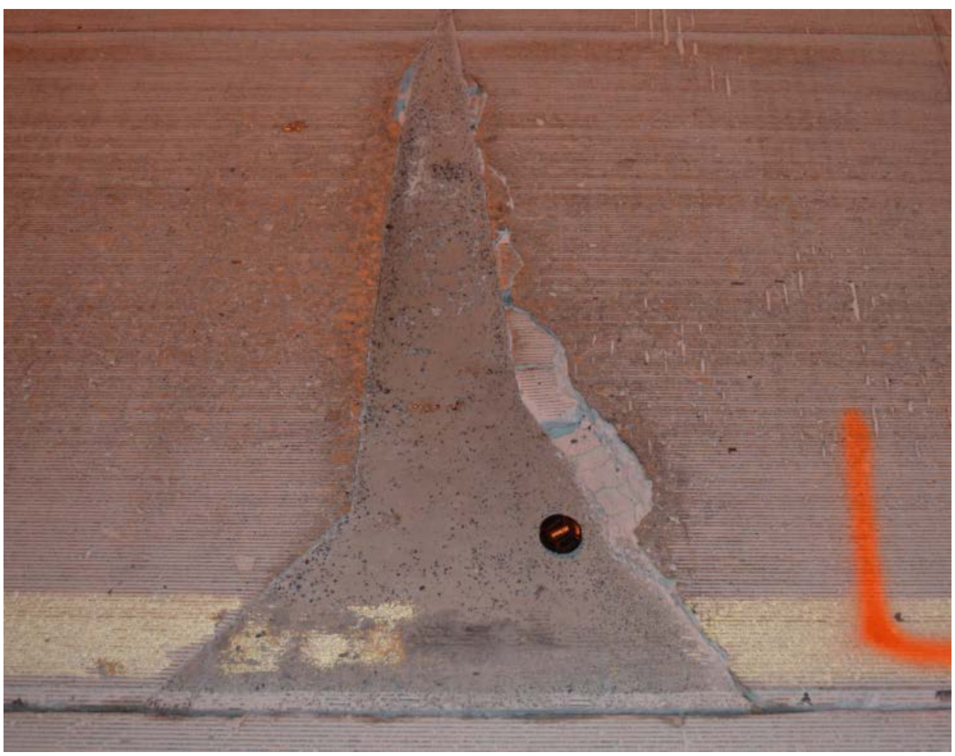

Figure B-6. RM-1 PDR (6)—deterioration along patch edges
(Site #1, US 12/18, Madison Beltline).

Figure B-7. RM-1 PDR (6)—close-up of deterioration along patch edge (Site #1, US 12/18, Madison Beltline).

Figure B-8. RM-1 PDR (7)—close-up of repair surface (Site #1, US 12/18, Madison Beltline).

Figure B-9. RM-1 PDR (8)—large, full lane-width patch
(Site #1, US 12/18, Madison Beltline).

Figure B-10. RM-1 PDR (9)—small patch
(Site #1, US 12/18, Madison Beltline).

Figure B-11. Workzone—areas being marked for full-depth repairs
(Site #1, US 12/18, Madison Beltline).

APPENDIX C. RM-2 SITE VISIT PHOTOGRAPHS

Figure C-1. Marquette Interchange bridge deck overview (Site #2, I-43, Milwaukee).

Figure C-2. Marquette Interchange bridge deck overview (2) (Site #2, I-43, Milwaukee).

Figure C-3. I-94 bridge deck over Menomonee River
(Site #3, I-94, Milwaukee).

Figure C-4. I-43 bridge deck over Beloit Rd.
(Site #4, I-43, New Berlin).

APPENDIX D. RM-3 SITE VISIT PHOTOGRAPHS

Figure D-1. RM-3 used to repair spalls, deteriorated joints and traffic loops (Site #5, W. Wisconsin Ave, Oconomowoc).

Figure D-2. PDRs to address spalling and deteriorated joints
(P Site #6, lover Rd., Plover).

Figure D-3. PDRs to address spalling along lane-shoulder joint
(Site #6, Plover Rd., Plover).

Figure D-4. Overview of existing concrete pavement
(Site #6, Plover Rd., Plover).

Figure D-5. Overview of RM-3 repairs
(Site #7, S. Taylor St., Green Bay).

Figure D-6. PDRs along transverse joint
(Site #7 S. Taylor St., Green Bay).

Figure D-7. Overview of existing concrete pavement
(Site #8, State Road 66, Stevens Point).

Figure D-8. D Cracking and full-depth repairs in existing concrete pavement (Site #8, State Road 66, Stevens Point).

Figure D-9. Small spot repairs in slab interior
(Site #8, State Road 66, Stevens Point).

Figure D-10. PSPA testing in progress on PDR in slab interior
(Site #8, State Road 66, Stevens Point).

APPENDIX E. RM-4 SITE VISIT PHOTOGRAPHS

Figure E-1. PDRs along transverse joint
(Site #9, Washington St., Grafton).

Figure E-2. PDRs to address deteriorated areas around utility hole
(Site #9, State Road 66, Stevens Point).

Figure E-3. PDR over a transverse crack
(Site #9, State Road 66, Stevens Point).

Figure E-4. Material loss along edges of a small PDR
(Site #9, State Road 66, Stevens Point).

Figure E-5. Concrete pavement deterioration adjacent to PDR
(Site #10, State Road 145, Menomonee Falls).

Figure E-6. Concrete pavement deterioration around PDR
(Site #10, State Road 145, Menomonee Falls).

Figure E-7. Typical rush hour traffic
(Site #11, US 10, Stevens Point).

Figure E-8. Concrete pavement deterioration around PDR
(Site #11, US 10, Stevens Point).

Figure E-9. PDR used to address transverse cracking
(Site #11, US 10, Stevens Point).

Figure E-10. Severely deteriorated areas in existing concrete pavement (Site #11, US 10, Stevens Point).

Figure E-11. Full depth repairs in existing concrete pavement (Site #11, US 10, Stevens Point).

Figure E-12. Corner break in existing concrete pavement
(Site #11, US 10, Stevens Point).

Figure E-13. PDR to address corner break
(Site #11, US 10, Stevens Point).

Figure E-14. Overview of existing concrete pavement
(Site #12, US 41, Marinette).

Figure E-15. PDR to address spalling at a transverse joint
(Site #12, US 41, Marinette).

Figure E-16. Concrete pavement deteriorating around the PDR (Site #12, US 41, Marinette).

Figure E-17. PDR used to address deteriorated longitudinal joint (Site #12, US 41, Marinette).

Figure E-18. PDRs on bridge deck (1)
(Site #13, State Road 32, Mountain).

Figure E-19. Map cracking pattern on PDR surface
(Site #13, State Road 32, Mountain).

Figure E-20. PDRs on bridge deck (2)
(Site #13, State Road 32, Mountain).

Figure E-21. PDR on bridge deck approach slabs, surface map cracking in shoulder area (Site #14, US 141, Pound).

Figure E-22. PDR to address joint spalling
(Site #15, Calumet St., Appleton).

Figure E-23. Existing concrete pavement deteriorating around PDRs
(Site #15, Calumet St., Appleton).

Figure E-24. PDRs on bridge deck
(Site #16, W. Main Ave., Ashwaubenon).

Figure E-25. RM-4 PDRs on bridge deck with starkly different colors
(Site #16, W. Main Ave., Ashwaubenon).

Figure E-26. Overview of existing concrete pavement in very good condition (Site #17, W. Johnson St., Fond du Lac).

Figure E-27. PDR to address an isolated area of distress in an otherwise distress free pavement (Site #17, W. Johnson St., Fond du Lac).

Figure E-28. PDR to address deterioration along transverse joints
(Site #18, I 41, Oshkosh).

APPENDIX F. RM-5 SITE VISIT PHOTOGRAPHS

Figure F-1. PDR to address deterioration along transverse joint (Site #19, State Road 93, Eau Claire).

Figure F-2. Overview of generally distress-free surrounding concrete pavement (Site #19, State Road 93, Eau Claire).

Figure F-3. Overview of existing concrete pavement (Site #20, N. Stoughton Rd., Madison).

Figure F-4. PDR used to address a severely distressed area
(Site #20, N. Stoughton Rd., Madison).

Figure F-5. Overview of existing concrete pavement
(Site #21, State Road 59, Waukesha).

Figure F-6. PDR to address corner spalling
(Site #21, State Road 59, Waukesha).

Figure F-7. Existing pavement deteriorating around PDR
(Site #21, State Road 59, Waukesha).

Figure F-8. Overview of existing concrete pavement
(Site #22, N. Ballard Rd., Appleton).

Figure F-9. PDR to address deteriorated joint (left) and
existing pavement deteriorating around PDR (right) (Site #22, N. Ballard Rd., Appleton).

Figure F-10. PDR to address longitudinal joint deterioration
(Site #22, N. Ballard Rd., Appleton).

Figure F-11. Overview of existing concrete pavement
(Site #23, W. Kemp St., Rhinelander).

Figure F-12. PDR to address transverse joint deterioration (left) and surrounding concrete pavement deteriorating around PDR (Site #23, W. Kemp St., Rhinelander).

Figure F-13. PDR to address joint deterioration (Site #23, W. Kemp St., Rhinelander).