Better Concrete Mixes for Rapid Repair in Wisconsin

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16. Abstract

With the increasing demands on the highway system and increasing costs of user delay, the use and development of rapid-repair techniques are expected to grow rapidly. High early strength (HES) portland cement concrete can help reduce the duration of traffic closures while being cost-competitive with other solutions such as concrete using proprietary cements and precast concrete. This research investigated the performance of HES portland cement concrete used for pavement repairs through multiple approaches. Thirteen mixtures were made and tested in the laboratory for strength, drying shrinkage, and scaling resistance. A field review of 12 recent rapid-repair pavement projects in Wisconsin was conducted to evaluate field performance of the pavements. An informal survey of Wisconsin concrete suppliers was conducted to obtain information on mix design, challenges, and approaches to rapid repair of pavement. Life-cycle cost analysis was conducted for pavement repairs using cost data assumed to be representative of the lab tested mixtures and precast concrete. The field review showed no significant durability issues except for one project where severe scaling occurred. Concrete using portland cement with calcium chloride accelerator can surpass WisDOT compressive strength requirements of 3000 psi in 8 hours and have satisfactory scaling resistance. Concrete using a non-chloride accelerator had excellent scaling resistance, slightly higher shrinkage than non-accelerator concrete, and may be a good alternative to calcium chloride when the strength requirement of 3000 psi can be extended to within 10 hours. Overall it appears that durability issues that occurred in rapid-repair pavements are more likely due to difficulties associated with construction or mix procedures than the WisDOT specifications. To ease these difficulties, the use of dry calcium chloride may be considered if it can be mixed uniformly with the concrete. An upper slump limit specifically for rapid-repair concrete would provide flexibility and may be higher than that for conventional concrete.

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EXECUTIVE SUMMARY

Full-depth rapid repair of highway sections increasingly is adopted as a repair strategy to maximize service function of the highway while minimizing the disruption to the transportation system. With the increasing demands on the highway system and increasing costs of user delay, we can expect the use and development of rapid-repair techniques to grow rapidly. Blended, special high-early-strength, and portland cements can be used for repair of concrete pavement. This study focused on the use of portland cement for full-depth rapid repair. Rapid-repair concrete may be more vulnerable to durability issues due to its unique features such as high cement content, unknown interactive effects of accelerators, superplasticizers, air entraining admixtures and other mixture constituents, and limited construction time. These features tend to lead to increased amounts and rates of shrinkage and may result in undesirable air void systems.

The overall objectives of this study were to evaluate the performance of current rapid-repair mixtures in Wisconsin and in other states, and to provide recommendations on better mixtures and construction practices for rapid repair of concrete pavement in Wisconsin. To achieve these objectives, the following five tasks were performed:

i. Literature review: A literature review of current concrete mixtures and procedures for pavement rapid repair in Wisconsin and other highway agencies was conducted.

ii. Field reviews: Twelve recent pavement rehabilitation projects using rapid-repair concrete mixtures in Wisconsin were reviewed. At each project site, a visual assessment of the pavement conditions was conducted following recommendations of FHWA-RD-03-031 "*Distress Identification Manual for the Long-Term Pavement Performance Project*"[1].

iii. Informal survey: In an effort to obtain additional information of current practices in Wisconsin, the research team conducted brief phone communications with rapid-repair concrete suppliers to garner information on their mix designs, challenges and general approach to full-depth rapid-repair projects.

iv. Lab testing: Based on information from the literature and field reviews and informal survey, candidate rapid-repair mixtures were proposed, made, and tested in the lab to evaluate their strength development, drying shrinkage, and scaling resistance. For each candidate or base mix, several deviant mixtures were made to investigate factors that may alter its performance. A total of 13 mixtures were made and tested.

v. Cost/benefit analysis: An economic analysis was performed (in general compliance with current WisDOT FD 14-15 policy) using representative cost data and assumed performance life values for

rapid repairs constructed using the mixtures that were tested in the laboratory. These life-cycle costs were compared with those similarly developed for precast concrete pavement repairs.

A summary of key findings is as follows:

- Overall no significant durability issues were found, with the exception of one rapid-repair project surveyed in the field review. Lack of field design and construction information prevented a correlation between observed pavement performance, and design and construction factors.
- Specimens from the lab mixtures all exhibited generally acceptable levels of scaling resistance, with average scaling mass loss after 60 freeze-thaw cycles less than or equal to 500 g/m^2, except for those from one mixture that used PAMS curing compound.
- Specimens from mixtures using type I portland cement with calcium chloride accelerator achieved compressive strength of 3000 psi within 6 hours after mixing and performed well in the scaling test. These mixtures tended to exhibit higher shrinkage than those without accelerator and with non-chloride accelerator. Calcium chloride in the form of dry flakes, if properly added, can result in concrete having strength, drying shrinkage, and scaling resistance similar to mixtures using calcium chloride solution.
- The calcium-nitrate based, non-chloride accelerator used in this study was a viable alternative to calcium chloride when the compressive strength requirement of 3000 psi can be extended to within 10 hours rather than 8 hours. Mixtures using the non-chloride accelerator exhibited excellent scaling resistance and slightly higher shrinkage compared to those without accelerator.
- In the laboratory, both calcium chloride and non-chloride accelerators appeared to improve scaling resistance compared to the control.
- Based on information from the literature and field reviews, an informal survey with concrete suppliers and results of the laboratory tested mixtures, it appears that durability issues, such as the premature severe scaling occurring at one project mentioned above, more likely stem from challenges associated with the construction of rapid pavement repairs. Regarding construction procedures, there has been a desire by rapid-repair concrete contractors to use dry calcium chloride to alleviate difficulties in controlling slump when the accelerator must be added at the job site. In this case, using calcium chloride in dry forms presented no difficulties when it was mixed uniformly with the concrete. Allowing a higher upper limit of slump can be another way to mitigate problems with

slump control. For a mixture similar to those used in this research, an upper slump limit of 6 inches may help to ensure constructability while providing satisfactory performance.

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CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

Full-depth rapid repair of highway sections increasingly is being adopted as a repair strategy to maximize service function of the highway while minimizing the disruption to the transportation system. With the increasing demands on the highway system and increasing costs of user delay, we can expect the use and development of rapid-repair techniques to continue growing rapidly. Repairs can take the form of partial-depth and full-depth repairs with a variety of proprietary products available for partial-depth repair. This study focusses on full-depth repairs of portland cement concrete pavements using portland cement concrete for the repair. Because of the larger volumes of full-depth repairs, more conventional concrete materials are commonly used with a mix design that is intended to achieve the required strength gain in a relatively short time.

In Wisconsin, rapid-repairs of concrete pavements have often been reported to have unsatisfactory durability and performance, resulting in reduced service life. It has not been clear whether the problem is caused by the materials or the construction techniques. The overall objectives of this study were to evaluate the performance of current rapid-repair mixtures in Wisconsin and other states, and to provide recommendations on better mixtures and construction practices for rapid repair of concrete pavement in Wisconsin. The life cycle costs associated with cast-in-place (CIP) rapid repairs were compared with those of repairs made using precast concrete pavement.

1.2 RESEARCH OBJECTIVES

The research objectives were to evaluate current practice in rapid repair of concrete pavements in Wisconsin and to recommend changes where appropriate. Four major deliverables were targeted:

- A basic inventory of current materials, concrete mixtures and procedures for rapid repair being used by WisDOT and other highway agencies;
- Identification of high-quality concrete rapid-repair mixtures that are capable of providing long life and good performance in the wet freeze-thaw climate that is typical for Wisconsin;
- A field review and performance analysis on recent Wisconsin rapid-repair projects to evaluate how repairs are performing;

- A categorization of technologies and methods available for installation, guidance for mix design and placement, and cost estimates for the concrete mixtures being evaluated for rapid repair.

1.3 RESEARCH APPROACH

The research objectives were accomplished through completion of the following tasks:

i. Literature survey;

ii. Field review;

iii. Survey with concrete suppliers;

iv. Laboratory testing;

v. Economic analysis.

CHAPTER 2. LITERATURE REVIEW

2.1 SUMMARY

A review of the basic inventory of current concrete materials and procedures being used for rapid repair by WisDOT and other highway agencies was conducted. Several key studies on the performance of rapid-repair concrete were also reviewed. The main findings are as follows:

- Portland cement concrete (PCC) has been successfully used to provide rapid pavement repairs with good quality and long-term performance. Special high-early-strength cements are recommended when opening to traffic is required in less than 4 hours.
- When portland cement is used, the cement content typically ranges from 650 to 900 lb/yd^3, with more cement added for earlier opening times. Supplemental cementitious materials (SCMs) are generally not used in rapid-repair concrete mixtures because of their tendency to slow strength development. Water-to-cement ratios (w/c) vary among different states, typically from 0.32 to 0.45. High-range water-reducers (HRWR) are frequently used to reduce w/c ratio while maintaining adequate workability. Calcium chloride is the most common accelerator for jointed plain concrete pavement; normal dosages are 1-2% by weight of cement.
- Problems with air void systems, such as poorly formed air voids and insufficient air contents, appeared to occur more often with the use of type III portland cement, HRWR, and calcium chloride accelerator.
- The performance of rapid-repair concrete depends on mixture components as well as on construction processes and conditions. Due to its impact on rapid hydration, calcium chloride is often added at the job sites, making it challenging to achieve the desired slump, to get the concrete out of the truck, and to obtain proper finishing. The difference in temperature between rapid-repair concrete (typically higher mixture temperatures) and low night-time ambient temperatures has been suggested as a cause for premature longitudinal cracking of rapid-repair concrete.

- Criteria for opening to traffic vary among states. Typical strength thresholds are compressive strength of 2000 to 3000 psi and/or flexural strength of 400 to 600 psi. Threshold selection impacts the choice of rapid-repair strategy.

2.2 CURRENT PRACTICES

2.2.1 Practices in Wisconsin

WisDOT standard specifications[2] (Section 416.2.5) for special high early strength concrete (SHES) pavement require the use of at least 846 lbs of cementitious materials per cubic yard of concrete. A minimum concrete compressive strength of 3000 psi is required within 8 hours of placement.

Calcium chloride, if used, is required to be added to the mix in solution. All the concrete must be discharged within 45 minutes after adding mixing water to the cement, or the cement to the aggregates, or within 30 minutes after adding an accelerating admixture, whichever comes sooner.

A review of the database of WisDOT pavement rehabilitation projects shows that most rapid-repair jobs used a cement content of 840-900 lb/yd^3. The total weight of aggregate ranged from about 2500 to 2900 lb. Fine aggregate proportion was between 35% and 50%. Water-to-cementitous materials ratio (w/cm) was between 0.29 and 0.42. Both calcium chloride and non-chloride accelerators, normal range, mid-range and high-range water reducers have been used. Set retarders have been used in some cases.

2.2.2 Practices being used by other highway agencies

The American Concrete Pavement Association's (ACPA) "Guidelines for Full Depth Repair"[3] recommends the use of Type I Portland cement with accelerator for pavement that requires traffic openings after 6-8 hours or longer. For shorter opening-to-traffic requirements, Type III Portland cement or special cements are suggested. These guidelines recommended strength for opening to traffic that ranges from 2000 to 3000 psi (compressive) and 250 to 490 psi (flexural) depending on repair length and slab thickness. A slump of 2 to 4 in. is recommended for finishing purposes. Air content of 4.5 to 7.7% is recommended for freeze-thaw durability.

The Federal Highway Administration (FHWA) has the following guidelines for full-depth repair[4]:

- Type I or Type III Portland cement content: 658-846 lb/yd^3.
- A set accelerator is typically used to facilitate opening in 4-to-6 hours.

- The use of proprietary concrete mixtures is generally necessary to achieve sufficient strength for opening in as little as 2 hours.
- Using insulating blankets (or boards) during the first few hours after placement also can improve the strength development of any mix.
- Calcium chloride results in fast setting; thus only 1% of calcium chloride by weight of cement is recommended when air temperature exceeds 27°C (80°F). Up to 2% is acceptable in lower temperatures. For on-site mixing, calcium chloride is added in liquid form to the mixer before other admixtures are added (except the air-entraining admixture).

Many states have provisions for concrete pavement rapid repair. Several examples are provided below.

Illinois DOT Specifications[5] (Section 1020) identifies five concrete mixtures (PP-1 to PP-5) for pavement patching with varying strength requirements. For situations requiring a compressive strength of 3200 psi or a flexural strength of 600 psi at 4 hours, 675 lb calcium aluminate cement is used per cubic yard of concrete. For situations requiring these strengths at 8 hours, 600-625 lb rapid hardening cement is used. Type I or type III portland cement may be used where the above strengths are required at 16 hours or longer. W/c ratio is typically 0.32-0.50. Slump is allowed to range from 2 to 8 in. The air content requirement for a typical patching mix is 4.0 to 6.0%.

CALTrans[6] provides guidelines for rapid full-depth repair in Chapter 8 of their MTAG-Volume II-Rigid Pavement Preservation and highlights the use of portland cement concrete mixtures for rapid full-depth repair based on cost and performance. Type III portland cement with accelerator is recommended to meet opening strength requirements within 4 to 6 hours.

Ohio DOT Specifications (Item 255.02.A) for full-depth repair concrete require a flexural strength of 400 psi in not less than 4 hours and not more than 6 hours. No detail of the mix materials is provided.

Tennessee DOT[7] has special provisions for rapid repair (SP502c) that require a compressive strength of 2500 psi in 6 hours and a maximum w/c ratio of 0.4, but the provisions are brief and do not contain detail.

NCHRP Report 540[8] *"Guidelines for Early-Opening-to-Traffic Portland Cement Concrete for Pavement Rehabilitation"* summarized specifications for early opening-to-traffic (EOT) repair materials of 16 state highway agencies including Wisconsin. Type I and Type III portland cements are used in many of these states with cement contents ranging from 658 to 900 lb/yd^3. W/c ratio typically ranges from 0.33 to less than 0.45. Calcium chloride is the most common accelerator with dosage of 1 to 2% by weight of cement. Opening-to-traffic criteria vary among the states. A compressive strength of 2000-3000 psi and/or a flexural strength of 300-400 psi are typically specified.

2.3 PERFORMANCE OF RAPID-REPAIR CONCRETE PAVEMENTS

Whiting and Nagi[9] conducted research on rapid repair concrete samples made in the field. Concrete mix designs were prepared and evaluated at two different field project sites – one in Georgia and one in Ohio. The mix designs included the use of Type I and Type III portland cements, Pyrament blended cements, regulated and rapid-set portland cements, high- and low-range water reducers, air-entraining agents, calcium chloride and non-chloride accelerators. Cement contents ranged from 650 lbs/cu yd. to 915 lbs/cu yd. with water-cement ratios ranging from 0.27 to 0.50. Evaluation tests included compressive strength, splitting tensile strength, freeze/thaw (F/T) testing (AASHTO T 161) and hardened concrete air void measurements (ASTM C457). Compressive strengths of 4,000 psi were achieved within 24 hours and F/T durability was generally good but varied somewhat by site location. Instances of poor F/T durability were attributed to the presence of calcium chloride as an accelerator, use of added water to achieve desired workability, and microcracking.

A study following on the work of Whiting and Nagi was conducted over 5 years (1994-1998) with the objectives to monitor and evaluate the performance of experimental full-depth repairs made with high-early-strength concretes[10]. The monitoring program consisted of annual visual distress surveys to monitor the development of cracking, faulting, and spalling. This study concluded that high early strength PCC can provide reasonable long-term performance to pavement full-depth repairs; however, adverse temperature conditions during construction can cause premature cracking. The risk of longitudinal cracking is high when the difference between concrete and ambient temperatures exceeds about 10°C (18°F). Extremely high concrete temperature during hydration, above 70°C (158°F), can also cause delayed ettringite formation.

Van Dam et al.[8] examined high early strength PCC pavement with funding from NCHRP. Their focus was on concrete mixtures suited for opening to traffic within 6 to 8 hours and 20 to 24 hours. In their field survey spanning the states of Ohio, Georgia, Texas and New York, no single cause of distress was reported. In general, the concrete was of good quality, and it was difficult to find “distressed” repairs for use in this study. The laboratory section of this research examined 14 mixtures for opening to traffic within 6-8 hours. There were 7 variables including cement type (type I and type III portland cements), cement content (716-885 lb/yd^3), w/c ratio (0.36-0.4), coarse aggregate type (siliceous and carbonate), accelerator type (calcium chloride and non-chloride), water reducer type (type E and type F) and curing temperature (73-150°F). Main evaluation tests were AASHTO T 161 (Resistance of concrete to rapid freezing and thawing), ASTM C672 (Deicer scaling resistance of concrete), and microscopic examinations of the concrete materials. The study concluded that in general the concrete obtained in the field and that produced in the laboratory were of good quality. Interactions between mixture constituents, for example type III cement and type F high range water reducer (HRWR) appeared to cause poorly formed air void systems and insufficient air contents in hardened concrete. This occurred more often in the 6-8 hour mixtures than the 20-24 hour mixtures. Less homogeneous paste and more microcracking was observed in the faster–setting mixtures. Scaling and dilation were significantly higher in the 6-8 hour mixtures than in the 20-24 hour mixtures. The problems appeared to be more severe in mixtures using type III cement and type F HRWR. No general conclusion was drawn regarding these constituents due to limited sources of materials used in the study.

2.4 EMERGING TECHNOLOGIES

2.4.1 Internal Curing

Internal curing of concrete has been developed in the last 20 years as a promising technology to improve concrete longevity and durability. The American Concrete Institute (ACI) defines internal curing as a “process by which the hydration of cement continues because of the availability of internal water that is not part of the mixing water”[11]. Among a variety of materials including superabsorbent polymers and pre-wetted wood fibers, the use of pre-wetted lightweight aggregate as internal water reservoirs in the concrete mixture is the most common material used for internal curing in the US[12]. Key potential benefits of using internal curing in concrete pavement include reduced shrinkage, permeability, and curling, improved strength, and less cracking. Internal curing can improve concrete fatigue capacity and longevity, and thus has the potential to reduce the life cycle costs of pavement[13]. Internal curing has been used in field trials of concrete bridge deck and pavement including full depth pavement repairs[13 14].

One concern in using internally cured concrete is a potential decrease in freeze-thaw resistance as the additional internal curing water may increase moisture content of the concrete and the higher porosity of the lightweight aggregates provides additional space for water to fill in the long term. Jones and Weiss (2015)[15] conducted an experimental study on freeze-thaw durability of internally cured concrete in which pre-wetted lightweight fine and coarse aggregates were utilized. This study investigated 12 concrete mixtures with w/cm of 0.36-0.56 and having lightweight aggregate content ranging from zero to 40.9% of the total mixture volume. Evaluation test was the *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing* (ASTM C 666, procedure A). When the amount of pre-wetted fine aggregate provided the curing water volume just equivalent to chemical shrinkage, it was observed that the internally cured concrete had satisfactory F-T durability, comparable to that of conventional aggregate concrete mixtures. In mixtures that used high volumes of pre-wetted lightweight aggregate (12.7% or more) or high w/cm ratio (w/cm = 0.56), however, significant F-T damage occurred. In practice, premature F-T deterioration can be a concern in late fall construction as freezing may occur shortly after casting and water in the aggregates do not have time to desorb.

Internally cured concrete may be a viable solution in pavement rapid repair as it can reduce early-age shrinkage cracking resulting from the fast hydration of high early strength concrete and has the potential to reduce life cycle costs. Possible negative effects of internal curing on F-T durability as mentioned above, however, deserve more study.

2.4.2 Precast concrete pavement

Precast concrete pavement (PCP) systems have been implemented by many highway agencies including Caltrans, New York DOT, New Jersey DOT, and Illinois Tollway as an effective solution to full-depth pavement repair and rehabilitation, especially in highway corridors with high traffic volume[16]. Key advantages of PCP are reduction of the required time for traffic closure, better quality control and longer performance life of the products, and possibly lower life-cycle costs. Another potential benefit of PCP is that the field installation of PCP is less dependent on weather conditions, thereby allowing extension of the construction season into late fall or even winter, while the construction of cast-in-place pavement is typically not allowed in these seasons in cold climate regions due to a high potential for premature freeze-thaw deterioration. Challenges in implementing PCP include higher initial costs and the lack of a robust competitive environment[16].

CHAPTER 3. SUMMARY OF FIELD REVIEW

The research team worked with WisDOT and the Wisconsin Concrete Pavement Association (WCPA) to select and conduct field reviews of 12 recent pavement repair projects that utilized high early strength concrete (HES) (Table 1). These projects were selected from a list of rehabilitation paving projects provided by the WCPA based on one or more of the following criteria:

- There is no asphalt overlay on the segments of interest.
- High early strength concrete (HES) or Special high early strength concrete (SHES), as specified in WisDOT Standard Specifications section 416.2.4, was utilized.
- Project locations and repair mixture type are available for the repaired segments.

The field survey followed recommendations of the FHWA-RD-03-031 "*Distress Identification Manual for the Long-Term Pavement Performance Project*"[1]. The repair segments were typically inspected for 12 types of distress including different forms of cracking, scaling, map cracking, and spalling. Main results from the field review are summarized below:

- The most common distresses were spalling of longitudinal or transverse joints (5 projects), longitudinal cracking (3 projects), scaling (2 projects), corner break (2 projects), transverse cracking (1 project), and map cracking (1 project).
- In general, serious concrete surface durability/performance issues were not observed. Severe scaling was observed at only one project (project #7).
- A correlation between pavement conditions and project data could not be established for multiple reasons: 1) key pieces of data such as mix design and names of admixtures were missing from the WisDOT project database, even in some of the most well-documented cases; 2) multiple generations of repair present in the same pavement sections were not clearly distinguishable; and 3) many repaired sections were diamond ground or covered with asphalt.

Detailed results of the field survey are provided in Appendix 1.

Table 1. Summary of WisDOT projects surveyed

Project #	Completion year	Project ID	Region	County	Route		Description
1	2011	1050-01-63	NW	Chippewa	STH	029	CHIPPEWA FALLS - CADOTT
2	2011	1050-01-64	NW	Chippewa	STH	029	CHIPPEWA FALLS - CADOTT
3	2013	1178-08-60	NC	Lincoln	USH	51	CTH K - CTH S
4	2010	4125-07-71	NE	Brown	USH	141	MAIN STREET, VILLAGE OF BELLEVUE
5	2011	1420-11-70	NE	Fond du Lac	USH	045	N MAIN STREET, CITY OF FOND DU LAC
6	2010	1440-16-60	NE	Sheboygan	STH	023	PLYMOUTH - SHEBOYGAN, CTH P -STH32
7	2014	1001-01-62 or 1003-10-86	SW	Rock	IH	39/90	JANESVILLE - MADISON
8	2013, 2014	1001-01-62, 1001-02-64, 1010-00-73, 1001-06-73	SW	Dane	IH	39/90	STOUGHTON - MADISON
9	Multiple generations	1016-00-65 1016-00-63 1016-00-61 1016-05-78	SW	Juneau/Sauk	IH	90/94	STH 33 - Wisconsin Dells
10	Multiple generations	1390-04-84, 1390-04-86, 1390-04-94/95	SW	Jefferson	STH	26	Fort Atkinson, NB
11	2010	1206-00-73	SW	Dane	USH	12/18	South Madison Beltline
12	2013	4075-31-71	NE	Outagamie	STH	96	WCL - APPLETON (STH 96)

CHAPTER 4. INFORMAL SURVEY OF CONCRETE SUPPLIERS

Although not part of the original work plan, we sought to contact six Wisconsin concrete suppliers of rapid-repair concrete to respond to an informal survey as follows:

- Lycon Inc.
- Zignego Ready Mix Inc
- Carew Concrete and Supply Co.
- Croell Redi Mix
- Bard Materials
- Schmitz Ready Mix Inc

Contact was eventually established with 4 of the 6 and a 15-to-20 minute phone discussion was structured to obtain information on their mix designs, challenges in projects of this type and general approaches to full-depth rapid-repair projects.

Their responses, both solicited and unsolicited, are summarized as follows:

- Mix design
    - No one claimed to be using Type III cement for full-depth rapid-repair projects and no one expressed any desire to use Type III cement for these projects. Both availability of the cement type and allocation of material silo space were identified as significant barriers to incorporating Type III cement.
    - Most (if not all) are using 9-bag (846 lb/yd^3) cement contents with calcium chloride accelerator. With 9-bag cement content as a base mix design, calcium chloride is generally needed to achieve 3000 psi in 8 hours. From there each supplier has a variety of customizations they do for particular situations. Some use one or more of heated water, high-range water reducer, non-chloride accelerators and other admixtures to achieve strength. Some advocated for the use of structural fibers to hold the mix together and inhibit cracking.
- Challenges

- Several mentioned difficulty in controlling air content but added that this not strictly limited to full-depth rapid-repair projects. Timing limitations (45 minutes to discharge) and small concrete quantities restrict the ability to make air entrainment adjustments that other types of projects allow.
- All agree that procedural and policy issues are the greatest difficulties in supplying concrete for full-depth rapid repairs. Difficulties stemming from these issues include:
  - Rapid hydration and concrete setting up in the truck before discharge.
  - Very tight timing between traveling to a site, discharging the load and achieving strength within the required timeline (45 minutes from water addition to discharge, etc.).
  - Profitability concerns predominated the conversations relating to allocation of personnel and equipment for limited-time night construction that may make that personnel and equipment unavailable the following day. Some mentioned they are hesitant to even submit bids for this type of construction.
  - Perhaps not surprising given the group, no one mentioned known durability and performance problems with these materials.

CHAPTER 5. LABORATORY TESTING

5.1 TESTING PROGRAM

5.1.1 Mix Matrix

Thirteen mixes* were made to evaluate the effects of coarse aggregate type, water/cement (w/c) ratio, slump, accelerator type and curing scheme (Table 2) on concrete strength development, drying shrinkage, and scaling resistance. Mixes 2 and 6 were selected as base mixtures that meet WisDOT specifications for Special High Early Strength (SHES) repair concrete and would likely perform satisfactorily in a freeze-thaw test. Mixes 1, 3, 4, 5, 7, and 8 were deviant mixtures to evaluate the effect of accelerator type and addition method.

Table 2. Mix matrix

Mix #	Coarse Aggregate	w/c	Slump	Accelerator	Curing scheme for C672 specimens
1	Crushed limestone	0.32	Normal	None	Standard[(a)]
2	Crushed limestone	0.32	Normal	$CaCl_2$ solution	Standard
3	Crushed limestone	0.32	Normal	$CaCl_2$ dry	Standard
4	Crushed limestone	0.32	Normal	Non-chloride	Standard
5	Igneous gravel	0.32	Normal	None	Standard
6	Igneous gravel	0.32	Normal	$CaCl_2$ solution	Standard
7	Igneous gravel	0.32	Normal	$CaCl_2$ dry	Standard
8	Igneous gravel	0.32	Normal	Non-chloride	Standard
9	Igneous gravel	0.32	Normal	$CaCl_2$ solution	Short[(b)]
10	Igneous gravel	0.32	High	$CaCl_2$ solution	Short
11	Igneous gravel	0.36	Normal	$CaCl_2$ solution	Short
12	Igneous gravel	0.36	High	$CaCl_2$ solution	Short
13	Igneous gravel	0.32	Normal	$CaCl_2$ solution	Curing compound[(c)]

Notes: (a)-Covered with wet burlap and plastic sheet for 24 hours, then in wet room until 14 days, and air-cured for 14 days; (b)-Covered with wet burlap and plastic sheet for 4 hours, then air-cured until 28 days; (c)-Curing compound applied 45 min. after fabrication, then air-cured until 28 days

* The approved work plan specified evaluation of 12 concrete mixes

It was originally envisioned that the field studies would dictate the mixes to be evaluated in laboratory testing. For reasons described previously, this proved to be infeasible. Details of the mixing sequence and accelerator addition methods are provided later in this report. Mixes 9-13 were deviants of mix 6 where the effects of curing, w/c ratio, and slump were examined. The target slump was 3 to 6 inches for normal slump mixes and 7 to 10 inches for high slump mixes. In mix 13, two sets of scaling specimens were made to evaluate potential incompatibility between curing compounds and a mixture containing calcium chloride and its consequence for scaling resistance; strength development and shrinkage were not investigated.

Base mix parameters were selected based on the WisDOT specifications, availability of materials in Wisconsin, and performance of materials in previous studies. The following parameters were the same for all the mixes:

- The target air content was 6±1 % as measured by the pressure method (AASHTO T 152).
- Type I portland cement from a single source was used at a rate of 846 lb/yd^3, which is the minimum amount required for SHES repair concrete by WisDOT specifications and consistent with what all the surveyed suppliers indicated they used for their mix design. All previous repair projects in Wisconsin that were reviewed in this study used Type I cement. As reported in an informal survey in CHAPTER 4, concrete suppliers in Wisconsin expressed little interest in using Type III portland cement.
- The fine aggregate content was 50% by weight of total aggregate, which is higher than the 30-40% fine aggregate content commonly used in conventional concrete mixes. The higher content of fine aggregate was selected to improve workability, consistent with WisDOT mix design Grade E specifications.
- One synthetic air-entraining admixture (AEA) commonly used in WisDOT pavement projects was selected. This AEA is based on sodium alpha olefin sulfonates.
- One polycarboxylate-based high-range water reducing admixture (HRWR) was used. HRWR based on polycarboxylate ethers (PCE) was successfully used to reduce w/c ratio in previous research[17].

5.1.2 Curing and Testing

Except for mix 13, concrete from each mix was tested for compressive strength (AASHTO T 22), drying shrinkage (ASTM C 157), and deicer scaling resistance (ASTM C 672). Strength testing was conducted at 4, 6, 8, 10, 12, 24, and 36 hours and 28 days after mixing, unless noted otherwise. Each set of strength samples consisted of three 4x8-inch cylinders cured in a wet room at 73°F until testing. Air-dry shrinkage testing was conducted at the same times as the strength tests. Each set of shrinkage samples consisted of three 4x4x10-inch prisms which were covered with wet burlap and plastic sheeting at lab temperature until testing. After the first measurement, the prisms were cured in a conditioning chamber at 73°F and 50% relative humidity. In mix 13, only one set of cylinders was tested for compressive strength at 28 days; shrinkage was not evaluated because expansion of the scaling durability data was the desired objective.

Freeze-thaw (F-T) durability of the concrete was evaluated through the ASTM C672 *"Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals"*. The sample was cylindrical, having a diameter of 12 in and a thickness of 3 in. To contain deicer solution, a PVC dike with an inner diameter of 10 in was glued to the test surface, resulting in a test surface area of approximately 78 in^2. The sample was hand-rodded and the surface was finished using a 2x4-in wood screed. Each sample was cured following one of the schemes noted in Table 2. Specimens from mixes 1-8, which followed a standard curing scheme, were cured with wet burlap and plastic sheet covering for 24 hours, then demolded and cured in a wet room until 14 days and air-cured for another 14 days before testing. For mixes 9-12 (short curing), plastic sheet covering was shortened to 4 hours and then the samples were air-cured in the laboratory for 28 days. In mix 13, two sets of scaling specimens were sprayed with either linseed oil or poly-alpha-methylstryene- (PAMS) based curing compound 45 minutes after finishing of the samples. The amount of curing compound applied to each specimen was 14.9 grams, corresponding to an application rate of approximately 141 ft^2/gallon which is higher than the manufacturer's recommended rate of 200 ft^2/gallon for the two curing compounds. Scaling tests of all the samples started 28 days after fabrication. Each sample went through 60 cycles of freezing and thawing. Approximately 1/4-inch of 4% sodium chloride solution was maintained on the concrete surface throughout the test. At the end of every 5 cycles, the sodium chloride solution was rinsed off over a #200 sieve; the materials scaled off the concrete surface and retained on the sieve were collected, dried at 105°C for 24 hours and weighed. A visual rating of surface scaling was also recorded. The results were averaged over 3 samples.

5.1.3 Materials

5.1.3.1 Cement

Type-I portland cement from Lafarge (Alpena plant) was used for all the mixes. Main physical properties and chemical compositions of the cement were provided by the cement supplier and are presented in Table 3.

Table 3. Physical and chemical properties of cement provided by cement supplier

Specific Surface (Blaine)-ASTM C204	383 m^2/kg
Silicon Dioxide (SiO_2)	19.6%
Aluminum Oxide (Al_2O_3)	4.5%
Ferric Oxide (Fe_2O_3)	3.0%
Calcium Oxide (CaO)	64.1%
Magnesium Oxide (MgO)	2.3%
Sulphur Trioxide (SO_3)	2.7%
Ignition Loss	2.5%
Tricalcium Silicate (C_3S)	62.0%
Tricalcium Aluminate (C_3A)	7.0%
Equivalent Alkalis	0.6%

5.1.3.2 Coarse aggregates

Two types of coarse aggregate were used. The first aggregate was crushed limestone from Waukesha County, and the second one was igneous gravel from Eau Claire County in Wisconsin. Both aggregates had a nominal maximum size of 3/4 in and met the WisDOT specifications (501.2.5.4) for coarse aggregate size No. 1. Sieve analysis and properties of the aggregates are presented in Table 4 and Table 5.

Table 4. Sieve analysis of coarse aggregates

Sieve size	% Passing by weight		
	Crushed limestone	Igneous gravel	WisDOT Specs.
1 in	100.0	100.0	100
3/4 in	99.3	99.3	90-100
3/8 in	53.8	45.1	20-55
#4	5.9	5.5	0-10
#8	0.5	2.0	0-5
#200	0.2	0.8	≤ 1.5

Table 5. Coarse aggregate properties

Properties	Crushed limestone	Igneous gravel
Nominal maximum size (in)	3/4	3/4
Specific gravity, oven dry	2.72	2.66
Absorption (%)	0.94	1.25
P200 (%)	0.2	0.8

5.1.3.3 Fine aggregate

A single source of fine aggregate from Janesville Sand and Gravel pit in Rock County in Wisconsin was used for all the mixes. Particle sizes and other properties met WisDOT specifications (501.2.5.3) and are given in Table 6 and Table 7.

Table 6. Sieve analysis of fine aggregate

Sieve size	% Passing by weight	
	Sand	WisDOT Specs.
3/8 in	100.0	100
#4	99.4	90-100
#16	70.1	45-85
#50	17.5	5-30
#100	3.4	0-10
#200	1.5	≤ 3.5

Table 7. Fine aggregate properties

Specific gravity, oven dry	2.69
Absorption (%)	1.17
P200 (%)	1.5

5.1.3.4 Admixtures

The same water-reducing admixture and air-entraining admixture was used for all the mixes. The air-entraining admixture was synthetic and based on sodium alpha-olefin sulfonates. The water reducer was polycarboxylate-based and met the requirements of both ASTM C 494 Type A (Normal range) and Type F (High range) water-reducing admixtures. Trial batches showed that a low-range water reducer commonly used for ordinary concrete, significantly delayed rapid-repair mixture set time and strength development. As a result, it was not used in this study. Two types of accelerator were used. The first one was calcium chloride in the form of hydrated flakes with calcium chloride content of 77-80%. The dosage of pure calcium chloride was 2% of weight of cement, which corresponds to a flake dosage of 2.6% assuming that the flakes had a calcium chloride content of 77%. The second accelerator was a non-chloride calcium nitrate-based accelerator in a liquid form. Dosage of the non-chloride accelerator was 32 oz/100 lb cement, as determined through trial batches.

5.1.4 Mixing and sampling

The mixing and sampling process followed the *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory* ASTM C192 with a minor change of the remixing time after the rest period from 2 minutes to 3 minutes. This change was aimed at facilitating the addition of dry sodium chloride in mixes 3 and 7, and was applied to all mixes for consistency. As a result, the mixing process for all the mixes consisted of three periods: 3-minute mixing, 3-minute rest, and 3-minute remixing. First, coarse aggregate was added into the mixer and the mixer was turned on, followed by air-entraining admixture, fine aggregate, accelerator solution (if used), cement, and water reducer. All materials were kept at lab temperature before mixing except that water was drawn from the tap shortly before mixing resulting in reasonable consistent beginning temperatures of materials. Mixture temperatures are shown in Table 8.

Air-entraining admixture and water reducer were pre-mixed with water before adding to the mixer. For mixes using calcium chloride solution, the calcium chloride flakes were dissolved in mix water before added to the mixer. For mixes using dry calcium chloride, the flakes were added to the rotating mixer within the first minute of the 3-minute remixing period. There has been a desire by rapid repair-concrete contractors to use calcium chloride in a dry form instead of solution where the accelerator needs to be added at the job site. Addition of calcium chloride solution at the job site increased concrete slump, in some cases to above specification limits, and thus required the concrete to stay in the truck longer than needed otherwise, making it challenging to get the concrete out of the truck. The use of calcium chloride in a dry form in this study was an effort to mimic a field situation where the accelerator is added to the concrete mixing drum (truck) at the job site.

5.2 RESULTS

5.2.1 Fresh Concrete Properties

Due to the lab mixer's capacity, two batches designated as A and B were required for each mix except for mix 13. Generally five sets of 3 cylinders and one set of 3 shrinkage prisms were made in batch A; three sets of 3 cylinders and one set of 3 ASTM C672 specimens were made in batch B. Slump (AASHTO T 119), fresh air content (AASHTO T 152), and concrete temperature (AASHTO T 309) were measured immediately after completion of mixing; results are presented in Table 8. Overall cohesiveness of all the mixes was satisfactory with no segregation issues. In mixes using 2% calcium chloride, workability was lost very quickly and it was rather difficult to consolidate cylinders by hand-rodding about 40 minutes after water and cement came into contact.

Table 8. Fresh concrete properties

Mix #	Batch A			Batch B		
	Slump (in)	Air content (%)	Concrete temperature (°F)	Slump (in)	Air content (%)	Concrete temperature (°F)
1	3.50	5.9	74	4.00	6.3	73
2	6.00	5.2	77	3.75	6.2	79
3	3.00	6.5	76	3.00	6.3	76
4	3.00	5.1	75	4.50	6.5	75
5	3.75	7.1	74	4.50	6.8	74
6	5.25	7.3	78	3.25	6.8	78
7	3.75	5.4	78	4.75	6.3	76
8	6.50	6.9	71	5.00	6.9	73
9	4.25	6.1	81	4.00	6.3	81
10	8.50	7.2	80	8.50	6.7	79
11	6.00	5.9	79	6.00	5.3	78
12	9.25	5.2	78	9.25	5.4	78
13	4.00	6.7	82			

5.2.2 Concrete Compressive Strength

5.2.2.1 Compressive strength results of all mixes

Compressive strengths of all the mixes are given in Table 9 as the average of 3 samples. For mixes using calcium chloride, the first cylinders were tested at 4 hours after mixing. For other mixes, the first test was done at either 6 or 8 hours as the strength development was slower. The average strengths after 8 hours ranged from 837 and 4763 psi. Overall only the mixes using calcium chloride and a w/c ratio of 0.32 had average strengths that met the WisDOT requirement of 3000 psi at 8 hours for SHES concrete (mixes #2, 3, 6, 7, 9, and 10) as shown in Figure 1. The average strengths after 28 days ranged between 7463 and 10845 psi.

Table 9. Concrete average compressive strength of all mixes (psi)

Mix #	Concrete age							
	4h	6h	8h	10h	12h	24h	36h	28 days
1			940	1747	2580	4557	5343	7497
2	2433	3760	4763	5610	5940	7113	7993	10845
3	1930	3407	4277	4780	5147	6700	7433	9150
4		1320	2753	3537	4020	5467	6865	8767
5			837	1473	2007	3943	4807	7463
6	2140	3340	4090	4210	4583	5713	6177	8943
7	1813	2897	3503	4173	4617	5660	6210	9047
8		977	2023	3170	3320	4810	5867	7463
9	2373	3397	3923	4683	5093	5990	6567	9510
10	2387	3620	4350	4577	4823	6047	6663	9200
11	1230	2033	2577	3067	3357	4607	5033	7840
12	1400	2383	2900	3380	3893	4737	5430	8303
13								9643

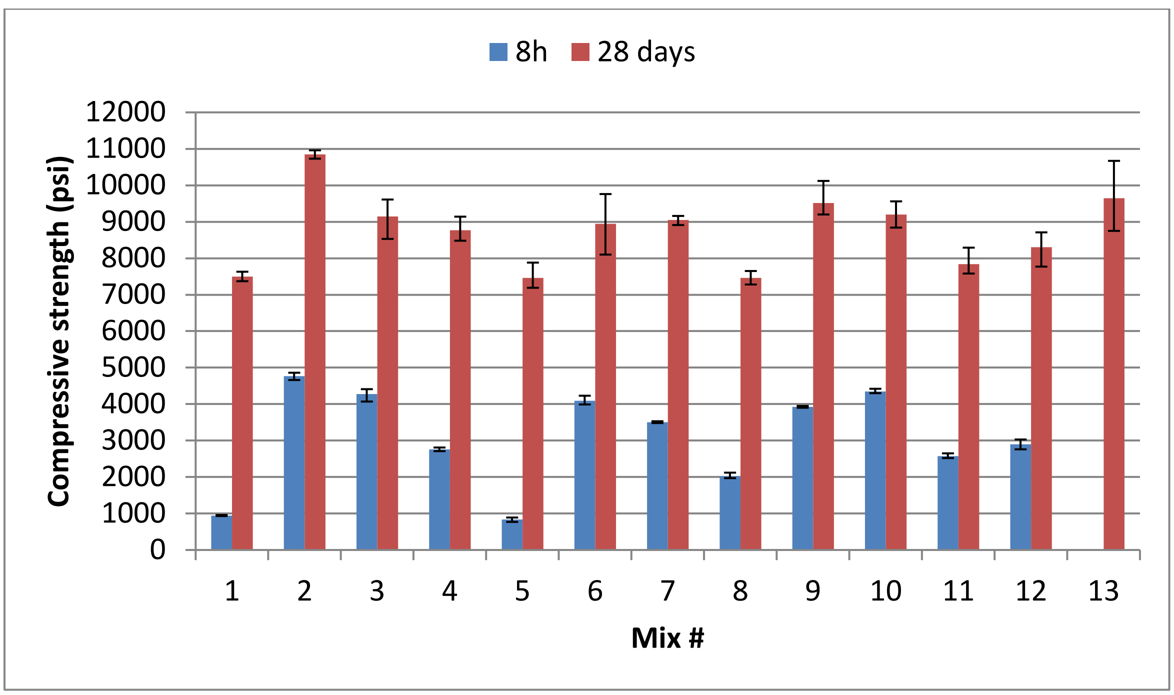

Figure 1. Concrete average compressive strengths (with max and min values) after 8 hours and 28 days

5.2.2.2 Effect of accelerators on strength development

Effect of accelerator type and addition method on concrete strength development in the first 36 hours is shown in Figure 2 for crushed limestone and Figure 3 for gravel mixes. In both cases, calcium chloride, dry or in solution, was more effective than the calcium nitrate based non-chloride accelerator. In the crushed limestone mixes, calcium chloride solution resulted in slightly faster strength gain than using dry flakes. In the gravel mixes, the difference in strength gain between the two addition methods was insignificant.

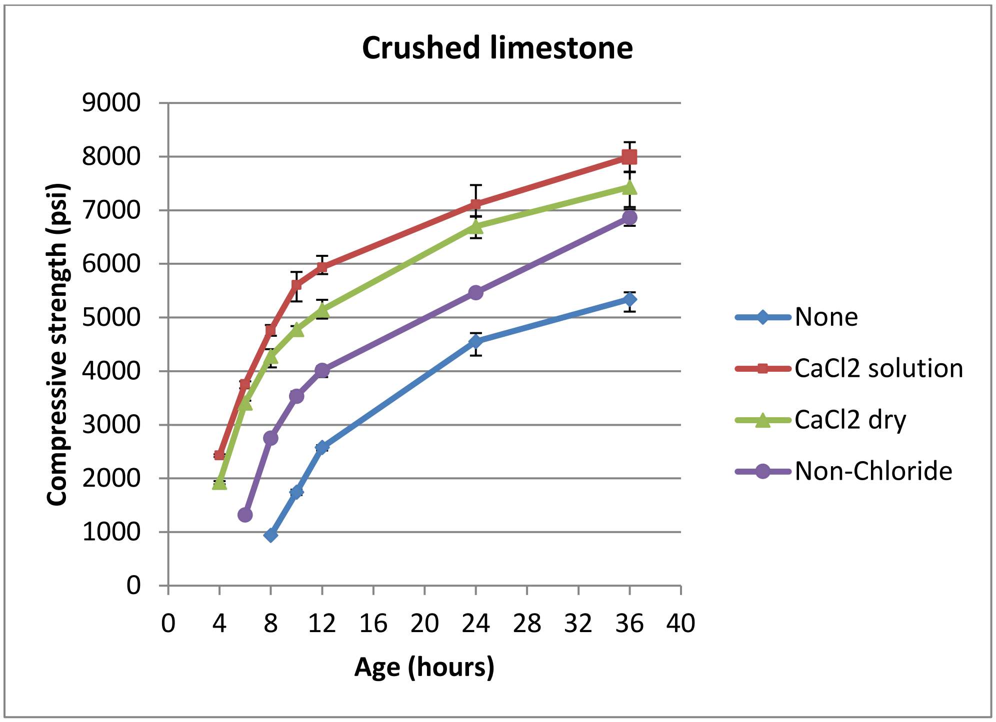

Figure 2. Effect of accelerator on concrete strength development, crushed limestone mixes 1-4, each data point shows average and range

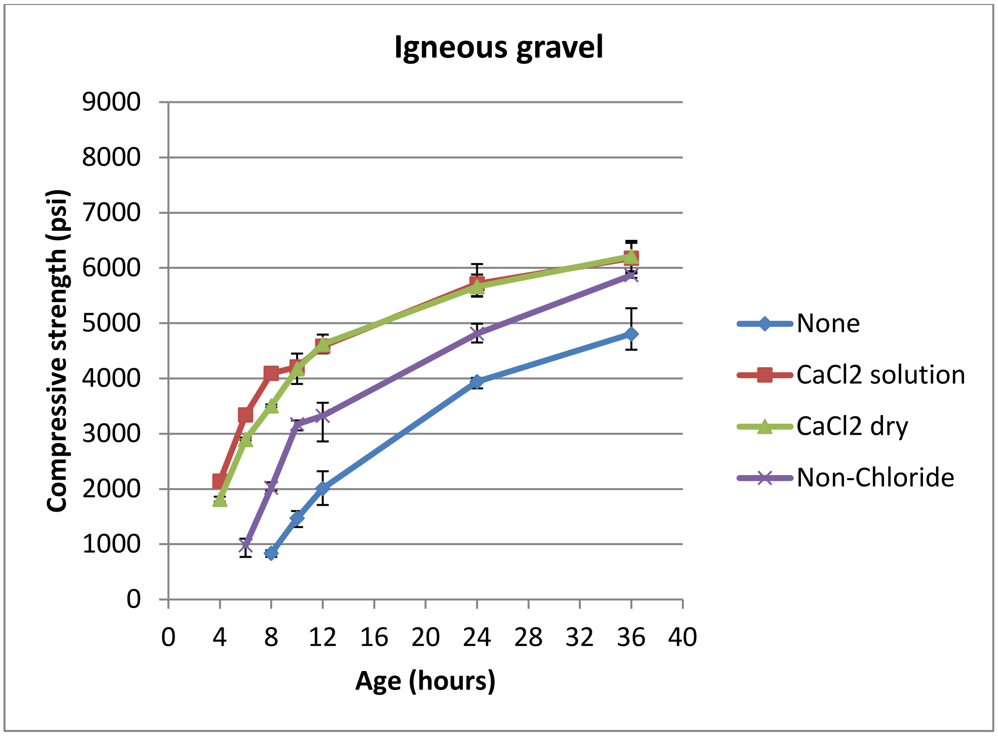

Figure 3. Effect of accelerator on concrete strength development, igneous gravel mixes 5-8, each data point shows average and range

5.2.2.3 *Effect of coarse aggregate type on strength development*

Strengths of specimens from crushed limestone and gravel mixes after 8 hours and 28 hours are compared side by side in Table 10. Regardless of the accelerator type and addition method, crushed limestone mixes had higher strengths than gravel mixes both after 8 hours and 28 hours. For repair concrete, using crushed limestone coarse aggregate may be particularly advantageous as the higher early strength would allow for shorter curing time before opening to traffic.

Table 10. Average concrete compressive strength (psi) of crushed limestone and gravel mixes after 8 hours and 28 days, mixes 1-8

Accelerator Condition	8 hour strength			28 day strength		
	Crushed limestone (psi)	Gravel (psi)	Difference (psi)	Crushed limestone (psi)	Gravel (psi)	Difference (psi)
None	940	837	103	7497	7463	33
$CaCl_2$ solution	4763	4090	673	10845	8943	1902
$CaCl_2$ dry	4277	3503	773	9150	9047	103
Non-Chloride	2753	2023	730	8767	7463	1303

5.2.2.4 Effect of w/c and slump on strength development

Effect of w/c on concrete strength development is shown in Figure 4 for mixes 9-12 which used gravel coarse aggregate and calcium chloride. Increasing the w/c from 0.32 to 0.36 significantly reduced concrete strengths in the first 36 hours. For example strengths at 8 hours of mixes 11 and 12 were 2577 and 2900 psi respectively, which were about 33% lower than those of mixes 9 and 10 (3923 and 4350 psi respectively). Increasing slump by using more water reducer as in mixes 10 and 12, meanwhile, did not considerably lower the concrete strengths when compared with mixes 9 and 11 respectively.

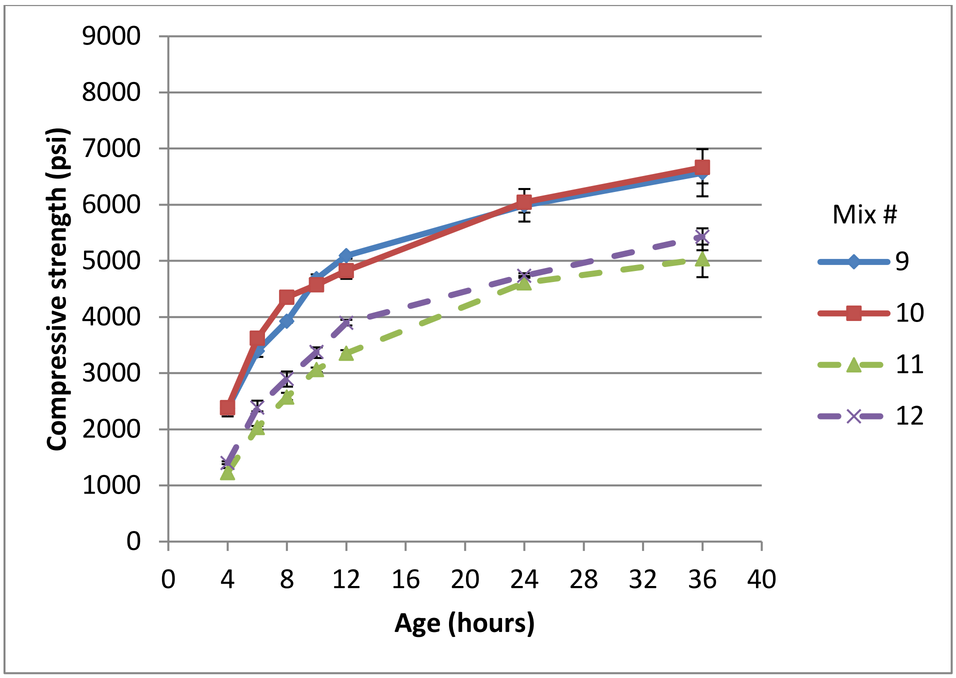

Figure 4. Effect of w/c on concrete strength development, solid lines for w/c=0.32, dashed lines for w/c = 0.36, each data point shows average and range

5.2.3 Shrinkage

Average concrete drying shrinkage values at different ages are provided in Table 11. Each shrinkage value was the average of 3 samples. For mixes using calcium chloride, the samples were demolded 4 hours after mixing. For other mixes, the samples were demolded between 6 and 10 hours after mixing as strength development was slower. The initial lengths were measured immediately after the samples were demolded. Subsequent length measurements were taken at the same times as strength testing. The shrinkage values of all mixes after being air-cured for 2 hours, 4 hours and 28 days are compared in Table 12. Shrinkage values after 28 days of mixes not containing calcium chloride were between 600 and 700 x 10^{-6}. Shrinkage after 28 days of most of the mixes containing calcium chloride was between 800 and 1000 x 10^{-6}. The higher shrinkage of the mixtures in this study compared to the typical range of 400 to 800 x 10^{-6} as reported by Kosmatka and Wilson[18] was likely because of the higher cement contents and the presence of accelerators in the rapid repair mixtures.

Table 11. Average concrete drying shrinkage (10^{-6} in/in)

Mix #	Concrete age							
	4h	6h	8h	10h	12h	24h	36h	28 days
1				demolded	110	143	200	610
2	demolded	230	267	297	317	413	450	853
3	demolded	253	297	313	347	467	503	943
4			demolded	130	153	190	230	683
5			demolded	97	83	130	157	633
6	demolded	240	283	300	317	393	463	933
7	demolded	217	260	283	310	400	463	987
8			demolded	87	97	133	163	683
9	demolded	193	223	247	263	343	397	877
10	demolded	183	203	213	223	297	347	747
11	demolded	217	260	280	293	353	407	913
12	demolded	200	233	263	277	343	390	863

Table 12. Average concrete drying shrinkage (10^{-6} in/in) after being air-cured (73°F, 50% RH)

Mix #		Mix characteristics			Time of air curing		
	Coarse aggregate	Accelerator	w/c	Slump	2 hours	4 hours	28 days
1	Crushed S.	None	0.32	Normal	110	(*)	610
2	Crushed S.	$CaCl_2$ solution	0.32	Normal	230	267	853
3	Crushed S.	$CaCl_2$ dry	0.32	Normal	253	297	943
4	Crushed S.	Non-Chloride	0.32	Normal	130	153	683
5	Gravel	None	0.32	Normal	97	83	633
6	Gravel	$CaCl_2$ solution	0.32	Normal	240	283	933
7	Gravel	$CaCl_2$ dry	0.32	Normal	217	260	987
8	Gravel	Non-Chloride	0.32	Normal	87	97	683
9	Gravel	$CaCl_2$ solution	0.32	Normal	193	223	877
10	Gravel	$CaCl_2$ solution	0.32	High	183	203	747
11	Gravel	$CaCl_2$ solution	0.36	Normal	217	260	913
12	Gravel	$CaCl_2$ solution	0.36	High	200	233	863

(*): no measurement

Effect of accelerator on shrinkage

For both crushed stone and gravel coarse aggregates, the use of calcium chloride accelerator significantly increased drying shrinkage both in early hours and leading to 28 days. For crushed stone mixes after 2 hours of air curing, in comparison with mix 1 (average shrinkage = 110 x 10^{-6}), the average shrinkage values of mixes 2 and 3 were 230 and 250 x 10^{-6} respectively, corresponding to increases of 109 and 130%. After 28 days, the average shrinkage values of mixes 2 and 3 were 853 and 943 x 10^{-6} respectively, corresponding to increases of 40 and 55% compared to mix 1 (average shrinkage = 610 x 10^{-6}). For gravel mixes after 4 hours of air curing, in comparison with mix 5 (average shrinkage = 83 x 10^{-6}), the average shrinkage values of mixes 2 and 3 were 283 and 260 x 10^{-6} respectively, corresponding to increases of 240 and 212%. After 28 days, the average shrinkage values of mixes 6 and 7 were 853 and 943 x 10^{-6} respectively, corresponding to increases of 47 and 56% compared to mix 5 (average shrinkage = 633 x 10^{-6}). The above results are in line with previous studies, for example, Rixom and Mailvaganam[19] reported test results by Bruere et al. (1971) showing that calcium chloride resulted in higher shrinkage at all times. The average drying shrinkage value at 28 day of mixes using 2% calcium chloride, having cement content of 514 lb/yd^3 (305 kg/m^3) and w/c of 0.6 was 450 x 10^{-6}, about 45% higher than that of mixes using no accelerator.

Use of the non-chloride accelerator generally increased shrinkage but the effect was much smaller than that of calcium chloride. For crushed stone aggregate, the average shrinkage of the non-chloride accelerator mix (mix 4) after 2 hours and 28 days was 130 and 683 x 10^{-6} respectively corresponding to increases by 18 and 12% compared to mix 1. For gravel aggregate, average shrinkage of mix 8 after 4 hours and 28 days was 97 and 683 x 10^{-6} respectively corresponding to increases by 16 and 8% compared to mix 5. The shrinkage value of mix 5 at 2 hours (97 x 10^{-6}) was an outlier as it was higher than both the shrinkage value of the same mix at 4 hours (83 x 10^{-6}) and that of mix 8 at 2 hours (87 x 10^{-6}). This value was likely an experimental error and thus was not used to compare with corresponding data of other mixes.

5.2.4 Concrete Scaling Resistance

Scaling test results of all mixes after 60 cycles of freezing and thawing are provided in Table 13. Each result value is the average of 3 samples. Although both scaled-off mass and visual rating are presented, the authors believe that the former was more objective and thus used for all analyses in this section. The visual ratings were rounded to the nearest integers and provided for reference purposes only. While no standard for an acceptable scaled-off mass is specified in the US, the maximum mass loss permitted after 56 cycles of freezing and thawing is 500 g/m^2 in Quebec, Canada and 1000 g/m^2 in Sweden[20]. According to Pigeon and Pleau[21], concrete that has scaled-off mass not exceeding 1000 g/m^2 after 50 cycles is generally considered to have adequate scaling resistance. A recent study conducted by Iowa State University (Hooton and Vassilev[22]) provided a summary of three variations of the ASTM C672 test which differ in the type and concentration of deicer chemical, finishing technique, curing regime, and acceptance limits for cumulative mass of scaled-off materials. The scaled-off mass limits after 50 cycles varied between 500 and 800 g/m^2. Assuming an acceptance limit of 500 g/m^2 after 60 cycles, all the mixes in this research performed satisfactorily except for the samples cured with PAMS curing compound (mix 13-P). The study by Hooton and Vassilev[22] also provided an equivalence chart relating visual rating to mass loss and characteristic of scaling surface, which agrees fairly well with the ratings and mass losses in this research. This chart was used to provide the qualitative assessment in Table 14 in Chapter 6 (Economic Analysis) of this research.

Table 13. ASTM C672 average results of all mixes after 60 cycles

Mix #	Coarse aggregate	w/c	Slump	Accelerator	Curing scheme	Scaled-off mass (g/m^2)	Visual rating (0-5)
1	Crushed stone	0.32	Normal	None	Standard	465	3
2	Crushed stone	0.32	Normal	$CaCl_2$ solution	Standard	264	2
3	Crushed stone	0.32	Normal	$CaCl_2$ dry	Standard	92	1
4	Crushed stone	0.32	Normal	Non-chloride	Standard	1	0
5	Gravel	0.32	Normal	None	Standard	144	1
6	Gravel	0.32	Normal	$CaCl_2$ solution	Standard	47	1
7	Gravel	0.32	Normal	$CaCl_2$ dry	Standard	314	2
8	Gravel	0.32	Normal	Non-chloride	Standard	43	1
9	Gravel	0.32	Normal	$CaCl_2$ solution	Short	205	1
10	Gravel	0.32	High	$CaCl_2$ solution	Short	48	1
11	Gravel	0.36	Normal	$CaCl_2$ solution	Short	139	1
12	Gravel	0.36	High	$CaCl_2$ solution	Short	500	1
13-L	Gravel	0.32	Normal	$CaCl_2$ solution	Linseed oil	291	2
13-P	Gravel	0.32	Normal	$CaCl_2$ solution	PAMS	1329	3

Effect of accelerator and coarse aggregate type on concrete scaling resistance

Scaling mass loss values after 60 cycles for different accelerator and coarse aggregate types are compared in Figure 5. Overall, the use of accelerators improved concrete scaling resistance. The concrete using non-chloride accelerator had the best performance for both crushed limestone and gravel mixes, with scaling mass losses of less than 50 g/m^2. Compared with concrete with no accelerator, liquid-based calcium chloride increased scaling resistance for both coarse aggregate types; whereas calcium chloride added in a dry form increased scaling resistance in the crushed limestone mix, but decreased the scaling resistance in the gravel mix. It is not clear why the concrete using dry calcium chloride performed worst among the gravel mixes.

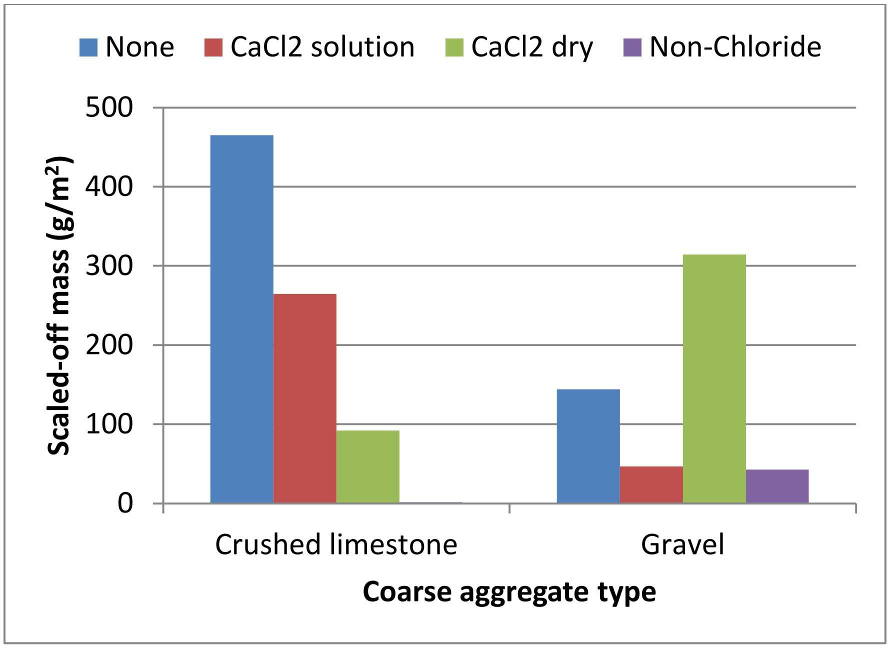

Figure 5. ASTM C672 average test results after 60 cycles, mixes 1-8

Effect of aggregate type on concrete scaling resistance in this study was mixed. For concrete without accelerator or with calcium chloride solution, gravel mixes performed better than crushed limestone ones; whereas for concrete using dry calcium chloride or non-chloride accelerator, the trend was opposite (Figure 5).

Effect of curing on concrete scaling resistance

Effect of curing regime is presented in Figure 6, which shows the cumulative scaling mass loss for mix 6 (standard curing), mix 9 (short curing), mix 13-L (linseed oil curing compound), and mix 13-P (PAMS curing compound). All these mixes used gravel coarse aggregate and calcium chloride solution. Each value is the average of 3 samples. The data shows that standard-cured samples (14 days of wet curing) had the best performance. It is interesting that short-cured samples, which were covered by sheeting for only 4 hours, also exhibited good scaling resistance with scaling mass loss of about 200 g/m^2 after 60 cycles. Performances of the samples using linseed oil and PAMS curing compounds were remarkably different. While linseed oil was effective with scaling mass loss of 291 g/m^2, PAMS specimens performed poorly with scaling mass loss of 1329 g/m^2. PAMS curing compound has been used in many studies at UW Madison[23 24]. This compound had high solid fraction and was highly effective in restricting water loss and improving concrete scaling resistance in comparison with other curing compounds such as linseed oil and wax[23 24].

The poor performance of PAMS specimens in this study suggests that an incompatibility between PAMS and calcium chloride or other mixture constituents may exist although other causes are not excluded.

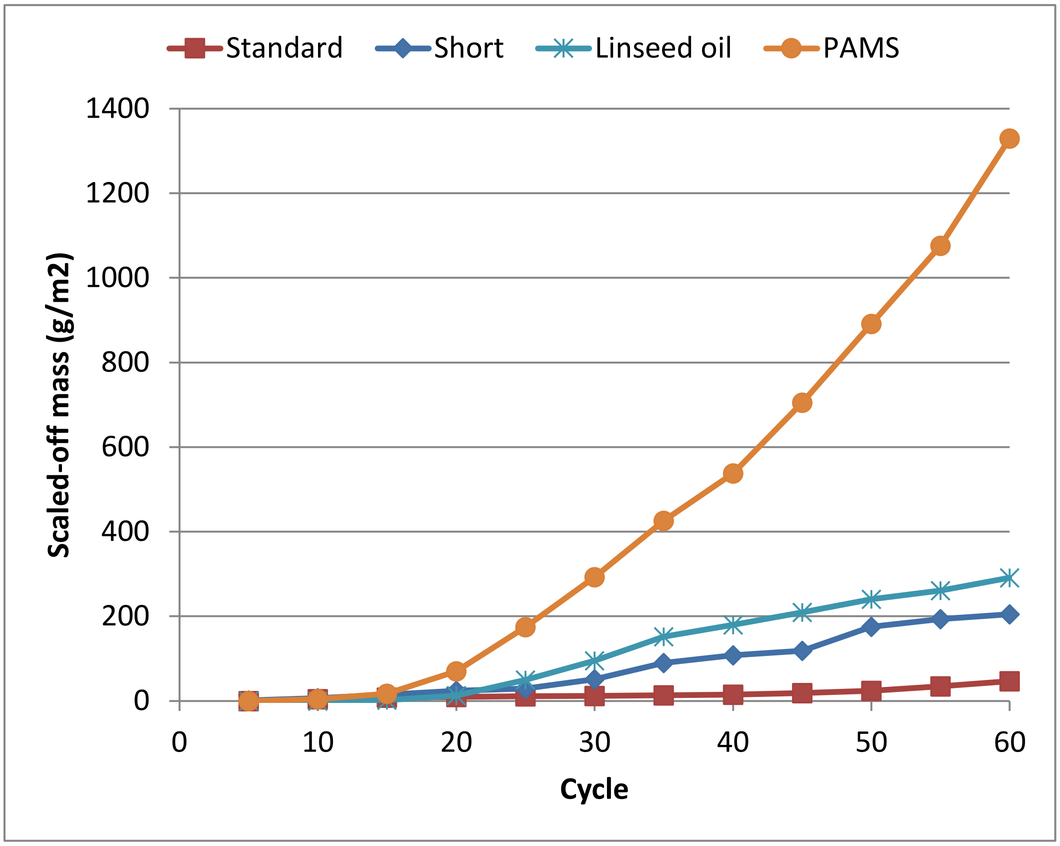

Figure 6. Effect of curing on scaling resistance, mixes 6, 9, 13-L, and 13-P

Effect of w/c ratio and slump on concrete scaling resistance

Effect of w/c ratio and slump on scaling resistance was investigated through mixes 9-12 as shown previously in Table 13. Among these four mixes, the best performer was mix 10 (w/c = 0.32 and high slump) and the worst performer was mix 12 (w/c = 0.36 and high slump). This result suggests that better scaling resistance requires not only a lower w/c ratio but also sufficient workability. On one hand, a lower w/c ratio resulted in higher strength and denser paste, both of which contribute to improving scaling resistance. On the other hand, lower w/c combined with the use of calcium chloride accelerator and superplasticizer led to faster loss of workability and finishability. Raising both w/c ratio and slump (mix 12) resulted in considerably lower scaling resistance.

CHAPTER 6. ECONOMIC ANALYSIS

6.1 GENERAL

The economic analysis described herein provides an estimate of the relative life-cycle costs associated with the use of the rapid-repair concrete mixtures that were evaluated in the lab portion of this study, and for precast concrete pavement repairs. The following economic analysis is, by necessity, somewhat speculative. It is likely that none of the mixtures tested in the lab portion of this study have ever been exactly replicated in the field, and it is even more likely that, if they were, the locations and performance records of the field construction are not available.

Furthermore, it must be recognized that there is potential for significant variability in construction quality, material properties, environmental conditions and loading conditions that, in turn, will result in significant variability in performance of any rapid-repair installation. The extent of that variability has not been sufficiently characterized to allow confident use of stochastic or "probabilistic" economic analysis techniques (i.e., we don't know the appropriate distribution [e.g., normal, uniform, etc.] to apply to the expected service lives of these materials, nor do we know the appropriate parameters for the selected distribution [e.g., mean and standard deviation for an assumed normal distribution] for use in a probabilistic life cycle cost analysis).

Within this context, a deterministic economic analysis has been performed in the spirit of the guidance provided in Wisconsin DOT Facilities Development Manual 14-15 "Pavement Type Selection" (Jan 13, 2017). Table 1.2 (Types of Pavement Work) of this document classifies "concrete pavement repair" as "Resurfacing", and Table 1.1 (LCCA Requirement Criteria) states that LCCA is required for "Resurfacing" projects $\geq$ 5 centerline roadway miles or more in length. Therefore, the following analyses assume that the repairs being considered are being applied on large projects that require an LCCA.

The scope of work for this project included a benefit-to-cost (B/C) analysis of the various rapid-repair mixtures. However, the most recent version of WisDOT FDM 14-15 states the following in Section 10.1 (Life-Cycle Cost Analysis (LCCA) Process): "The LCCA is a process for comparing alternatives over a specified period of time. During this time, each alternative has expenditures for initial construction, maintenance and rehabilitation. The expenditures are converted to *present worth costs* (emphasis added) and then summed together." This is the approach taken in this analysis. It should be pointed out that present worth analyses are direct and always present consistent economic rankings of alternative options with B/C, equivalent uniform annual cost/worth, rate of return, and other economic analysis techniques. In addition, it can be assumed that all repair materials provide the same benefits (i.e., provide an acceptable roadway surface for the traveling public), although their costs may be significantly different (due to initial cost differences, service life differences, etc.). This concept is backed by WisDOT FDM 14-15 Section 10.2 (The LCCA Parameters), which states that "The LCCA is used only to analyze pavement-related costs for each alternative." Therefore the consideration of other economic measures, such as benefits and user costs, are considered to be equivalent among alternatives and, therefore, negligible in this analysis.

WisDOT FDM 14-15 Section 10.3 (The LCCA Computation) states that "the LCCA calculation is performed using standard engineering economic analysis procedures for computing present worth costs." The following sections describe the determination of appropriate inputs for that analysis and, ultimately, the results of the analyses.

6.1.1 Analysis Period

Section 10.3 (The LCCA Computation) states that WisDOT policy uses a 50-year analysis period for new pavement construction and reconstruction. Instructions are also provided for using linearly prorated rehabilitation costs to credit to the "Total Facility Costs" when rehabilitation cycles extend beyond the analysis period. Given that the specified 50-year analysis period includes a 25-to-31-year performance period for the initial concrete pavement structure (for undrained and drained structures, respectively, a 19-to-25-year analysis period for the rapid-repair activities is consistent and appropriate.

FHWA LCCA guidelines recommend the use of an analysis period that includes the initial performance period and at least one rehabilitation activity. For the purposes of the analyses performed here, an analysis period of 25 years was selected to allow the consideration of at least one rehabilitation activity (repair replacement) for even long-lived repair options (i.e., those that might last 20 years, on average).

6.1.2 Discount Rate

Section 10.3 (The LCCA Computation) states that WisDOT policy uses a 5% discount rate. While this rate is excessive in the current economic climate and far exceeds current OMB Circular A-94 Appendix C recommendations for real interest rate on long-term investments, it is assumed that the 5% value has been adopted for justifiable reasons; therefore, a 5% discount rate is used in the analyses that follow. The sensitivity of the analyses to selected discount rate was evaluated by also performing the same analysis at discount rates of 3 percent and 7 percent.

6.1.3 Service Life

Table 10.6 "Rehabilitation Service Life" in section 10.2.5 (Pavement Design Service Lives) assigns a service life of 8 years to "concrete repair and grind" (it is noted previously in the document that grinding is not always necessary). Section 10.2.5 further states that the service life for pavement rehabilitation (e.g., concrete repair and grind) is assumed to be unaffected by the presence or absence of a drained base material. A service life of 15 years is given to HMA overlay of JPCP.

It is assumed that the 8-year repair service life assigned to concrete repairs represents the average expected service life of all cast-in-place concrete repairs (including both conventional and rapid repairs) and that it reflects a wide range of service life values. This range is assumed to include service lives as short as 1 year or less (e.g., for repairs that exhibit immediate structural failure) to lives of 20 years or more for well-designed repairs that are well-constructed using durable materials.

For this project, a service life estimate has been made for each lab-tested rapid-repair material based on strength, shrinkage and scaling test results and the assumption of good construction practices. A low value of 4 years is assumed based on the fact that field project 7 exhibited "severe scaling" after 2 years but has not yet been removed from service; in other words, it appears that a non-catastrophic failure like scaling is likely to shorten the expected service life but will not result in immediate replacement).

Table 14 provides a summary of strength, shrinkage and scaling information for each test mixture, along with the assumed service life for each mixture for the purposes of performing this economic analysis. Mix 13 presented in Table 2 was an additional mix where concrete strength and shrinkage were not evaluated and thus was excluded from the economic analysis in this chapter.

The service life of precast repairs has not been well-established, but is expected to be 20 years or more for well-designed and -constructed precast repairs that have been fabricated with durable materials. The oldest precast panels placed using modern designs, materials and construction techniques were placed at the Tappan Zee toll plaza near New York City in 2001; they have performed well and remain in service today. Heavy vehicle simulator testing of precast panels in California suggested that those particular panels would carry more than 140 million ESALs and last more than 40 years before failure (Kohler et al, 2007). With this background, a service life range of 20 to 40 years was selected for precast repairs in this study.

6.1.4 Repair Life Cycle Assumptions

WisDOT FDM 14-15 Table 10.3 presents an assumed pavement life cycle for jointed plain concrete pavement (JPCP) with dowels and includes the initial construction, two repair activities (with grinding, if necessary), one repair and HMA overlay activity, and then reconstruction.

The analysis performed in this study uses the rehabilitation portion of the WisDOT FDM 14-15 Table 10.3 assumed pavement life cycle except that a second repair-and-overlay activity is included for any repair material with an assumed service life of less than 5 years. This was done to avoid having any material require reconstruction inside of the 25-year analysis period, which would have required introducing potentially overwhelming assumptions concerning the cost of reconstruction. When the impact of service life greater than the 25-year analysis period was evaluated (for the precast repairs), linear depreciation of the initial cost was assumed in the residual value at year 25 and no further rehabilitation was considered; these assumptions were made for simplicity in the consideration of sensitivity to performance life, even though they are not strictly compliant with good LCCA practices.

6.1.5 LCCA Cost Assumptions

An analytical unit of one lane-mile was selected.

There are no significant differences in material and installation costs for the 12 rapid-strength repair materials considered in the laboratory portion of this study (i.e., none of the admixture costs are believed to increase unit material costs outside of the range of normal variations in bid pricing). It is assumed for the purposes of this analysis that all the mixtures evaluated would be produced in a manner that does not significantly increase or reduce construction rates in the field. Thus, for the 12 mixtures evaluated in the project lab study, it is assumed that they all have essentially the same cost and that the only difference in the life-cycle costs of using them is due to differences in their projected service lives. The cost of using precast concrete pavement repairs (which were not evaluated directly in this research) has been included for comparison and economic evaluation as well.

The costs of constructing either cast-in-place rapid-strength or precast repairs varies widely with project size (repair quantities), length of the daily construction windows, competition, traffic control issues and many other factors. Cost data for recent intermittent repairs on Wisconsin projects using both rapid cast-in-place and precast products are summarized in Table 15. Figure 7 presents a graph of the National Highway Construction Cost Index and shows a fairly level trend in 2013 – 2015, suggesting that the unit costs presented in Table 15 can be considered without adjustment for inflation.

Table 14. Summary of Test Results and Assigned Service Life for 12 Lab Mixtures

Mix No.	f'c, psi (8-hour/28-day)	Qual. Strength*	Shrinkage, με (2-hour/28-day)	Qual. Shrinkage	Scaling Mass Loss, g (30 cycles/60 cycles)	Qual. Scaling	**Assigned Service Life, years**	Comments
1	940/7500	--	110/610	++	400/465	---	**2**	Poor early-age strength, moderate scaling at early age.
2	4760/10845	++	230/850	-	15/264	-	**8**	Best overall strength, moderate long-term scaling
3	4280/9150	++	250/940	--	10/92	+	**20**	Good overall strength and scaling performance.
4	2750/9150	0	130/680	+	1/1	+++	**20**	Marginal early strength, excellent scaling performance.
5	840/7460	--	100/630	++	113/144	-	**2**	Worst early-age strength, slight scaling.
6	4090/8940	++	240/930	--	12/47	++	**20**	Good overall strength and scaling performance.
7	3500/9050	+	220/990	--	46/314	-	**8**	Good overall strength, moderate long-term scaling.
8	2020/7460	-	90/680	+	26/43	++	**8**	Below average early strength, very good scaling resistance.
9	3920/9510	++	190/880	-	51/205	0	**8**	Best overall strength, moderate long-term scaling
10	4350/9200	++	180/750	0	18/48	++	**20**	Good overall strength and scaling performance.
11	2580/7840	-	220/910	--	75/139	0	**6**	Substandard early strength and slight scaling.
12	2900/8300	0	200/860	-	140/500	--	**4**	Marginal early strength, slight-moderate scaling.

* WisDOT requires 3000 psi in 8 hours

Table 15. Summary of Unit Costs for Recent Wisconsin Projects Constructed Using both Rapid Cast-in-Place and Precast Full-Depth Repairs

2013 I-94 Hudson			2014 Madison Beltline			2015 Madison I-39		
Item	Quantity	Unit Price	Item	Quantity	Unit Price	Item	Quantity	Unit Price
Precast Repair	2340 s.y.	$345	10-in Precast Pvt Repair	3849 s.y.	$400	12-in Precast Pvt Repair	1645 s.y.	$495
			Special Sawing Precast Panel Install	17566 l.f.	$1.89	Special Sawing Precast Panel Install	8030 l.f.	$2.75
Special PCCP Repair Doweled	3770 s.y.	$159.09	SHES PCCP Repair	5112 s.y.	$150	PCCP Repair Overnight	2100 s.y.	$162.35
Sawing PCC	27876 l.f.	$2.10	Sawing PCC	61765 l.f.	$1.89	Sawing PCC	10353 l.f.	$2.75
Drilled Tie Bars	1340 ea	$8.00	Drilled Tie Bars	65304/ea	$6.00	Drilled Tie Bars	431 ea	$7.81
Drilled Dowels	7940 ea	$13.99	Drilled Dowels	25320/ea	$14.00	Drilled Dowels	7584 ea	$14.75
Estimated Unit Prices ($/s.y.), Installed*:			Estimated Unit Prices ($/s.y.), Installed*:			Estimated Unit Prices ($/s.y.), Installed*:		
Precast:		$345	Precast:		$428	Precast:		$527
Cast-in-Place (Overnight):		$190.97	Cast-in-Place (Overnight):		$180.36	Cast-in-Place (Overnight):		$197.74

*Per David Layton (WisDOT SW Region), precast repair prices for 2013 I-94 Hudson project were all-inclusive while drilled dowels and ties and sawing were bid separately for cast-in-place repairs. After that project, sawing and drilled dowels were bid separately for precast as well. WisDOT standards call for 8 drilled dowels per joint. There are no tie bars in precast installations, but they are spaced at 30 inches and held 15" from transverse joints and doweled repair ends for cast-in-place repairs. Per Mr. Layton, an average repair length of 8 ft was selected over the full lane width. Thus, repair unit costs were estimated assuming 10.66 s.y./repair, 16 drilled dowels in two transverse joints, 4 drilled tie bars (one longitudinal joint only for cast-in-place repairs, only) and 40 l.f. of sawing the full repair perimeter.

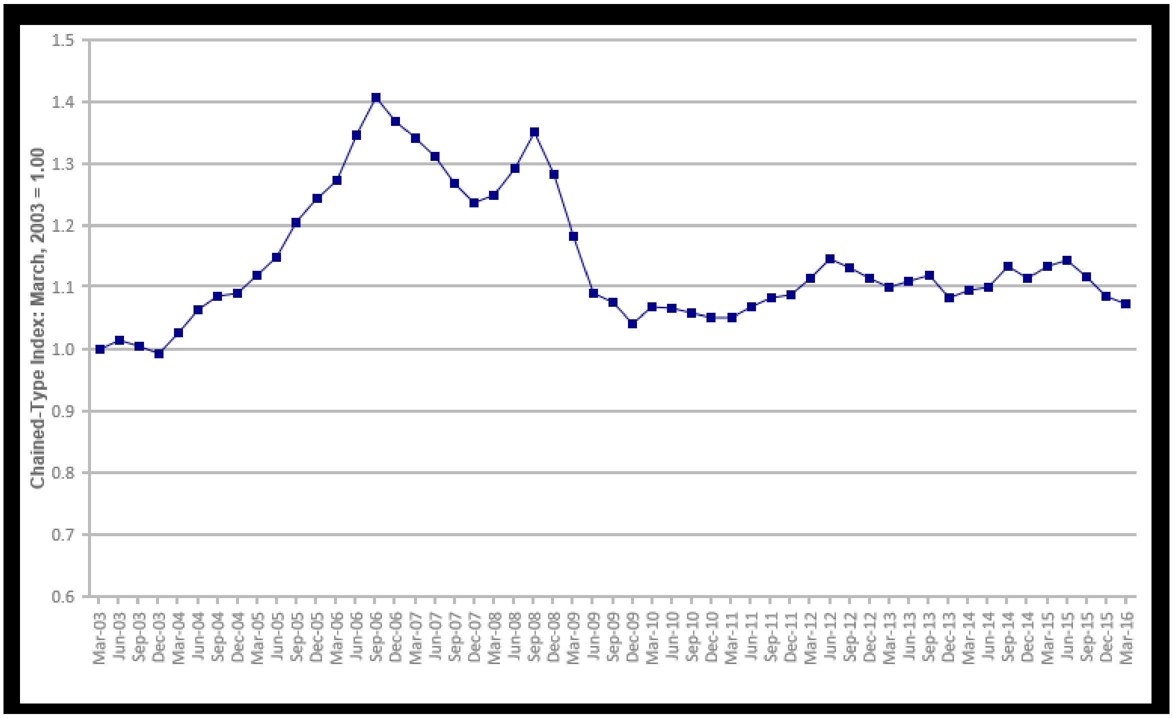

Figure 7. National Highway Construction Cost Index (March 2003 – March 2016) (Source: https://www.fhwa.dot.gov/policyinformation/nhcci/pt1.cfm - last updated August 2016)

The 2014 Madison Beltline project contained the largest repair quantities and presents values near the middle of the observed range; therefore, these unit costs were selected for use in the economic analysis presented here (i.e., \$428/s.y. for precast and \$180.36/s.y. for rapid repair). It should be noted that these values are below recent unit costs recently reported by the Illinois Tollway (approximately \$550/s.y. for precast and \$450/s.y. for 16-hour, 2500-psi accelerated cast-in-place repairs[25]), especially for the cast-in-place rapid repairs.

For simplicity, it has been assumed that each initial repair is the width of the travel lane (typically 12 ft) and 6 ft long, for a total of 8 s.y. of repair area – i.e., a typical joint repair. At the end of the initial service life, it is assumed that each repair is replaced with another repair using the same material, except that the replacement repair must be about 2 ft longer to remove the embedded dowels from the previous repair. (Note: removal along the original boundaries is possibly when using special dowels, such as the Fort Miller SuperDowels™, or other techniques that are not typically performed). Therefore, each subsequent repair at the same location will need to be about 2 ft longer than the previous repair (adding 2.66 s.y. to the area of each subsequent repair).

It is assumed that the initial repairs are performed at 25 percent of all transverse joints, which results in the replacement of 10 percent of the pavement area. For analytical simplicity, it is assumed that these are the only locations that will be repaired during the life cycle (although each repair is assumed to be replaced one or more times over the analysis period, as described previously).

It is assumed that the HMA overlay that is placed with the 3rd repair operation is nominally 4 inches thick. Estimated milling and overlay costs were determined based on WisDOT 2016 average bid data provided by MK Kang (WisDOT Pavement Engineer): 3MT 58-28 S (overlays $\geq$ 4 inches = \$77.30/ton; 4MT 58-28 S (overlays < 4 inches = \$74.75/ton, 110 lbs/sy-in for asphalt overlays, removing asphaltic surface – milling = \$4.35/s.y. Using the provided data, the cost of the 4-inch overlay was estimated at \$16.72/s.y.

6.2 ANALYSIS RESULTS

Table 16 summarizes total 25-year costs and discounted life-cycle costs (net present worth of costs) for the assumed 5% discount rate, as well as for 3% and 7% discount rates. This information is presented graphically in Figure 8. Detailed analyses are provided in tables in APPENDIX 2, which document representative present worth analyses conducted in support of Table 16 and Figure 8, performed using the input values described previously for various assumed service lives for the repair materials, along with some intermediate service lives, all using the WisDOT standard 5 percent discount rate. Tables for the 3 and 7 percent discount rates are similar, but are not presented (although their data are represented in Table 16 and Figure 8.

Table 16. Summary of Expected Costs (Total Costs and Discounted Costs) over 25-Year Analysis Period for Various Discount Rates.

Repair Type and Service Life	Total Cost	DR = 3%	DR = 5%	DR = 7%
Rapid 2	$786,514	$693,295	$639,392	$592,852
Rapid 4	$679,240	$574,688	$517,175	$468,426
Rapid 6	$581,692	$478,772	$423,723	$377,920
Rapid 8	$493,870	$402,931	$353,530	$312,791
Rapid 10	$406,048	$330,429	$290,194	$257,689
Rapid 15	$239,839	$208,687	$191,744	$177,937
Rapid 20	$169,298	$160,066	$153,284	$147,328
Precast 20	$401,749	$379,843	$363,749	$349,615
Precast 25	$301,312	$301,312	$301,312	$301,312
Precast 30	$251,093	$277,327	$286,482	$292,059
Precast 35	$215,223	$260,195	$275,890	$285,450
Precast 40	$188,320	$247,346	$267,945	$280,493

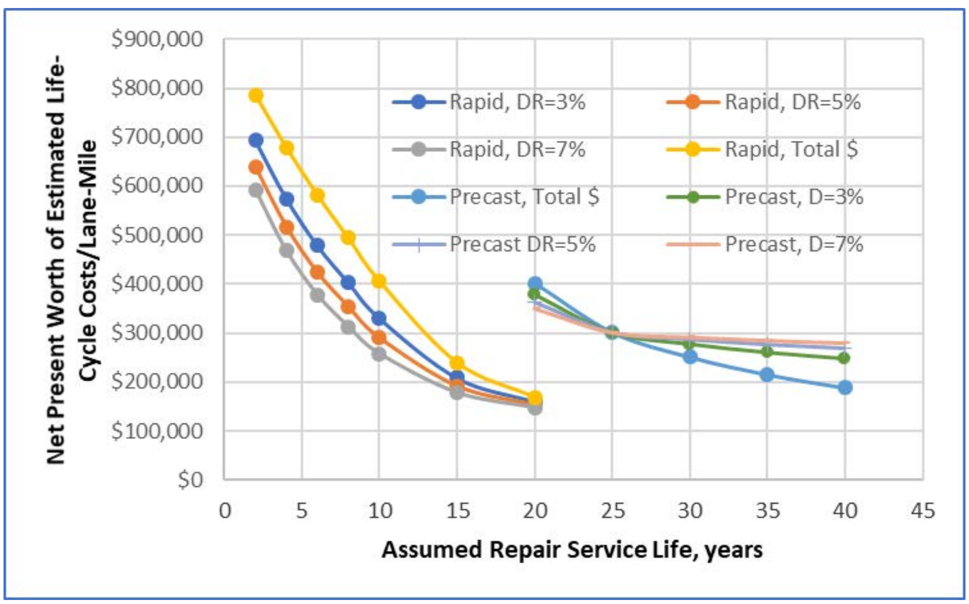

Figure 8. Net present worth of estimated costs vs. assumed repair service life using recent WisDOT construction cost data and various discount rates.

Discussion

There are no surprises in this analysis of materials that are all assumed to have the same initial cost within their respective repair type (i.e., rapid cast-in-place or precast). All differences in expected total costs or life-cycle (net present worth) costs between materials with similar initial costs can be assigned to expected differences in performance life and the assumed maintenance and rehabilitation cycles. In this study, mixtures 3, 4, 6 and 10 performed the best overall in the strength, shrinkage and scaling studies (especially mixture 4), while test results for mixtures 1, 5 and 12 indicate relatively short performance life potential.

The economic analyses performed here are based on numerous assumptions, not the least of which is that there is no actual field performance data available for any of the mixtures that were being evaluated. Therefore, these analyses should be considered to provide qualitative indications (rather than quantitative measures) of the likely relative costs associated with the mixtures that were evaluated.

With this context in mind, one could group the mixtures into 3 groups: those that are likely to have service lives that exceed current 8-year service life expectations (i.e., mixtures 3, 4, 6 and 10); those that are likely to provide very short service lives (i.e., 1, 5 and 12); and those that are likely to provide service lives that will approximately meet current 8-year expectations (i.e., mixtures 2, 7, 8, 9, and 11). Using life-cycle activities that are based on current WisDOT policy and the previously assumed activity costs, one can infer that using long-life repair materials offers the potential to reduce facility maintenance and rehab costs by nearly 50% over standard 8-year service life repairs (see Figure 8). Similarly, the use of poor quality materials with inadequate initial strength or long-term durability can result in approximately a doubling of facility maintenance and rehab costs when compared to those associated with standard 8-year service life repairs (see Figure 8).

Given the assumed pricing structure, precast concrete repairs with a 20-year service life have comparable anticipated life-cycle costs to cast-in-place rapid repairs with service lives of 6-to-8 years (depending on the assumed discount rate). When a 40-year service life is assumed for precast, the costs compare favorably to those of rapid repairs with service lives of 9-to-13 years (depending on the assumed discount rate). It must be noted that these comparisons are highly dependent on the difference in initial costs for rapid repairs and precast repairs and that the large difference observed in the selected values is not necessarily representative of the differences observed in other installations (e.g., the Illinois tollway, as described previously).

Finally, even with the price structure used in this analysis, the benefits of using longer-lived repair materials (e.g., 20 years or longer, whether rapid repairs or precast) far exceed those of using repair materials with short expected service lives (i.e., less than 6 years).

CHAPTER 7. SUMMARY, CONCLUSIONS, RECOMMENDATIONS

7.1 SUMMARY AND CONCLUSIONS

Field review:

A field review was conducted to evaluate the current performance of rapid-repair concrete mixes in Wisconsin and to relate durability issues, if any, to the project design and construction records. Twelve recent pavement rehabilitation projects in Wisconsin were surveyed. Overall the survey found no serious durability issue except for one project on I-90 where severe scaling was observed in many repaired panels. A correlation between the pavement conditions and project records could not be established.

Informal survey with concrete suppliers:

In addition to the field survey, the research team surveyed four Wisconsin suppliers of rapid-repair concrete to obtain information on their mix designs, challenges, and general approach to full-depth repair projects. None of the suppliers use type III portland cement for repairs; most are using 9-bag (846 lb/yd^3) cement mixes. Calcium chloride is generally used and required to achieve 3000 psi in 8 hours. All of the suppliers mentioned that procedural and policy issues are the greatest difficulties in supplying concrete for full-depth repairs.

Laboratory evaluation:

A total of 13 mixes were made to evaluate the effects of coarse aggregate type, w/c ratio, slump, accelerator type and curing scheme on concrete strength development, drying shrinkage, and scaling resistance. Conclusions drawn from the lab tests include:

- All the lab mixes exhibited generally acceptable levels of scaling resistance, with average scaling mass loss after 60 freeze-thaw cycles less than or equal to 500 g/m^2, except for mix 13, which used PAMS curing compound.
- Mixes using calcium chloride solution:

- Calcium chloride significantly increased concrete early strength. All the mixes using 2% calcium chloride in the form of solution and w/c of 0.32 reached 3000 psi within 6 hours, surpassing the WisDOT strength requirement of 3000 psi in 8 hours.
- Calcium chloride appeared to improve scaling resistance.
- Calcium chloride significantly increased drying shrinkage.

- Mixes using dry calcium chloride: Calcium chloride added in the form of dry flakes had similar effects to calcium chloride solution. Mixes using dry calcium chloride also surpassed the WisDOT strength requirement and performed satisfactorily in the scaling test.
- Effect of the non-chloride accelerator:
  - The non-chloride accelerator used in this research significantly increased concrete early strength, which reached 3000 psi within 10 hours but did not meet the WisDOT strength requirement of 3000 psi in 8 hours.
  - Concrete using the non-chloride accelerator had the best scaling resistance among the mixes in this research.
  - Drying shrinkage of specimens containing the non-chloride accelerator was slightly higher than mixes without accelerator and significantly lower than those using calcium chloride.
- Effects of curing, w/c ratio, and slump on scaling resistance:
  - Among different curing schemes, standard curing samples performed the best in the freeze-thaw scaling test. Samples subject to a 4-hour sheet-covered curing period and those cured with linseed oil curing compound exhibited acceptable scaling resistances. Samples with PAMS curing compound performed poorly in the scaling test, suggesting a possible incompatibility between the PAMS curing compound and calcium chloride or other mixture constituents.
  - Raising both the w/c ratio and slump significantly lowered scaling resistance.

Economic analysis:

- The economic analysis was performed using values that represent the estimated costs and service lives for cast-in-place and precast concrete pavement rapid repairs. The findings of the analysis provided qualitative rather than quantitative guidance because of the lack of actual field performance data for any of the repairs considered. As the initial costs of the twelve mixes were assumed to be the same, it is obvious that using a mixture of higher quality (i.e. having higher strength and scaling resistance and lower shrinkage) would be more cost-effective than using a mixture of lower quality.
- In general, precast concrete pavement has higher initial cost, but may be more durable than CIP rapid-repair concrete. Given the assumed pricing structure, precast concrete repairs with 20-year service life have comparable anticipated life-cycle costs to CIP rapid repairs with service life of 6-8 years depending on the assumed discount rate. When a 40-year service life is assumed for precast, the costs compare favorably to those of rapid repairs with service lives of 9-13 years depending on the assumed discount rate.

Overall:

- Based on the laboratory test results of this research, a rapid-repair concrete mixture that meets the current WisDOT specifications and has mix proportions similar to those used in this study, i.e. 846 lb/yd^3 type I portland cement, w/c ratio of 0.32, and 2% calcium chloride, would likely provide pavement repairs with satisfactory durability if properly constructed. Durability issues such as the premature scaling observed in the field review was more likely related to the construction or mix process than the underlying materials.
- Rapid-repairs consisting of cast-in-place concrete present a highly constrained construction problem. Limits in concrete delivery time compounded by limits in time to achieve strength and night time construction with small volumes of concrete create an environment where the vulnerability to variable field conditions increase and the typical robustness and care of mainline concrete paving cannot be achieved. By definition, the rapid-repair construction process can be rushed and every aspect must be accomplished within a tight time schedule. DOT records are not sufficiently complete and detailed to document these challenges but the interviews with the suppliers alluded to these difficulties. Better quality

assurance can be achieved by re-examining these constraints and changing the rapid-repair construction process.

7.2 RECOMMENDATIONS

One of the constraints in construction of pavement rapid repairs that was revealed through communications between the research team and the WCPA is related to the requirement of using calcium chloride in solution form. When calcium chloride solution must be added at the job site, this may significantly raise the slump to above the WisDOT's 4-inch limit, requiring the concrete to stay in the truck longer than needed otherwise and increasing the risk of concrete setting up in the truck. This constraint may be alleviated in two ways:

i. Using calcium chloride in dry form. Results from this research showed that concrete mixtures that had calcium chloride added in the form of dry flakes exhibited comparable performance compared with those using calcium chloride solution as long as the accelerator can be mixed uniformly with the concrete.
ii. Increasing the slump upper limit. WisDOT specifications currently do not have slump limits specifically for pavement repair using SHES concrete. Given unique features of this type of concrete such as having low w/c ratio and containing HRWR and accelerator, we recommend that the upper limit for SHES concrete be set higher than that for conventional concrete. For a mixture similar to those used in this research, an upper slump limit of 6 inches may help to ensure constructability while providing satisfactory performance.

In previous studies at UW Madison[23 24], PAMS has been shown an effective curing compound for typical concrete pavement mixtures. The poor scaling performance of samples coated with PAMS curing compound in this research suggests that an incompatibility of this curing compound and other mixture constituents may exist although other causes are not excluded. Further investigation into this problem is recommended before its use is allowed for rapid repairs.

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APPENDIX 1. FIELD REVIEW

A 1. Summary

The research team worked with WisDOT and the Wisconsin Concrete Pavement Association (WCPA) to select and conduct a field review of 12 pavement repair projects that utilized high early strength concrete (HES) in the last several years (Table 17). These projects were selected from a list of rehabilitation paving projects provided by the WCPA based on one or more of the following criteria:

- There is no asphalt overlay on the segments of interest.
- High early strength concrete (HES) or Special high early strength concrete (SHES) was utilized.
- Information is available for field review such as locations, mix type of the repaired segments and remarkable deterioration.

The field review followed recommendations of the FHWA-RD-03-031 "Distress Identification Manual for the Long-Term Pavement Performance Project". The repair segments were typically inspected for 12 types of distress including different forms of cracking, scaling, map cracking, and spalling. The main results from the field review are as follows:

- The most common distresses were spalling of longitudinal or transverse joints (5 projects), longitudinal cracking (3 projects), scaling (2 projects), corner break (2 projects), transverse cracking (1 project), and map cracking (1 project).
- Overall, widespread durability/performance issues were not observed. Severe scaling was observed at only one project (project #7).
- A correlation between the pavement conditions and project records could not be established for multiple reasons: WisDOT project database is not complete with key pieces of data missing, multiple generations of repair in the same pavement sections were not discernible, and many repaired sections were diamond ground or covered with asphalt.

Table 17. Summary of WisDOT projects surveyed

Project #	Completion year	Project ID	REGION	COUNTY	ROUTE		DESCRIPTION	Coarse Aggregate
1	2011	1050-01-63	NW	Chippewa	STH	029	CHIPPEWA FALLS - CADOTT	Crushed stone
2	2011	1050-01-64	NW	Chippewa	STH	029	CHIPPEWA FALLS - CADOTT	Crushed stone
3	2013	1178-08-60	NC	Lincoln	USH	51	CTH K - CTH S	Gravel No.1
4	2010	4125-07-71	NE	Brown	USH	141	MAIN STREET, VILLAGE OF BELLEVUE	Crushed stone No.1
5	2011	1420-11-70	NE	Fond du Lac	USH	045	N MAIN STREET, CITY OF FOND DU LAC	Crushed stone
6	2010	1440-16-60	NE	Sheboygan	STH	023	PLYMOUTH - SHEBOYGAN, CTH P -STH32	Gravel & Crushed stone, No. 1
7	2014	1001-01-62 or 1003-10-86	SW	Rock	IH	39/90	JANESVILLE - MADISON	Gravel
8	2013, 2014	1001-01-62, 1001-02-64, 1010-00-73, 1001-06-73	SW	Dane	IH	39/90	STOUGHTON - MADISON	n/a
9	multiple generations	1016-00-65 1016-00-63 1016-00-61 1016-05-78	SW	Juneau/Saulk	IH	90/94	STH 33 - Wisconsin Dells	Crushed stone
10	multiple generations	1390-04-84, 1390-04-86, 1390-04-94/95	SW	Jefferson	STH	26	Fort Atkinson, NB	Gravel
11	2010	1206-00-73	SW	Dane	USH	12/18	South Madison Beltline	n/a
12	2013	4075-31-71	NE	Outagamie	STH	96	WCL - APPLETON (STH 96)	n/a

A 2. Details of field review

Project 1 (1050-01-63, CHIPPEWA FALLS - CADOTT (STH 29))

A total of 11 patches with sizes ranging from 5 yd^2 to 18 yd^2 were surveyed. These patches were not diamond ground and appeared in satisfactory condition. No scaling or map cracking was observed. There were one medium-severity longitudinal crack and a few low-severity spalled joints. The crack in the repaired patch seems aligned with longitudinal cracks in the two adjacent slabs and appears to be consequence of lack of base support rather than a material issue. Summary of distress is provided in Table 18.

Table 18. Distress survey summary of project 1

Distress No.	Distress Type	Severity Level		
		Low	Moderate	Severe
1	Corner break			
2	Durability cracking/D-Cracking (Number of affected slabs)			
	Area affected (ft2)			
3	Longitudinal cracking (length, ft)		12.0	
	Length sealed		0	
4	Transverse cracking (No. of cracks)			
	Length of crack, ft			
5a.	Transverse joint seal damage (Sealed?)			
5b.	Longitudinal joint seal damage (Sealed)(length of damage, ft)			
6	Spalling of longitudinal joints (length, ft)	5.0		
7	Spalling of transverse joints (No. of affected joints)	4		
	Length spalled, ft	6.8		
8a	Map cracking (Number)			
	Area affected (ft2)			
8b	Scaling (Number)			
	Area affected (ft2)			
9	Polished aggregate (Area, ft2)			
10	Popouts (Number per ft2)			
11	Blowup/Crushed patch (number)			
12	Faulting of transverse joints and cracks			

(Blank means no distress was observed in the segments under review)
(Gray means that the distress was not reviewed)

Typical patch	Medium-severity longitudinal crack
	

Figure 9. Pictures of typical repaired segments – Project 1 (1050-01-63)

Project 2 (1050-01-64, CHIPPEWA FALLS - CADOTT (STH 29))

A total of 34 patches were surveyed. These patches were not diamond ground and in good condition. No scaling or map cracking was observed. There were two patches having a medium-severity longitudinal crack and a few medium-severity spalled joints. The crack in the repaired patches seems aligned with longitudinal cracks in the two adjacent slabs and appears to be consequence of lack of base support rather than a material issue. Summary of distress is provided in Table 19.

Table 19. Distress survey summary of project 2

Distress No.	Distress Type	Severity Level		
		Low	Moderate	Severe
1	Corner break			
2	Durability cracking/D-Cracking (Number of affected slabs)			
	Area affected (ft2)			
3	Longitudinal cracking (length, ft)		12.0	
	Length sealed			
4	Transverse cracking (No. of cracks)			
	Length of crack, ft			
5a.	Transverse joint seal damage (Sealed?)			
5b.	Longitudinal joint seal damage (Sealed)(length of damage, ft)			
6	Spalling of longitudinal joints (length, ft)		10.0	
7	Spalling of transverse joints (No. of affected joints)			
	Length spalled, ft			
8a	Map cracking (Number)			
	Area affected (ft2)			
8b	Scaling (Number)			
	Area affected (ft2)			
9	Polished aggregate (Area, ft2)			
10	Popouts (Number per ft2)			
11	Blowup/Crushed patch (number)			
12	Faulting of transverse joints and cracks			

(Blank means no distress was observed in the segments under review)
(Gray means that the distress was not reviewed)

Typical patch	Medium-severity longitudinal crack
	

Figure 10. Pictures of typical repaired segments – Project 2 (1050-01-64)

Project 3 (1178-08-60, US 51, Merrill – Tomahawk)

A total of 7 repaired segments having lengths from 20 ft to 330 ft were surveyed. All sections except for one were diamond ground. The inspected sections were in satisfactory condition with a few spalling of transverse joints and corner breaks. No scaling or map cracking was observed. Summary of the distress is provided in Table 20.

Table 20. Distress survey summary of project 3

Distress No.	Distress Type	Severity Level		
		Low	Moderate	Severe
1	Corner break	6		
2	Durability cracking/D-Cracking (Number of affected slabs)			
	Area affected (ft2)			
3	Longitudinal cracking (length, ft)			
	Length sealed			
4	Transverse cracking (No. of cracks)			
	Length of crack, ft			
5a.	Transverse joint seal damage (Sealed?)			
5b.	Longitudinal joint seal damage (Sealed)(length of damage, ft)			
6	Spalling of longitudinal joints (length, ft)			
7	Spalling of transverse joints (No. of affected joints)	4	1	
	Length spalled, ft	4.5	9	
8a	Map cracking (Number)			
	Area affected (ft2)			
8b	Scaling (Number)			
	Area affected (ft2)			
9	Polished aggregate (Area, ft2)			
10	Popouts (Number per ft2)			
11	Blowup/Crushed patch (number)			
12	Faulting of transverse joints and cracks			

(Blank means no distress was observed in the segments under review)
(Gray means that the distress was not reviewed)

Typical patch	Spalling of transverse joint
	

Figure 11. Pictures of typical repaired segments – Project 3 (1178-08-60)

Project 4 (4125-07-71, US 141, MAIN STREET, VILLAGE OF BELLEVUE)

A total of 19 repaired segments having lengths from 20 ft to 90 ft were surveyed. Most of the sections were diamond ground. Distress included transverse cracking, spalling of longitudinal and transverse joints, and map cracking as summarized in Table 21. Pictures of typical repaired segments are provided in Figure 12.

Table 21. Distress survey summary of project 4

Distress No.	Distress Type	Severity Level		
		Low	Moderate	Severe
1	Corner break			
2	Durability cracking/D-Cracking (Number of affected slabs)			
	Area affected (ft2)			
3	Longitudinal cracking (length, ft)			
	Length sealed			
4	Transverse cracking (No. of cracks)		6	
	Length of crack, ft		71	
5a.	Transverse joint seal damage (Sealed?)			
5b.	Longitudinal joint seal damage (Sealed)(length of damage, ft)			
6	Spalling of longitudinal joints (length, ft)	19.0		
7	Spalling of transverse joints (No. of affected joints)	10		
	Length spalled, ft	40		
8a	Map cracking (Number)	2		
	Area affected (ft2)	8		
8b	Scaling (Number)			
	Area affected (ft2)			
9	Polished aggregate (Area, ft2)			
10	Popouts (Number per ft2)			
11	Blowup/Crushed patch (number)			
12	Faulting of transverse joints and cracks			

(Blank means no distress was observed in the segments under review)
(Gray means that the distress was not reviewed)

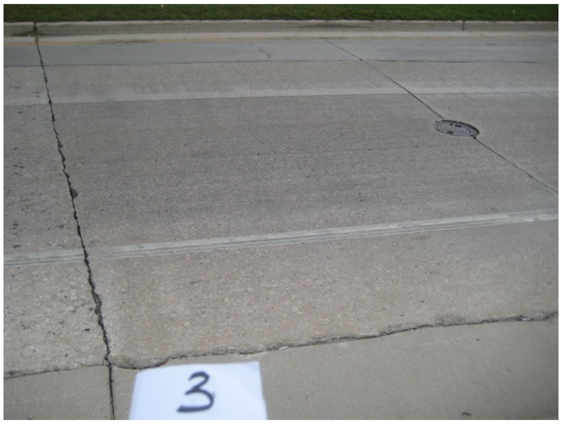
Typical patch	Medium-severity transverse crack
	

Figure 12. Pictures of typical repaired segments – Project 4 (4125-07-71)

Project 5 (1420-11-70, US 45, N MAIN STREET, CITY OF FOND DU LAC)

The entire length of this project is 2800 ft in which 100-yd^2 SHES concrete was used at the intersection of USH 45 and Scott Street. This SHES concrete repair section was not discernible from surrounding pavement. The entire pavement at the intersection was in good condition with no distress.

Figure 13. Pictures of typical repaired segments – Project 5 (1420-11-70)

Project 6 (1440-16-60, STH 32, STH 32 – STH 23 Intersection)

A total of 41 repaired segments having lengths from 6 ft to 140 ft were surveyed. The segments appeared to belong to several generations some of which used gravel coarse aggregate while the others used crushed stone. Most of the sections were diamond ground. Distress included corner break, longitudinal cracking, and spalling of longitudinal and transverse joints as summarized in Table 22. The medium-severity longitudinal cracks as the one shown in Figure 14 appeared to be consequence of severe deterioration of the adjacent panels that likely led to lack of load transfer. Pictures of typical repaired segments are provided in Figure 14.

Table 22. Distress survey summary of project 6

Distress No.	Distress Type	Severity Level		
		Low	Moderate	High
1	Corner break	1		
2	Durability cracking/D-Cracking (Number of affected slabs)			
	Area affected (ft2)			
3	Longitudinal cracking (length, ft)		21.0	
	Length sealed			
4	Transverse cracking (No. of cracks)			
	Length of crack, ft			
5a.	Transverse joint seal damage (Sealed?)			
5b.	Longitudinal joint seal damage (Sealed)(length of damage, ft)			
6	Spalling of longitudinal joints (length, ft)	3.0		
7	Spalling of transverse joints (No. of affected joints)	21		2
	Length spalled, ft	54		12
8a	Map cracking (Number)			
	Area affected (ft2)			
8b	Scaling (Number)			
	Area affected (ft2)			
9	Polished aggregate (Area, ft2)			
10	Popouts (Number per ft2)			
11	Blowup/Crushed patch (number)			
12	Faulting of transverse joints and cracks			

(Blank means no distress was observed in the segments under review)
(Gray means that the distress was not reviewed)

Typical patch	Medium-severity longitudinal crack
	

Figure 14. Pictures of typical repaired segments – Project 6 (1440-16-60)

Project 7 (I39/90 – West bound - near the intersection of I90 with State Hwy 59)

The WisDOT Project ID is 1003-10-86 or 1001-01-62. The repairs were done in 2014 and surface scaling with visible coarse aggregate was observed in patches near the intersection of I90 with State Hwy 59 (Figure 15). These mixes used gravel coarse aggregates.

Typical patch	Scaled-off surface
	

Figure 15. Pictures of typical repaired segments – Project 7 (I39/90)

Project 8 (I39/90 – West bound, Stoughton - Madison)

There were several generations (multiple projects) of patch repairs in this stretch of roadway. Probable WisDOT Project IDs are 1001-01-62, 1001-02-64, 1010-00-73, or 1001-06-73. It was not entirely clear which patches were associated precisely with which project and which generation of repair. The project records show that these repairs were made between 2012 and 2014. At projects 1001-01-62 and 1001-02-64, slump was allowed up to 9 in and the actual values varied from 3 to 8 in. Generally these repairs appeared to be performing satisfactorily.

Project 9 (I90 – West bound - State Hwy 33 to Wisconsin Dells exits)

WisDOT Project IDs are 1016-00-65, 1016-00-63, 1016-00-61, or 1016-05-78. At least three generations of patch repairs are present in this stretch of highway and were generally discernible. In the older patches, some aggregate particles were visible, likely from scaling. It appeared that crushed stone coarse aggregate were used in these patches.

Typical patch	Typical patch
	

Figure 16. Pictures of typical repaired segments – Project 9 (I90)

Project 10 (STH 26 near Fort Atkinson, US-12 and State Hwy 89)

The patches likely belonged to WisDOT Project ID 1390-04-84, 1390-04-86 or 1390-04-94/95 and appeared to be performing satisfactorily. Gravel coarse aggregate was used in the patches.

Typical patch	Medium-severity transverse crack
	

Figure 17. Pictures of typical repaired segments – Project 10 (STH 26)

Project 11 (1206-00-73, USH 12/18 South Madison Beltline, Fish Hatchery Road-Verona Road)

The road has been covered with asphalt. No observation was possible.

Project 12 (4075-31-71, STH 96, WCL - APPLETON)

The road has been covered with asphalt. No observation was possible.

APPENDIX 2. ECONOMIC ANALYSIS TABLES

Table 23. Present Worth of Cost Analysis for CIP Rapid Repairs with 20-Year Service Life (assumed to be applicable to lab mixtures 3, 4, 6 and 10) – 5% Discount Rate.

20-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2			0.907	-
3			0.864	-
4			0.823	-
5			0.784	-
6			0.746	-
7			0.711	-
8			0.677	-
9			0.645	-
10			0.614	-
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15			0.481	-
16			0.458	-
17			0.436	-
18			0.416	-
19			0.396	-
20	2nd Repair	$169,298	0.377	63,807
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$296,271**		**190,780**
25	Pro-rata residual of last activity:	-$126,973	0.295	(37,496)
	TOTALS:	**$169,298**		**153,284**

Table 24. Present Worth of Cost Analysis for Precast Repairs with 20-Year Service Life – 5% Discount Rate.

20-year Precast Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$301,312	1.000	301,312
1			0.952	-
2			0.907	-
3			0.864	-
4			0.823	-
5			0.784	-
6			0.746	-
7			0.711	-
8			0.677	-
9			0.645	-
10			0.614	-
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15			0.481	-
16			0.458	-
17			0.436	-
18			0.416	-
19			0.396	-
20	2nd Repair	$401,749	0.377	151,415
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$703,061**		**452,727**
25	Pro-rata residual of last activity:	-$301,312	0.295	(88,978)
	TOTALS:	**$401,749**		**363,749**

Table 25. Present Worth of Cost Analysis for CIP Rapid Repairs with 15-Year Service Life – 5% Discount Rate.

15-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2			0.907	-
3			0.864	-
4			0.823	-
5			0.784	-
6			0.746	-
7			0.711	-
8			0.677	-
9			0.645	-
10			0.614	-
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15	2nd Repair	$169,298	0.481	81,435
16			0.458	-
17			0.436	-
18			0.416	-
19			0.396	-
20			0.377	-
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$296,271**		**208,409**
25	Pro-rata residual of last activity:	-$56,433	0.295	(16,665)
	TOTALS:	**$239,839**		**191,744**

Table 26. Present Worth of Cost Analysis for CIP Rapid Repairs with 10-Year Service Life – 5% Discount Rate.

10-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2			0.907	-
3			0.864	-
4			0.823	-
5			0.784	-
6			0.746	-
7			0.711	-
8			0.677	-
9			0.645	-
10	2nd Repair	$169,298	0.614	103,934
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15			0.481	-
16			0.458	-
17			0.436	-
18			0.416	-
19			0.396	-
20	3rd Repair and HMA Overlay	$329,331	0.377	124,121
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$625,603**		**355,029**
25	Pro-rata residual of last activity:	-$219,554	0.295	(64,835)
	TOTALS:	**$406,048**		**290,194**

Table 27. Present Worth of Cost Analysis for CIP Rapid Repairs with 8-Year Service Life (assumed to be applicable to lab mixtures 2, 7, 8 and 9) – 5% Discount Rate.

8-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2			0.907	-
3			0.864	-
4			0.823	-
5			0.784	-
6			0.746	-
7			0.711	-
8	2nd Repair	$169,298	0.677	114,587
9			0.645	-
10			0.614	-
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15			0.481	-
16	3rd Repair and HMA Overlay	$329,331	0.458	150,870
17			0.436	-
18			0.416	-
19			0.396	-
20			0.377	-
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$625,603**		**392,431**
25	Pro-rata residual of last activity:	-$131,732	0.295	(38,901)
	TOTALS:	**$493,870**		**353,530**

Table 28. Present Worth of Cost Analysis for CIP Rapid Repairs with 6-Year Service Life (assumed to be applicable to lab mixture 11) – 5% Discount Rate.

6-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2			0.907	-
3			0.864	-
4			0.823	-
5			0.784	-
6	2nd Repair	$169,298	0.746	126,333
7			0.711	-
8			0.677	-
9			0.645	-
10			0.614	-
11			0.585	-
12	3rd Repair and HMA Overlay	$329,331	0.557	183,384
13			0.530	-
14			0.505	-
15			0.481	-
16			0.458	-
17			0.436	-
18			0.416	-
19			0.396	-
20			0.377	-
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$625,603**		**436,690**
25	Pro-rata residual of last activity:	-$43,911	0.295	(12,967)
	TOTALS:	**$581,692**		**423,723**

Table 29. Present Worth of Cost Analysis for CIP Rapid Repairs with 4-Year Service Life (assumed to be applicable to lab mixture 12) – 5% Discount Rate.

4-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2			0.907	-
3			0.864	-
4	2nd Repair	$169,298	0.823	139,282
5			0.784	-
6			0.746	-
7			0.711	-
8	3rd Repair and HMA Overlay	$329,331	0.677	222,904
9			0.645	-
10			0.614	-
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15			0.481	-
16			0.458	-
17			0.436	-
18			0.416	-
19			0.396	-
20			0.377	-
21			0.359	-
22			0.342	-
23	Mill, 4th Repair and HMA Overl	$402,280	0.326	130,971
24			0.310	-
25			0.295	-
	Subtotals:	**$1,027,882**		**620,130**
25	Pro-rata residual of last activity:	-$348,642	0.295	(102,955)
	TOTALS:	**$679,240**		**517,175**

Table 30. Present Worth of Cost Analysis for CIP Rapid Repairs with 2-Year Service Life (assumed to be applicable to lab mixtures 1 and 5) – 5% Discount Rate.

2-year Cast-in-Place Repairs				
Year	Item	Assumed Cost	Present Worth Factor	Present Worth Cost
0	Initial Construction	$126,973	1.000	126,973
1			0.952	-
2	2nd Repair	$169,298	0.907	153,558
3			0.864	-
4	3rd Repair and HMA Overlay	$329,331	0.823	270,942
5			0.784	-
6			0.746	-
7			0.711	-
8			0.677	-
9			0.645	-
10			0.614	-
11			0.585	-
12			0.557	-
13			0.530	-
14			0.505	-
15			0.481	-
16			0.458	-
17			0.436	-
18			0.416	-
19	Mill, 4th Repair and HMA Overl	$402,280	0.396	159,196
20			0.377	-
21			0.359	-
22			0.342	-
23			0.326	-
24			0.310	-
25			0.295	-
	Subtotals:	**$1,027,882**		**710,669**
25	Pro-rata residual of last activity:	-$241,368	0.295	(71,277)
	TOTALS:	**$786,514**		**639,392**