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Evaluation of Bonding Agent Application on Concrete Patch Performance

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</table>

Table of Contents

List of Figures

List of Tables

List of Abbreviations

Brookhaven National Laboratory (BNL)
Illinois Department of Transportation (IDOT)
Mid-America Transportation Center (MATC)
Nebraska Transportation Center (NTC)
Polyvinyl Acetate (PVA)
Water-Cement Ratio (w/c)

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Disclaimer

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Abstract

The durability of partial depth repair is directly related to the bond strength between the repair material and existing concrete. Bond strength development sensitivity to wait time with the use of bonding agents in partial depth repair was investigated in this study. Cementitous grouts, epoxy, acrylic latex, and polyvinyl acetate were used as bonding agents for portland cement concrete repair material. Portland cement concrete repairs with dry and saturated surface dry conditions, and three rapid repair cementitious materials were used for comparative purposes to investigate the benefits over other alternatives for using bonding agents. Laboratory samples were made by placing repair concrete 0, 2, 5, 10, and 30 minutes after bonding agent application. The bond strength was then measured using a direct shear test. Field tests were performed using the repair materials and bonding agents. When the agents were applied in the field, the wait times between bonding agent application and repair material application were 0, 15, 30, and 45 minutes. Seven-day and 5-month direct tension pull-off tensile tests were performed during the field experiment. The data from both experiments show that when using cement grout bonding agents, after 15 minutes, bond loss can be expected. Wait times did not have a significant effect on epoxy and acrylic latex bonding agents as long as they were placed before setting. The polyvinyl acetate agent and repair materials can develop high bond strength in laboratory settings, but when used in the field, the bond strengths experience strength loss with time. The results also showed that adequate bond strength for many repairs can be obtained by placing the repair concrete on a substrate in saturated surface dry condition.

Chapter 1 Introduction

1.1 Background

Daily use and weathering of pavements produce deterioration. Aging and deteriorating pavements require improved methods of repair to prevent repair failures that occur all too often. Recently, the topic of partial depth pavement repair has undergone extensive investigation because pavement restoration is often more cost-effective than demolishing inadequate pavement and constructing new pavements, or is needed as a stop-gap measure until pavement reconstruction.

The success of a partial depth repair depends on bond strength development between the repair material and the substrate concrete (J. R. Parker 1985). Factors such as increasing compressive strength of the repair material in a repair (E. B. Julio 2006), applying bonding agents, increasing substrate surface roughness (Courard 2013; E. B. Julio 2004), and using rapid repair materials (Al-Ostaz 2010) to increase bond strength have been studied previously and effects on bond strength improvement have been noted. The addition of bonding agents and having clean and roughened substrate surface (E. B. Julio 2004) prior to repair material placement have shown to improve bond strength, but the condition of the bonding agent prior to repair material being placed hasn't been studied.

1.2 Research Objectives

The purpose of the study was to examine how wait time from bonding agent application until repair material placement affects bond strength development between the existing concrete and fresh repair material. The wait time effects on regular portland cement grouts, epoxy, and latex bonding agents were examined. Control samples were constructed and tested having both a dry surface and a saturated surface dry (SSD) moisture condition prior to repair material placement to determine the benefits, if any, of using bonding agents. Three commonly used rapid repair materials were also tested for comparative purposes.

1.3 Research Overview

The study was divided into two separate phases. The first phase consisted of composite concrete samples that were constructed, bonded, and shear tested in a laboratory setting. A set of samples was put through freeze-thaw cycles to accelerate the weathering on the bond interface and to observe the effects on bond strength.

For the second phase, the bonding agents and rapid repair materials were tested in the field environment. The bond agents and rapid repair materials were placed on field slabs, and tensile tests were performed at two separate ages. The first test was at early age to examine the early strength. The second test was performed after one winter season had passed to observe the loss in strength due to external weathering effects.

1.4 Report Layout

The report is divided into seven chapters, which are described as follows: chapter 2 is the literature review, chapter 3 describes the materials used in the study, chapter 4 the methods used, chapter 5 shows the results, chapter 6 is discussion of the results, and chapter 7 is the conclusions and recommendations.

Chapter 2 Literature Review

Partial-depth concrete patching is commonly used to repair concrete pavements. Effective partial-depth patch repairs can greatly extend the life of concrete pavements. Premature failure of newly repaired concrete is an all-too common problem faced by owners. The mechanisms and factors that contribute to partial-depth concrete failure success and failure deserve further discussion.

2.1 Pavement Repair

2.1.1 Pavement Damage

Pavement damage can be caused by disintegration, moisture, environmental effects, service loading, and construction related effects (Emmons 1993; ACI International 2003). Plastic shrinkage, plastic settlement, and early thermal contraction (ACI International 2003) cracks can occur during construction of the pavement. Plastic shrinkage occurs when settlement in the plastic concrete forces the aggregate to settle allowing the water to migrate to the surface. The surface water can evaporate. When the surface water evaporates faster than the rate of bleed water rising to the surface, plastic shrinkage cracks can form (ACI International 2003). Plastic settlement cracking occurs when tensile forces are produced on the surface of the pavement during the aggregate settlement while the concrete is still plastic (ACI International 2003). Thermal contraction cracks occur in thick pavements because of the heat produced during the cement hydration process. Eventually the concrete will cool, causing the pavement to contract. Restraint provided by friction with the subbase prevents the pavement from fully contracting during cooling. Tensile forces are then generated, which cause surface cracks to form (ACI International 2003).

Disintegration is often a result of alkali-silica reaction, sulfate attack, deicer-salt scaling, and freezing and thawing (Emmons 1993). Disintegration often occurs where free moisture is available. Disintegration can cause the pavement surface to scale and delaminate, and portions of the concrete to crumble. Alkali-silica reaction occurs when alkalis in the pore solution react with reactive silica in some aggregates, and forms an alkali-silicate gel (ACI International 2003). The gel causes expansion when it absorbs water. The expansion causes tensile forces, which produce cracking in the surface. Sulfate attack occurs when concrete is exposed externally to sulfates. Sulfate attack can cause expansive formation of ettringite, causing cracking and crumbling of the concrete (ACI International 2003). Freeze-thaw damage occurs when water trapped in the pores of the concrete expands when temperatures drop below freezing (ACI International 2003). Deterioration is most often seen first at the joints because of higher availability and penetration rates of water at the joints (Emmons 1993).

Once cracking occurs, introduction of foreign containments into the pavement can accelerate the rate at which cracks propagate. Incompressibles become lodged in the cracks. When the pavement experiences expansion or contraction, the incompressibles cause stress in the pavement (T.P. Wilson 2000). Traffic loads can accelerate the rate of deterioration if cracks are present. When pavement deterioration is left unintended cracks are allowed to propagate and the condition of the concrete worsens. Figure 2.1 shows a pavement where the cracks have been allowed to propagate and the quality of the pavement has deteriorated. Figure 2.2 illustrates minor cracks that have started on the pavement.

Figure 2.1 Pavement with surface cracks

Figure 2.2 Pavement with surface cracks

2.1.2 Repair Types

Pavement repairs can be categorized into two types: partial-depth repairs and full-depth repairs (Felt 1960). Partial-depth repairs require the removal of damaged concrete on pavement only near the surface and replacement with repair material. Once the repair material has been placed, monolithic composite action is required for the pavement to be successful (ACI International 2003). Full-depth repair requires removal of the full-depth pavement section and replacement of the damaged concrete. When repairing pavements with reinforcement, such as steel or dowels, the reinforcement will need to be either replaced or cleaned before the repair concrete is applied. If the steel is replaced, the new steel is attached to the existing steel on the pavement (ACI International 2003). Figure 2.3 shows cross sections of (a) full-depth concrete repair and (b) a partial depth concrete repair.

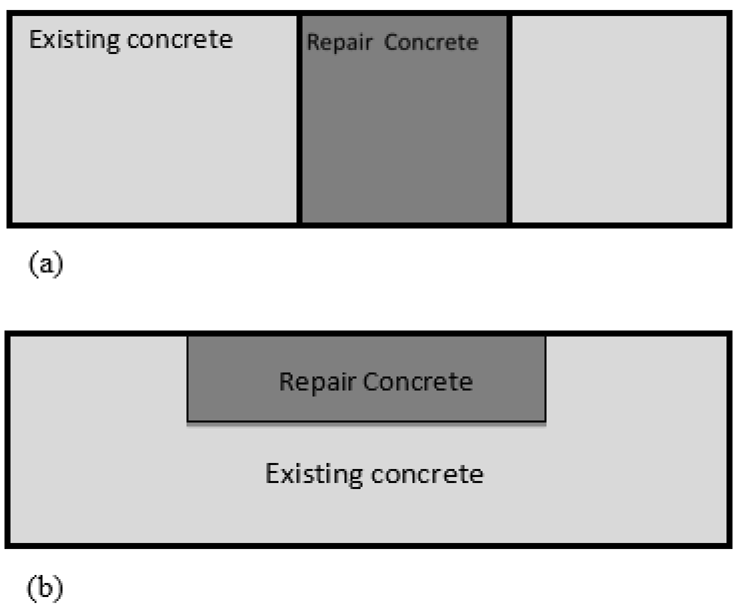

Figure 0.1 Pavement repair full depth (a) partial depth (b)

2.2 Partial Depth Concrete Repair Process

2.2.1 Evaluation

Visual evaluation is a straightforward method to evaluate if a pavement requires repair. When pavements exhibit severe visible distress such as cracking, spalling, disintegration, honeycombing, and scaling (Emmons 1993), proper repair will stop the damage from expanding. Partial depth repairs can be used where there are spalls and wide cracks present (Dar-Hao Chen 2011). Partial depth concrete repairs should not be used in areas that experience durability cracking, high shear stresses, or in areas where the depth of partial depth repair is deeper than the top third of the slab thickness (T.P. Wilson 2000).

Pavement cores can be obtained for evaluation and testing using a concrete coring drill and carbide-tipped drill bits (T.P. Wilson 2000). Field cores can vary in length and diameter and can be tested for durability and compressive strength in order to assess the pavement. After evaluation of the pavement is complete, specific repair methods can be selected. If the full depth of the pavement does not need to be replaced, a partial-depth repair can be performed, which can be much more cost-effective.

2.2.2 Boundary Conditions

When the damaged pavement is identified, all of the damaged areas need to be removed during a repair. This often involves removing concrete some distance beyond the identified damaged areas in order to ensure that damaged concrete that was not visible was not missed. Simple boundary conditions should be established for pavement repairs. Square or rectangular boundaries should be used, because uncommon irregular shapes will expose the repair material to edges that can produce stresses and can lead to premature material failure (T.P. Wilson 2000; Dar-Hao Chen 2011). The repair should be cut to provide the minimum perimeter. Minimizing the perimeter can lower the overall repair cost, even if more repair material is needed because it lowers the amount of saw cutting required, and can help the bond last longer by reducing stress concentrations and cracking. Good performance on field patch repairs can be obtained, but only when all of the damage has been removed by removing slightly more concrete than is known to be damaged (Dar-Hao Chen 2011). This helps ensure that any difficult to detect micro-cracking at the edge of the damaged concrete is removed. The minimum depth of a partial depth patch should be more than two inches (KDOT 2007) in depth but no more than 1/3 slab thickness (T.P. Wilson 2000). This ensures that the patch is thick enough to have the strength to resist basic load induced cracks. If the patch is too thick, the old concrete may be damaged during removal or load transfer devices such as dowels may be damaged during removal. The outside boundaries should be a minimum of 2 inches from the spalled concrete and a maximum of 6 inches (T.P. Wilson 2000). An example boundary layout for a damaged area is illustrated in figure 2.4. Boundaries with four edges are ideal since boundaries with more edges will require additional cuts to be made (Emmons 1993; Dar-Hao Chen 2011; Fowler D 2008).

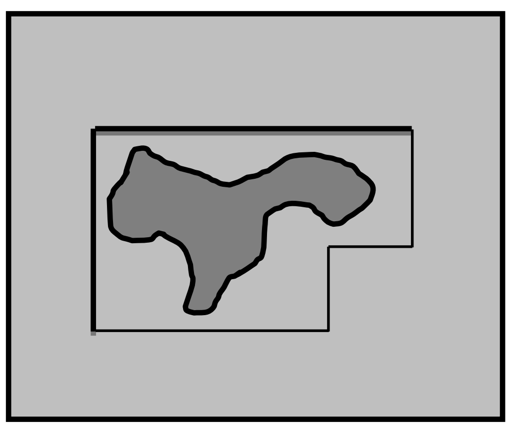

Figure 2.4 Example of simple boundary for pavement repair

2.2.3 Cutting and Removing Concrete

Concrete cutting and removal is typically performed by first saw cutting the perimeter, followed by removing the concrete inside the saw cut boundary. A concrete walk-behind saw with a carbide blade is able to make a 90-degree angle on repair boundaries, thus allowing uniform repair material placement and avoidance of feathered edges (Emmons 1993). Feathered edges develop when boundary edges are sloped, giving edges that are too thin to resist cracking. Transportation agencies have implemented minimum edge slopes to improve patch performance, such as the Kansas Department of Transportation, which limits the edge of a repair to be from 60 to 90-degrees (KDOT 2007).

Concrete removal for partial depth repairs is typically performed using a chipping hammer, milling machine only, or hydro removal (T.P. Wilson 2000). Chipping hammers are commonly used for concrete removal because they are compact and require only one operator. Only 15-or 30-pound hammers should be used for pavement repairs because higher capacity hammers will increase pavement damage in the concrete that remains. Micro-cracking that can be induced by overzealous removal practices is called bruising (Emmons 1993; ACI International 2003).

A field study of partial depth repairs was performed using polyurethane and epoxy based repair materials. For both materials chip-and-patch and saw-and-patch procedures were used. The repairs were opened to traffic and the repair performance was evaluated by the amount of time until the repair showed signs of visible distress. The chip-and-patch and saw-and-patch methods didn't show signs of visible distress until 6 and 9 years after the repair (Dar-Hao Chen 2011). The authors credit the successful patch because all of the delaminated concrete was removed during the patching (Dar-Hao Chen 2011). The study indicates that sawing and removing with a chipping hammer can improve patch performance more than just by concrete removal using only a chipping hammer by eliminating feathered edges and helping reduce bruising at the edges.

2.2.4 Cleaning Substrate Surface

Debris must be fully removed from the surface boundary of the section being repaired before pavement repair material is placed on the repair boundary. Cleaning the existing concrete of loose material allows the new repair material to interlock at the bond interface of the concrete and develop bond strength (Felt 1960; Luc Courarda 2014). Debris can be removed by compressed air and other mechanical methods (Felt 1960; Santos, M.D and Dias-da-Costa 2012). However, when using compressed air, no oil residue should be present in the compressed air that could deposit on the concrete surface. Dust particles or oily substances on the surface will not allow a bond to form between the existing concrete and new repair material.

2.2.5 Bonding Agent Application

Bonding agents can improve bond strength between repair concrete and existing concrete. When a bonding agent is selected for a repair, it is typically applied with a brush or evenly sprayed on the repair surface before the repair material is placed on the repair surface.

2.2.6 Repair Material Placement

Repair serviceability demands dictate the required repair material, and the placement process varies on the material used depending on material chosen. For example, portland cement concrete can be applied without bonding agents, but portland cement concrete requires the use of vibration after placement in order for the concrete to fill the repair boundaries. A laboratory test was performed where repair portland cement concrete was used with and without a cement grout bonding agent made with one part water, one part cement with and without vibration (Felt 1960).

The samples made without a bonding agent had bond strength of 200 psi, whereas the sample made with a bonding agent had bond strength of 300 psi (Felt 1960) with no vibration used when placing samples. When the samples were vibrated, the bond strengths were 210 psi without a bonding agent and 360 psi with the bonding agent used (Felt 1960). Rapid setting repair materials reach maturity at rates faster than ordinary portland cement with no accelerators and rapid repair materials are able to develop strong bonds without the use of bonding agents (Al-Ostaz 2010). Troweling still must be used to level the repair material onto the existing concrete whether it is a portland cement concrete or rapid repair material. Rapid repair materials such as magnesium phosphate and calcium sulfoaluminate can be self-leveling because of the self-consolidating properties (Fei Qiao 2010; J. Pe´ra 2004).

2.2.7 Curing

Multiple methods are used to cure repair materials. The methods fall under two categories: water curing, and sealant curing (T.P. Wilson 2000). Curing compounds and plastic sheeting coverings are sealant curing and work to prevent water already present as mix water from evaporating. Methods such as wetting the surface or applying wet burlap after initial placement is water curing and aims to add additional water to the surface and reduce water evaporation from the surface. Properly curing the freshly placed repair material reduces drying shrinkage-based volume change (Felt 1960) in the repair materials, which can apply stresses at bond interface. These stresses can lead to de-bonding of the repair material from the existing concrete (Santos, M.D and Dias-da-Costa 2012).

When repair material is cured, a joint sealant is applied between joints of the new repair material and the existing concrete. The sealant prevents water and foreign incompressible material from entering the joint.

2.3 Concrete Surface Preparation

Increasing repair concrete strength and durability has been studied as a factor to increase pavement repair performance (E. B. Julio 2006; Langlois 1994). High strength in the repair material, however, does not necessarily translate into a high performance repair (E. B. Julio 2006). Adding fibers to the repair material increases durability and tensile properties, but, as noted, “The durability of thin concrete repairs is generally related to the durability of the bond between the old and the new concrete, not the durability of the new concrete” (Langlois 1994). The condition of the surface of the existing concrete will influence the bond strength development between the repair material and existing concrete by providing mechanical interlock with the new surface and providing open pores for cementitious material to enter.

2.3.1 Moisture Content

Having proper moisture content on the substrate concrete prior to placing the repair material could affect bond strength. SSD conditions on the existing concrete prevent the absorption of extra moisture by the existing concrete from the repair material. Pooling water on the surface before a repair material is placed, however, would decrease bond (Felt 1960). Excess pooling water on the surface of the substrate material can increase the effective concrete water-cement ratio (w/c) at the interface, lowering the bond strength (Santos, M.D and Dias-da-Costa 2012). In a laboratory study where fresh concrete was placed on existing concrete with a dry surface condition and a saturated with pooling water condition, the bond strength dropped from 530 psi to 250 psi (Santos, M.D and Dias-da-Costa 2012). In another study, saturated existing concrete was compared to dry surface with the use of bonding agents. Dry surfaces of existing concrete had a direct shear bond strength of 400 psi, while over-saturated bases had an average of 310 psi. SSD conditions with no pooling water have demonstrated improved bond strength between existing concrete and portland cement repair concrete (Santos, M.D and Dias-da-Costa 2012).

2.3.2 Substrate Surface Roughness

For optimum bond interface, surface preparation by abrasive blasting produces the best bond development between repair material and existing concrete (E. B. Julio 2004) (Courard 2013). Concrete surface profiles can be measured by the International Concrete Repair Institute roughness scale. Smooth surfaces provide weak bond strength development because the repair material cannot readily infiltrate the surface of the substrate concrete and rougher surfaces produce more mechanical interlock (E. B. Julio 2004). Surface roughening techniques that use large amounts of energy, such as that provided by large chipping hammers, can create micro-cracks in the concrete that is not removed. Micro-cracks (Courard 2013) are tiny cracks formed by high impacts. For optimum bond strength, the top surface layer of concrete of the existing concrete should be removed and the aggregate exposed before the repair material is placed (E. B. Julio 2004).

The concrete removal method has been shown to provide a different level of bond. The surface profiles were polished, shot blasted, and water blasted (Courard 2013) before the repair material was placed. It was found that the samples with polished surfaces had a pull off tensile strength averaging 200 psi. The samples with the shot blasted surface had a bond strength of 300 psi. The samples that were prepared with a chipping hammer had a strength of 175 psi. The highest bond strength was from the water blasted samples with a strength of 350 psi (Courard 2013). Adequate bond strength was obtained when the existing concrete surface was roughened, but when high impact forces were used, the bond strength was lowered due to micro-cracking in the substrate concrete.

2.3.3 Steel Anchors

Additional concrete anchors in the repair surface provide further surface area for repair material to bond with the existing concrete. Steel reinforcement can add additional shear strength if bond development occurs. Steel U-bars, varying in diameter and surface profile, can be drilled into the existing concrete, thus adding shear strength between the repair material and existing concrete. When using U-bars in a repair, the U-bar height is limited by the repair depth, which limits the use of U-bars in shallow repairs. Using No. 2, 4, and 6 U-bars increases shear and tensile strength between the existing concrete and repair material, but concrete nails exhibit no significant strength increase because concrete nails have less surface area (Parker, et al. 1985). The addition of steel anchors requires extensive labor, and allows possible steel corrosion, thus damaging the repair and negating repair benefits.

2.4 Bonding Agents

2.4.1 Benefits

Properly selecting and applying a bonding agent between repair materials and existing concrete has been shown to improve bond strength between repair materials and new concrete (Langlois 1994; Winkelman 2002; Santos, M.D and Dias-da-Costa 2012). Selected bonding agents depend on the required performance of the repair. When the repair concrete is portland cement-based grouts, epoxy-based bonding agents and latex bonding agents can be used. Rapid setting repair materials such as magnesium phosphates do not require bonding agents, and if bonding agents are used, the bond strength is typically lowered.

2.4.2 Portland Cement Grouts

Portland cement grouts use cement and water to produce bonding agents that can be used between existing concrete and repair concrete. Grouts with a 0.3 w/c has been demonstrated to increase bond strength (Langlois 1994). A field investigation was completed on existing concrete pavement where a dry substrate, 0.3 w/c grout, wet substrate, and a water/silica fume slurry were used. After the repair material was placed pull off, tensile tests were performed after 7 days and 10 months of ageing and weather exposure. The pull-off tensile strengths were 200 psi for the portland cement grout, 145 psi for the water/ silica fume slurry, and 130 psi for the wet and dry surface conditions (Langlois 1994)

2.4.3 Epoxy Bonding Agent

Epoxy bonding agents must be high modulus, moisture tolerant, and compliant with ASTM C881 (ASTM C882 2013) requirements. Structural epoxies are typically made up of a two-part system of chemicals that are mixed before application. The hardener and the modifier must be thoroughly mixed before the bonding agent is applied between the repair material and the existing concrete. Epoxies must have a minimum gel time of 30 minutes (ASTM C882 2013). Like many chemical reactions, the epoxy hardening process is a temperature-dependent process. Hot weather conditions decrease epoxy gel time and cold weather increases gel time and must be accounted for in the field (Mailvaganam 1997).

In a laboratory study where epoxy bonding agents were used on multiple substrate surface preparations, the samples that used epoxy bonding agents had higher bond strengths (Santos, M.D and Dias-da-Costa 2012) then with samples that did not. The surfaces examined were left as cast, wire brushed, and shot blasted (Santos, M.D and Dias-da-Costa 2012). Both dry and saturated surface conditions were examined. The samples were examined using a direct shear test, and the samples made with epoxy agents after shot blasting the substrate had the highest bond strength of 700 psi. The same sample with no agent had a bond strength of 530 psi.

Even the samples left as cast substrate surfaces, which had a bond strength of 200 psi with no bonding agent, had a strength of 420 psi when using epoxy bonding agents.

2.4.3 Application

Bonding agents are applied to the existing concrete with a brush in a thin continuous layer before the repair material is placed. The entire repair section surface must be covered by the bonding agents (Mailvaganam 1997). When using epoxy, the repair concrete should be applied before the working time is exceeded. Exceeding the gel time will inhibit bond strength development (ASTM C882 2013).

2.5 Repair Materials

Serviceability requirements dictate appropriate repair materials (T.P. Wilson 2000). For repairs that are not time-sensitive, portland cement mortar or concrete can be used. For repairs that are time-sensitive, rapid-setting repair materials may be required. Rapid setting repair materials include magnesium phosphate and calcium sulfoaluminate cement. Rapid repair cements materials can reach high compressive strength within hours of being placed, allowing for fewer delays to traffic in pavement repairs (Fei Qiao 2010; J. Pe´ra 2004).

2.5.1 Polymer Modified Concrete

Polymer modified concrete is created by adding common polymers such as polyvinyl acetates, styrene butadine rubber, and polyvinyl dichlorides to the concrete (M.M. Al-Zahrani 2003). Polymers are added during the batching phase in liquid state in water or added dry mixed with the aggregates. Liquid state polymers can behave as a water reducer, thus improving workability and reducing initial shrinkage. The advantages of polymer modified concrete are as follows: increased abrasion resistance, lower permeability, and increased resistance to freeze thaw exposure (ACI International 2003). The disadvantages of using polymer modified materials are that the permissible temperature range for placement is lower, they can be susceptible to shrinkage cracking, the modulus of elasticity is lower, and polyvinyl acetates should not be exposed to moisture (ACI International 2003). Polymer modified concretes were used in a field study where the materials were applied to existing highways in repair section that were irregular and square in shape. The removal method for the irregular shaped repair sections were by chipping hammer only, while the square shaped areas were prepared by a concrete saw and a chipping hammer. The longevity of the repairs was six years for the irregular shapes and nine years for the square sections (Dar-Hao Chen 2011). Adequate performance was recorded when using polymer concrete in a field study as long as the whole delaminated areas of concrete were removed and replaced (Dar-Hao Chen 2011).

2.5.2 Magnesium Phosphate Cements

Magnesium Phosphate cement (MgP) is produced by mixing dry magnesium and phosphate in a liquid state. The acid-base reaction is shown in equation 2.1 (Fei Qiao 2010):

$$MgO + KH_2PO_4 + 5H_2O => MgKPO_4 + 6H_2O \tag{2.1}$$

The magnesium oxide content of MgP is 85% by mass (Fei Qiao 2010). During the batching process, ammonium gas is produced. MgP also produces more heat during the curing process than portland cement concrete. Temperatures as high as 195°F have been recorded during magnesium phosphate curing (ACI International 2003). The addition of aggregates and retarders to pre-packaged products can lower the heat produced during mixing and increase the setting time (Fei Qiao 2010). In a laboratory study, the observation that the compressive strength of MgP cement after one curing day averaged similar results to the one with the setting time manipulated by the addition of retarders and aggregates (Fei Qiao 2010). When comparing MgP to portland cement, the MgP had 85-180% (Fei Qiao 2010) higher tensile bond than the portland cement. MGP should be applied on dry surface conditions with no water introduced during the repair process. Advantages of MgP are as follows (Li Yue 2013): setting time from 10-20 minutes after initial placement, high early strength with strengths reaching 2000 psi within the first two hours, ability to harden in low temperatures, high bond strength, and high durability. The disadvantages of MgP are that only non-calcareous aggregates can be used and use on a carbonated surface forms carbon dioxide, which weakens the paste and aggregate bond (ACI International 2003).

2.5.3 Calcium Sulfoaluminate Cements

Calcium sulfoaluminate (CSA) cements are made from calcium sulfate, limestone, and bauxite (Winnefeld and Lothenbach 2009). When CSA hydrates in the absence of calcium hydroxide, the reaction proceeds according to equation 2.2. When it proceeds in the presence of calcium hydroxide, the reaction proceeds according to equation 2.3 (J. Pe´ra 2004).

$$C_4A_3S + 2CSH_2 + 36H => C_6AS_3H_{32} + 2AH_3 \tag{2.1}$$

$$C_4A_3S + 8CSH_2 + 6CH + 74H => 3C_6AS_3H_{32} \tag{2.3}$$

Advantages of CSA cements are as follows: high early strength, fast setting, durable, and expansive, which when properly proportioned, can be used to prevent shrinkage, sulfate resistance, and carbonation resistance (Winnefeld and Lothenbach 2009; J. Pe´ra 2004).

2.6 Bond Strength Test Methods

In order to ensure that the repair performs to the specified requirements, tensile, compressive, and shear tests can be conducted. Testing also offers insight into repair effectiveness. There are three methods of testing the bond strength of new concrete to an existing concrete substrate: the slant shear test, the direct shear test, and the direct tension pull-off test.

2.6.1 Slant Shear Test

The slant shear test uses a composite sample of new and old concrete with a bond interface at a 30-degree angle (ASTM C882 2013; A. Momayeza 2005). ASTM C882 describes variants of the slant shear test. The slant shear sample is axially loaded until failure is experienced. Slant shear strength can be calculated by dividing the magnitude of axial load that causes failure by the area of the composite interface surface (A. Momayeza 2005). The slant shear test and composition of the sample are illustrated in figure 2.5. The test is ideal for comparing repair materials, but it is not an ideal representation of field testing conditions. Slant test results are higher than direct tensile and shear tests because axial loading provides a compressive force at the interface that adds friction to the bond interface (A. Momayeza 2005). Failures can be classified into four categories (Al-Ostaz 2010):

1. Strict bond failure with the existing concrete and repair concrete experiencing minor damage
2. Failure at the bond with little damage to the existing concrete
3. Failure at the bond and at least ¼ inch into the existing concrete
4. Complete failure in the existing concrete and the repair material

The slant shear test is used to evaluate bond strength by the resin manufacturing industry (A. Momayeza 2005).

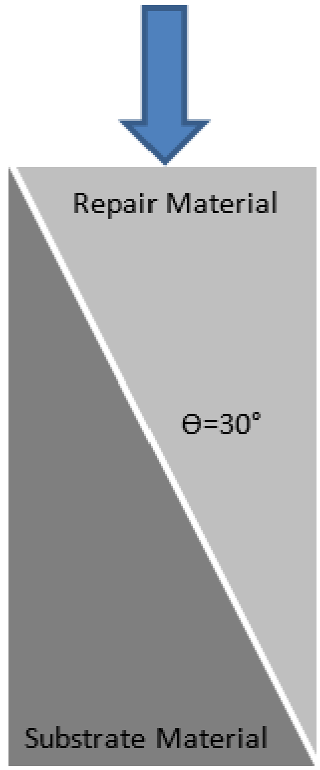

Figure 0.2 Slant shear test

2.6.2 Direct Shear Test

The direct shear test applies shear using a Brookhaven National Laboratory Guillotine Shear Test apparatus (Illinois Department of Transportation 2012). Substrate parent samples must first be made using a 4 in. x 4 in. concrete cylinder. The samples being tested are cast by placing repair material 1.25 in. thick on the pre-made concrete cylinder. Composite samples are loaded at a rate of .22 inches per minute; shear strength is derived by dividing the maximum load recorded to cause failure by the cross-sectional area of the sample. The direct shear test is illustrated in figure 2.6.

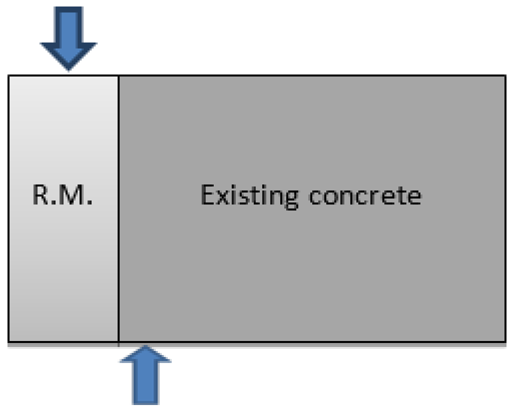

Figure 0.3 Direct shear test

2.6.3 Direct Tensile Pull-Off Test

The direct pull-off tensile test can be performed in the lab or field and is described in ASTM C1583 (ASTM C1583 2013). The test requires 2-inch cores to be drilled into the repair material and to enter a minimum of ½ inch into the substrate concrete (ASTM C1583 2013). When the cores have been drilled, aluminum disks are attached with an epoxy adhesive to the concrete surface. After the adhesive cures, the aluminum disks are pulled off at a constant rate with a tensile loading device. Four failure modes can occur during the test (ASTM C1583 2013):

1. Failure located at substrate concrete
2. Failure located at bond interface
3. Failure located in repair material
4. Failure located between adhesive and disk

Failure one represents a strong bond and higher tensile strength in the repair material and bond interface then in the existing concrete. The second failure is a result of weak bond strength as both the repair material and the existing concrete have higher tensile strengths, and the third failure indicates lower tensile strength in the repair material than in the bond interface and the existing concrete. The final failure is failure in the adhesion between the aluminum disk and the repair sample and is considered an invalid test (ASTM C1583 2013).

2.6.4 Method Comparison

The slant shear test has been shown to give much higher bond strength than the direct shear and direct tensile test (A. Momayeza 2005). In the study, composite concrete samples using consistent mix designs and surface roughness showed that the direct shear test showed higher bond strength than the direct tension pull-off test (A. Momayeza 2005). The lowest bond strength was the pull off tensile test with a recorded bond strength of 125 psi (A. Momayeza 2005). The study shows that the bond strength depends on the type of stress applied to the interface. This suggests that when determining the proper quality control test for the bond interface strength, the type of stresses on the repair should be considered.

2.7 Conclusion Drawn from Literature

Bond strength of repair material to the existing concrete in a partial depth concrete repair is dependent on a number of factors that include surface moisture, roughness, repair material, surface preparation, and bonding agent application. Through proper preparation and application proper bond strength can be obtained during a partial depth repair.

Chapter 3 Materials

3.1 Cements

One ASTM C150 (ASTM C150 2012) Type I portland cement and one ASTM C150 Type III portland cement were used in this study. The chemical composition of the cements is shown in table 3.1.

Table 0.1 Cement composition

Property	Type I	Type III
SiO_2 (%)	21.9	22.0
Fe_2O_3 (%)	3.2	3.4
Al_2O_3 (%)	4.2	4.2
CaO (%)	64	63.5
MgO (%)	2.2	2.0
SO_3 (%)	2.7	3.2
Loss on ignition (%)	1.1	1.5
Insoluble Residue (%)	0.2	0.3
Free Lime (%)	1.2	1.0
Na_2O (%)	0.2	0.2
K_2O (%)	0.5	0.5
Na_2O_{eq} (%)	0.5	0.9
C_3S (%)	53.1	48.8
C_2S (%)	22.8	26.4
C_3A (%)	5.7	5.3
C_4AF (%)	9.8	10.4
Blaine Fineness (m^2/kg)	379	589

Laboratory substrate samples were made using the Type I cement. The portland cement-based bonding agents and repair mortar were made with Type III cement. The field slab samples were constructed using ready-mixed concrete made with a Type I cement. The grouts and repair concrete were made with the Type III cement.

3.2 Rapid Repair Materials

The rapid repair materials used in the laboratory and field tests were a magnesium phosphate (MgP) cement, Pavemend®, and a calcium sulfoaluminate (CSA) cement. All of the materials required the substrate concrete to be clean and free of oil prior to placement after having the substrate surface roughened.

MgP consisted of a part A and B components. Both part A and B are pre-packaged materials that are to be mixed together using 50 lb. of part A and one gallon of the liquid part B. The powdered part A was mixed with the part B liquid component in a five gallon plastic container, and mixed with a portable paddle mixer as specified by the manufacturer.

Pavemend only required two quarts of water to be added and mixed with the 51 lb. of powder provided in a five gallon container. The material was mixed with a portable paddle mixer in a plastic five gallon container. Pavemend placement required vibration or rodding.

The CSA cement used came in prepackaged dry powder material that was mixed with water. The CSA cement required five quarts (10.4 lb.) of water to be added to a 55 lb. bag of the dry powder component. The water was added to the dry mix and mixed with a portable paddle mixer in a five gallon container. After the material was mixed, the material was placed on the substrate concrete.

3.3 Aggregates

The fine aggregate used for the laboratory samples was a siliceous natural sand with a fineness modulus of 3.24, called MCM sand hereafter. The course aggregate used was granite aggregate from Mill Creek Oklahoma and met the requirements for an ASTM C33 (ASTM C33 2013) number 57/67 rock with a nominal maximum size of ¾ inch.

The field slab was constructed using ready-mixed concrete made with the MCM sand, a number 57/67 limestone coarse aggregate from the Bayer Zeandale quarry in Kansas, and will be called limestone. The repair mortars used for the field tests were made using MCM sand and the UD-1 sand with a fineness modulus of 4.23 called hereafter UD1 Sand. The aggregate gradations are shown in figure 3.1.

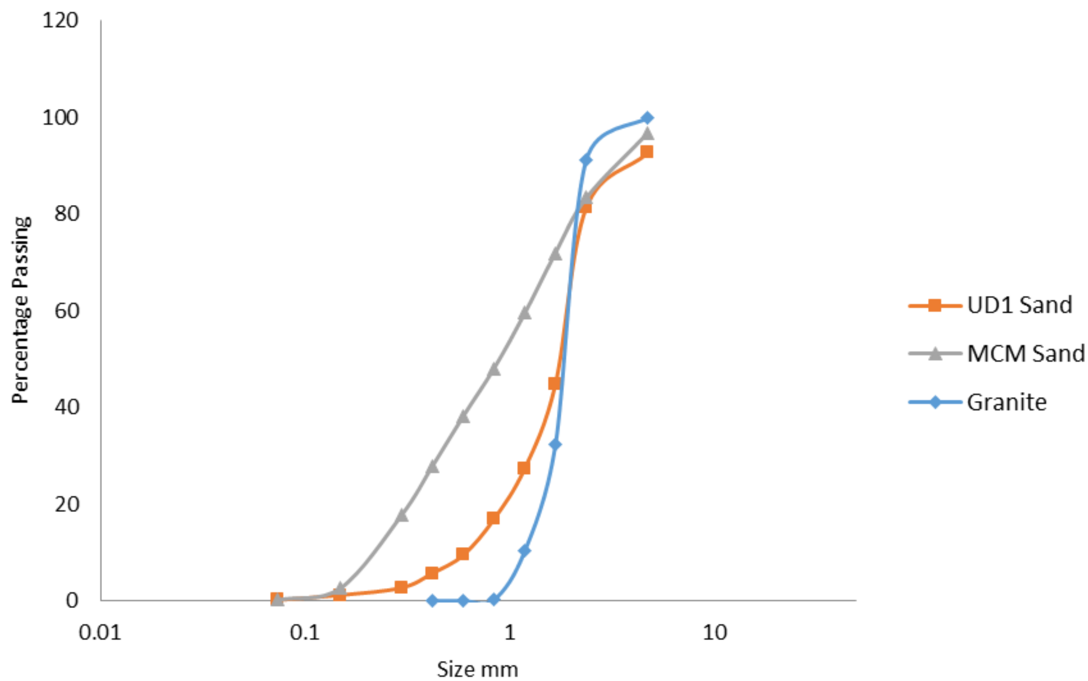

Figure 0.1 Aggregate gradation

3.4 Bonding Agents

Three cement grouts, one epoxy, and two latex bonding agents were tested during the laboratory and field testing.

The latex agents used were a non-reemulsifiable acrylic based and a reemulsifiable polyvinyl acetate (PVA) based bonding agent. Both of the bonding agents met the requirements of ASTM C1059 (ASTM C1059 2013).

The ASTM C881 (ASTM C882 2013) compliant epoxy bonding agent used was prepared by mixing equal parts by volume of part A and B solutions. The epoxy is mixed in a container with a paddle mixer for three minutes prior to application. The epoxy agent used was a high modulus, medium viscosity, and moisture tolerant agent. The epoxy requires a minimum temperature of 40°F during application, and for the concrete substrate surface to be sand blasted, free of foreign contaminant, and be mixed in a well-ventilated room

Type III portland cement grout with 3-1, 0.5, and 0.3 w/c were used in the laboratory testing. For the field portland-cement based bonding agents, Type III portland cement grouts with a w/c of 3-1, 1-1, and 0.5 were used. The same latex and epoxy agents used in the laboratory testing were used for the field testing.

3.5 Concrete Admixtures

Air entraining admixture was used for the laboratory substrate samples to meet the required air content. The field slabs had both air entraining and water reducing admixtures.

3.6 Laboratory Substrate Concrete Mixture

The substrate concrete design used for all of the samples constructed in the laboratory is provided in table 3.2. The ASTM C150 Type I cement was used in this concrete mixture.

Table 0.2 Substrate concrete mix design

Cement	Water	MCM Sand	Granite	Air Entraining Agent
602 lb./yd^3	235 lb./yd^3	1552 lb./yd^3	1552 lb./yd^3	1.12 oz./ 100 lb. cement

3.7 Laboratory Repair Mortar Mixture

The laboratory grout bonding agents were prepared by placing the proportioned cementitious materials in a 5L Hobart mortar mixer and mixed following ASTM C305 for mixing cementitious pastes. The mortar used was produced with Type III cement and had a w/c of 0.4. A sand-cement ratio of 2.75 was used in this study.

3.8 Laboratory Bonding Agents

The cementitious grouts were mixed using a 5L Hobart mortar mixer and mixed following ASTM C305. For the epoxy bonding agent, 16 oz. of part A and part B were mixed together following manufacturer recommendations in a five gallon plastic container using a paddle mixer and a high torque drill.

For the PVA bonding agent, 16 oz. of PVA bonding agent were diluted with 16 oz. of water in a five gallon plastic container using a paddle mixer and a high torque drill following manufacturer recommendations

For the laboratory testing, the acrylic bonding agent was used with type III cement grout and water to make a bonding agent. The bonding agent was made following manufacturer recommendations by combining 16 oz. of acrylic latex agent, 16 oz. of water, and 2 lb. of cement. The acrylic bonding agent was mixed in a 5L Hobart mortar mixer.

3.9 Field Substrate Concrete

Two concrete field slabs were constructed using ready-mixed concrete. The ready mixed concrete used an ASTM C150 Type I/II portland cement. Both of the slabs were constructed using ready-mix concrete with a maximum aggregate size of ¾”. The concrete design is provided in table 3.3.

Table 0.3 Substrate concrete design

Cement	Water	MCM Sand	Limestone	Air Entraining Agent	Water Reducer
620 lb./yd^3	249 lb./yd^3	1944 lb./yd^3	1035 lb./yd^3	3 oz./yd^3	37.2 oz./yd^3

3.10 Field Repair Mortar

The portland cement mortar used in the field slab repair was produced using Type III cement and a w/c of .38. Two fine aggregates used to create the mortar were the UD-1 and MCM sand. The repair mortar proportions are shown in table 3.4.

Table 0.4 Repair Mortar Mixture Proportions

Cement	Water	MCM Sand	UD1 Sand	Air Entraining Agent
750 lb./yd^3	285 lb./yd^3	1388 lb./yd^3	1287 lb./yd^3	0.9 oz./ 100 lb. cement

3.11 Field Bonding Agents

The cementitious grout bonding agent w/c were 3, 1, and 0.5. The epoxy bonding agent was constructed by mixing 32 oz. of part A and B in a five gallon plastic container with a paddle attached to a low torque drill. The PVA agent was made by diluting 32 oz. of the agent with 32 oz. of water. The agent was mixed in similar fashion. The acrylic bonding agent was not made into a cementitous grout, but was applied directly as a film on the existing concrete.

Chapter 4 Methods

4.1 Laboratory Testing

For the laboratory testing, a modified version of the Illinois Department of Transportation (IDOT) specification "Standard Method of Test for Shear Strength of Bonded Polymer Concrete" was used. The test was modified to use a lower thawing temperature during the freeze-thaw cycles. The samples were heated in an oven at 120 °F instead of 150 °F as specified in the IDOT test method. The test requires the construction of composite cylindrical samples that are composed of substrate concrete and repair material. Three sets of three samples each were constructed, two on concrete substrate and one on steel substrates. The concrete samples were abrasive blasted to acquire roughen the surface to develop a bond between the existing concrete and new repair material. Bonding agents were applied when used, and the repair material was placed. A set of concrete samples and steel substrate samples were put through freezing and thawing cycles. At the end of the thermal cycles all three sets of samples were loaded using a direct shear test.

4.1.1 Substrate Concrete

Four inch by four inch substrate cylindrical concrete samples were constructed using Type I portland cement concrete. Concrete substrate mixtures were made according to ASTM C192 (ASTM C192 2010). Concrete slump and air content were measured following ASTM C143 (ASTM C143 2012) and ASTM C231 (ASTM C231 2012), respectively. For each bonding agent, 30 4 x 4 in. cylinder samples and 6 4 x 8 in. cylinder samples were cast in plastic molds that were sealed for a period of 24 hours and allowed to cure in a room at 73°F. After the initial 24 hours in the plastic molds, the substrate samples were de-molded and moist cured for three days. The 4 x 8 in. cylinders were tested for compressive strength following ASTM C39 to establish the substrate concrete compressive strengths at 3 and 14 days. The samples were then cured for a final period of 14 days in a room with 50% relative humidity and a constant temperature of 73°F to dry the concrete cylinder surface for repair mortar application. For the laboratory testing, steel blanks were also used as a substrate sample. The steel samples were 4 in. x 4 in. cylinders.

4.1.2 Substrate Surface Preparation

The concrete substrate samples were sandblasted with #70-140 glass beads to remove concrete laitance and add surface roughness. The substrate concretes were sand blasted until aggregates were seen. The testing also required for 4 x 4 in. sand blasted steel cylinders with white metal finish with a blast profile between 25-75 Microns to be used. Placement of bonding agents and rapid repair materials could be started once the substrate concretes were prepared. The steel substrate samples were also sandblasted before repair material application.

4.1.3 Applying Bonding Agent and Rapid Repair Materials

Thirty composite samples were constructed with a portland cement substrate concrete and repair mortar. Fifteen samples were cast using the sandblasted steel pucks and the repair mortar. The substrate samples were slipped into plastic molds with sides 1.25 in. above the substrate so the bonding agent and repair concrete could be cast above it. The bonding agents were applied to the substrate concrete using a foam brush as shown in figure 4-1. Figure 4.1 (a) was a steel sample with grout applied, and figure 4-1 (b) was a concrete sample. The bonding agents were applied in a room with 50% relative humidity and a constant temperature of 73°F, and were allowed to sit for 0, 2, 5, 15, and 30 minutes before the repair mortar was cast to investigate the sensitivity of the bonding agents to drying time. Two sets of samples were cast without the use of bonding agents. For these two sample sets, the repair concrete was cast on substrates with either

SSD or dry surface. The repair concrete specimens with no bonding agents were used as a reference control. The three rapid repair materials were placed on the substrate concrete following the manufacturer recommendations without bonding agents.

(a) (b)

Figure 0.1 Substrate samples with applied bonding agent

The same mortar mix design was used for all of the bonding agent tests as well as the samples that did not have bonding agents, except for the rapid repair materials that were tested without bonding agents. The repair material was rodded 20 times with a 1/4 in steel tamping rod following the Illinois Standard Method of Test of Shear Strength of Bonded Polymer Concrete. After rodding, the samples were covered with plastic lids and stored in a 73°F 50% relative humidity room for a period of 24 hours. The samples were then de-molded and freeze-thaw cycles commenced. Figure 4.2 shows an example of the composite sample.

Figure 0.2 Composite concrete sample

4.1.4 Freeze-Thaw Cycles

Freeze-thaw cycles were performed on three concrete samples and three steel substrate samples after repair material hardening for each bonding agent drying time. The Illinois Department of Transportation specification "Standard Method of Test for Shear Strength of Bonded Polymer Concrete" was used as the basis for the freeze-thaw cycling performed on some samples prior to shear tests except that different freezing and thawing temperatures were used.

For each setting time three concrete samples were put through five thermal cycles, and the other three steel samples and concrete samples were kept in a room with 50% relative humidity and a constant temperature of 73°F for 14 days. After three days of curing, the composite samples that were subjected to freeze thaw cycles were subjected to the temperature changes as follows:

1. Samples were placed in an oven with a constant temperature of 120°F ± 2°F for a period of 22 hours

2. Moved to a temperature of 73°F ± 2°F for two hours for thermal stabilization
3. Placed in a freezer with a constant temperature of 0°F ± 2°F for 22 hours
4. Moved to a temperature of 73°F ± 2°F for two hours for thermal stabilization
5. Steps 1 through 5 were repeated for five cycles.

4.1.5 Loading

The Brookhaven National Laboratory (BNL) guillotine shear test apparatus was used to measure the concrete bond shear strength. When the freeze-thaw cycles were completed, both sample groups that were subject to thermal and non-thermal cycles were loaded until failure, as seen in figure 4.3, at a rate of .22 in. per minute with the BNL guillotine. The shear stress was calculated by dividing the maximum load recorded by the surface area of the cylindrical sample.

Figure 0.3 BNL guillotine

4.1.6 Bonding Agents Application

Control Samples

Two separate control samples were investigated. The first group of samples had the repair mortar placed directly on the substrate concrete with no bonding agents. The second group of samples had the repair mortar placed with the surface of the substrate concrete in SSD condition that was made by lightly misting a water spray bottle and allowed to soak in briefly prior to the addition of the repair mortar.

3-1 W/C Grout

The first bonding agent that was subject to the applications testing was the 3-1 water to cement Type III portland cement grout. The grouts were applied with a foam brush to a thickness of 1-2 mm, and allowed to set for 0, 2, 5, 15, and 30 minutes. The effects of the bonding agent grout drying out from evaporation and absorption by the substrate concrete can be seen in figure 4.4. As shown in the figure, the sample with 0 wait time is still very fluid. After 15 minutes the grout began to thicken. By the end of the 30 minutes much of the water had evaporated. The grout on the steel samples did not lose as much water as the samples with the concrete substrate because the steel substrate does not absorb water.

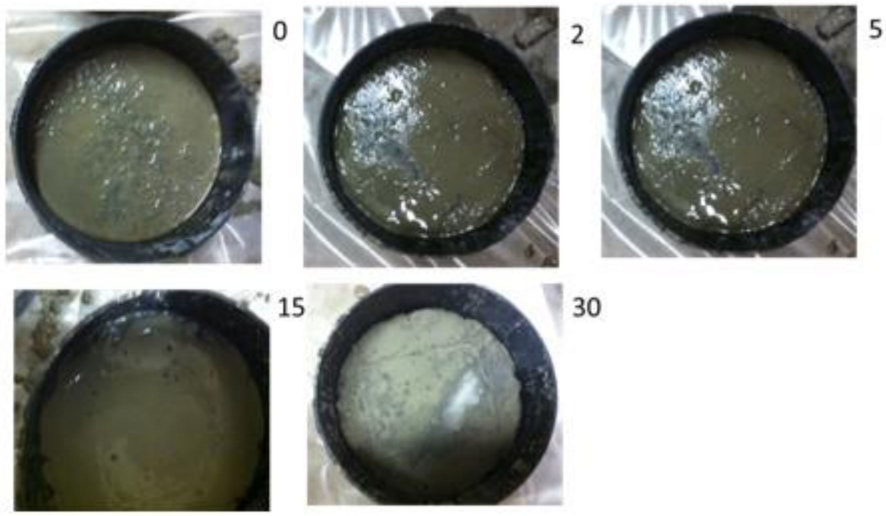

Figure 0.4 Waiting time effects for 3-1 grout

0.5 W/C Grout

The 0.5 bonding agent was much more viscous than the 3-1 grout used. Figure 4.5 illustrates how wait time affected the bonding agent. The 0.5 w/c grout lost its free water much sooner. After it dried, instead of becoming more of a paste-like consistency the 3-1 grout used, it started to resemble dried clay.

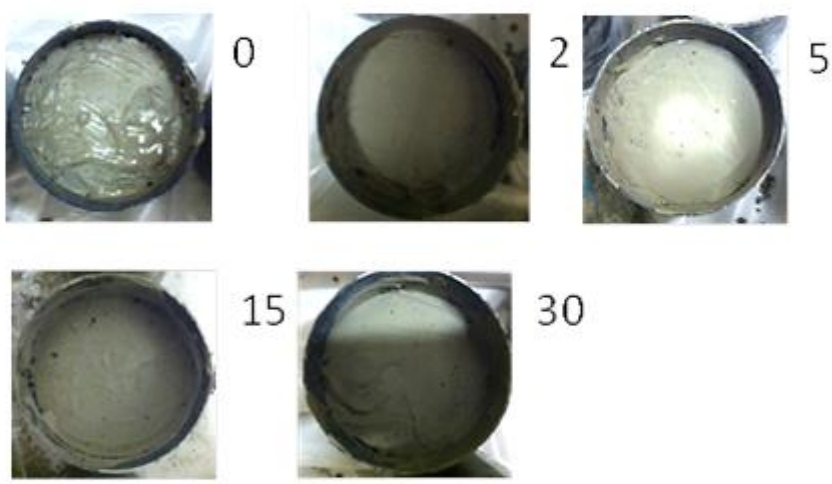

Figure 0.5 Waiting time effects for 0.5 W/C grout

0.3 W/C Grout

The workability of the 0.3 grout was the lowest compared to the other grouts. Because of the low workability, it had to be applied by hand applications instead of with a foam brush. The material appeared to dry significantly after 30 minutes of drying, as shown in figure 4.6.

(a) (b)

Figure 0.6 Effects of wait time on 0.3 grout 0 minutes (a) and 30 minutes (b)

Epoxy and Latex Bonding Agents

The room the epoxy and the latex agents were mixed in was a well-ventilated room at 73°F. The epoxy and latex bonding agents were applied to the substrate samples and allowed to wait for 0, 2, 5, 15, and 30 minutes after bonding agent application until the repair material was placed. These agents were prepared and applied following manufactures recommendations in a well-ventilated 73°F room, with 68% relative humidity.

The acrylic agent requires the existing concrete surface to be in the SSD condition. The acrylic bonding agent can be applied in two ways. One was is to apply it directly on the surface before the repair material was cast. The second way to apply the agent is to dilute it with a 1:1 ratio of water, and add cement to produce a paste. The SSD condition was met by lightly misting water with a spray bottle and then applying a coat of the bonding agent on the existing concrete.

For the laboratory testing the acrylic bonding agent was made into a cementitious grout following manufacturer's recommendations.

The manufacturer recommendations for the reemulsifiable PVA bonding agent called for the agent to be diluted with a 1:1 ratio of water before application. According to the manufacturer, the bonding agents had a setting time of 1-2 hours.

4.2 Field Testing

Two concrete slabs were constructed in the field. One of the slabs was made with one repair strip, and the other with two strips for repair material placement. Forms were placed on the top section of the concrete form to allow for a void strip for a partial depth repair to be made. The repair sections had the boundary edges saw cut and bottom surface roughened prior to the bonding agents and repair materials to be placed on the existing concrete. The epoxy, latex, and grout bonding agents were used with repair materials cast at various setting times to observe bond strength development. The three rapid repair materials were also tested on the field. After the repair material was placed and cured, the bond strength was measured.

4.2.1 Site Preparation

The field testing took place at the Civil Infrastructure Systems Laboratory at Kansas State University. Ten inch thick field slabs were constructed, one with dimensions of 8 x 24 ft. and the other 6 x 24 ft. The slabs were cast alongside already existing slabs. Ground leveling was completed using a skid-steer loader. Once the ground was level, wooden forms were set and stakes were placed so that the concrete forms would hold the pressure of the concrete during the placing process. The finished site before the first concrete slab was placed can be seen in figure 4.7.

Figure 0.7 Site preparation

4.2.2 Field Slabs Fabrication

The first slab was cast on September 24th, 2013. The concrete was supplied by a ready-mix concrete truck. Air-content and slump tests were performed immediately after arrival of the truck to make sure the concrete met required specifications. Compressive strength test cylinders were made to evaluate the compressive strength of the concrete used in the slabs. A concrete vibrator having a 1.5 in. diameter head was used to consolidate the concrete. The vibrating end was inserted and removed from the concrete in a vertical motion. The concrete slab was screeded with a wooden 2 x 6 in. beam that was ten feet in length. When the surface of the concrete slab was level, a 6 in. x 4 in. wooden box that spanned 22 ft. was placed in the center. The wooden box allowed a rectangular section in the middle of the slab to be open that was 6 in. wide and 2 in. deep. The cut out section was left in the concrete slab to make space for the repair and lessen the amount of concrete that would need chipped out later. Once the wooden frame was placed in the slab the surface was finished with a bull float. The finished field slab 1 is shown in figure 4.8. After one day of curing, the wooden box frame was removed from the slab.

Figure 0.8 Field slab 1

Field slab two was constructed using the same process and mix design as the first slab and was placed on October 4th of 2013. The difference between slab 1 and 2 was that slab two had two box frames placed in the slab. Field slab 2 is shown in figure 4.9. After the two boxes were placed on the slab, weights were used to keep the boxes from being uplifted by the buoyant force.

Figure 0.9 Field slab 2

4.2.3 Preparing Field Slab Surfaces

Before placing the bonding agents and repair materials on the repair sections of the field slabs, the surface interface had to be prepared to ensure bond strength development. A saw cut was made one inch from the edge of the formed void in the slab. The concrete between the saw cut and the formed edge was then removed. This left an eight inch wide void two inches deep. Edge removal is shown in figure 4.10. After the edges of the repair section were cut, the surface of the repair area was roughened with the use of a needle scabler and is shown in figure 4.11. The top layer of the concrete surface was removed and aggregate was exposed. The surface had a roughness of 5 on the International Concrete Repair Institute surface roughness scale. The interface surface between the field slab and the repair material was kept clean and free of oil and dust. Figure 4.12 shows the condition if the field slab before bonding agents and repair materials were placed.

Figure 0.10 Saw cutting of edges

Figure 0.11 Prepared concrete surface

Figure 0.12 Concrete void strips before repair application

4.2.4 Placing Bonding Agents

The surface of the repair slab sections were cleaned again before bonding agents and repair materials were placed. Because of the difficulty placing the 0.3 w/c grout in the laboratory tests, a grout with a w/c of 1 was used instead. The w/c for the portland cement grouts used were 3-1, 1-1 and 0.5. The bonding agent setting times before repair material placement were 0, 15, 30, and 45 minutes. The bonding agents were applied on the surface with a foam brush. Pictures were obtained of the setting time effects for the epoxy, PVA, and acrylic bonding agents, and are shown in figures 4.13 through 4.15.

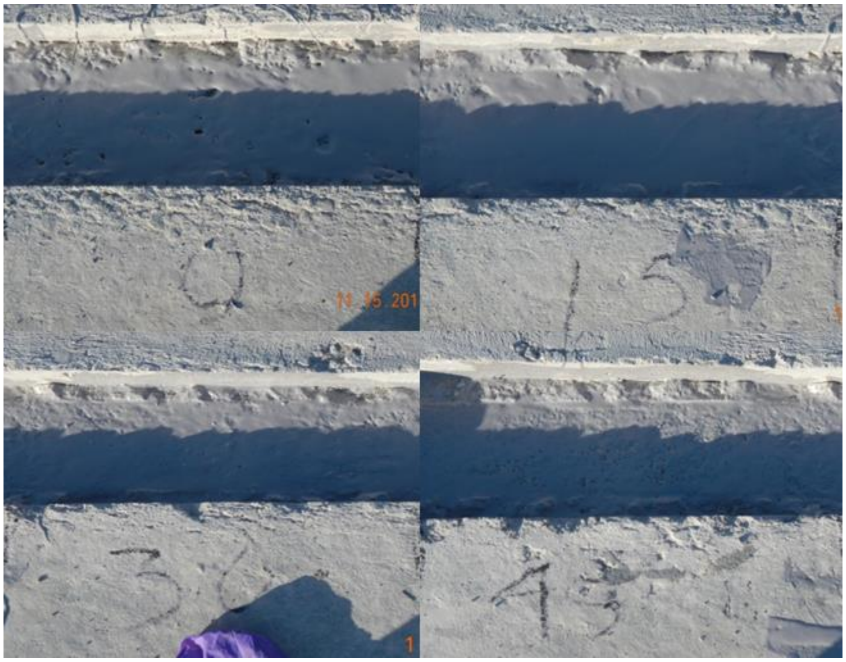

Figure 0.13 0, 15, 30, and 45 minutes after epoxy bonding agent application

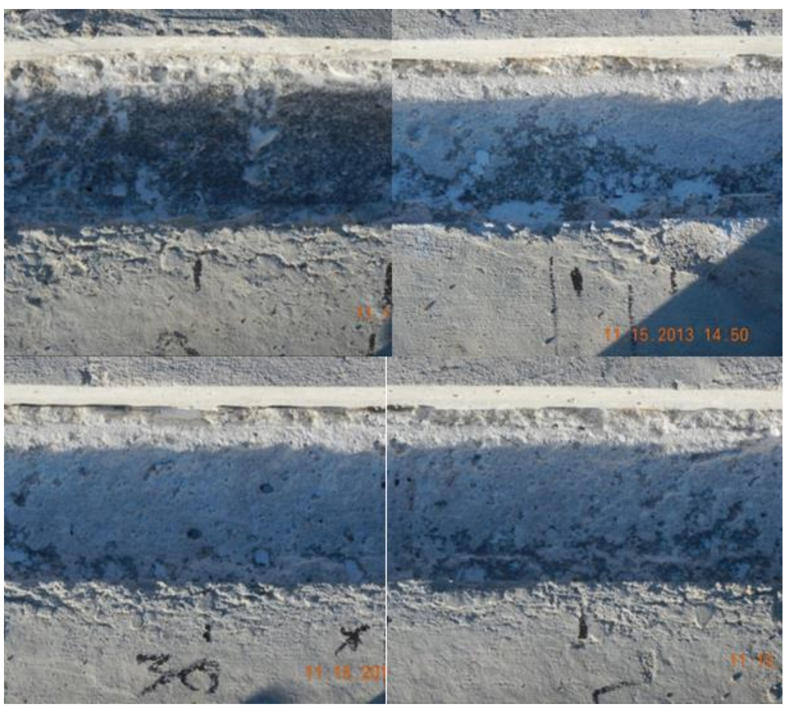

Figure 0.14 0, 15, 30, and 45 minutes after PVA bonding agent application

Figure 0.15 0, 15, 30, and 45 minutes after acrylic bonding agent application

4.2.5 Repair Materials

The rapid repair materials were placed on slab 1 and mixed using a portable electric concrete mixer. Compressive strength cylinders were made for the rapid repair materials and repair concretes used. The CSA and MgP were self-consolidating and were placed into the slab with no vibration used. The Pavemend was not self-consolidating, so after placement the Pavemend was rodded with a 1 inch diameter steel rod. The control sections that contained no bonding agents were placed on slab 1. Magnesium trowels were used to finish the repair materials, and were cured following manufacturer recommendations. The boding agents were used in slab 2. After a predetermined waiting period after bonding agent application, the repair material was placed. The repair concretes were consolidated by using a 1 inch diameter concrete vibrator. The vibrating end was placed into the concrete in a vertical motion and caution was taken to ensure that the vibrator would not touch the surface of the field slabs. The repair concrete was then troweled and finished. The repair materials were cured with the use of plastic sheeting for 24 hours. Figure 4.16 shows the epoxy and latex bonding agent section with repair concrete 1 placed. Thermocouples were placed in the repair materials to measure the concrete temperature evolution.

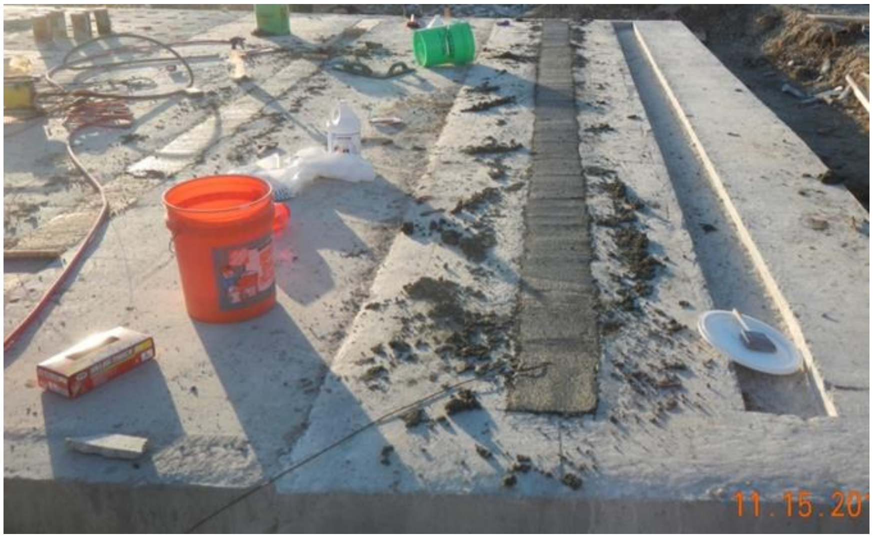

Figure 0.16 Repair material during placement

The repair materials were cured after placement by covering the repair with plastic sheeting to reduce moisture loss due to evaporation. The repair materials were cured for a minimum of 24 hours.

4.2.6 Pull-Off Tests

Pull-off tensile tests were conducted 7 days and 5 months after repair material placement. ASTM C1583 was followed when using the pull off procedure. Two inch diameter cores were first drilled 2.5 inches deep. ASTM C1583 requires that the cores have a minimum depth of 0.5 inches into the substrate material past the bond interface surface. Four cores were drilled for each waiting time and bonding agent used. After coring, aluminum disks were epoxied onto the core top surface. The aluminum disks were sand blasted prior to being attached to the repair material to guarantee that the disk was free of containments. The pull-off tensile loading was displacement controlled with a loading rate of 0.18 in./min. The concrete repair material after the pull-off tests can be seen in figure 4.17. The maximum tensile force during the pull-off test was recorded. If any failures occurred between the epoxy and the aluminum disk the test was considered invalid according to ASTM C1583. The type of failure that occurred during the pull-off test was recorded.

Figure 0.17 Pull-off tensile testing

The four types of failure are illustrated in figures 14-8 (a), (b), (c), and (d). For type 1 failure, the substrate concrete is still attached to the repair concrete by the bond interface layer. Type 2 breaks are located right at the bond interface. Type 3 failure is located in the repair material, and type 4 failure is located at the epoxy interface between the aluminum disk and repair material.

(a) Type 1

(b) Type 2

(c) Type 3

(d) Type 4

Figure 0.18 Types of pull-off test failures

Chapter 5 Results

5.1 Laboratory Data

The compressive strength of the substrate concrete is provided on table 5.1. For each waiting time examined, three samples were tested in shear. Figure 5.1 shows the shear strength of the materials that did not use bonding agents. Figures 5.2 to 5.8 shows the shear strength of individual bonding agents using steel substrates after five cycles of freezing and thawing cycles, and the concrete substrates with and without the five cycles of freezing and thawing cycles. Figures 5.8 to 5.10 show the shear strength of the bonding agents when compared with one another for the different substrate and curing before strength testing. Appendix A contains the laboratory shear strength data and standard deviations in tabular form.

Table 0.1 Substrate concrete data

Repair Mortar	Substrate Concrete	Bonding Agent	Compressive Strength (psi)		Percent Air
			3 Day	14 Day	
M1	B1	3-1 Grout	4200	7400	5.3
M2	B2	3-1 Grout	4500	6700	6.3
M3	B3	0.5 Grout	4300	6700	5.3
M4	B4	0.3 Grout	3800	5400	5.8
M5	B5	Epoxy	4600	5800	5.1
M6	B7	PVA	3100	4800	5.8
M7	B8	Acrylic	4000	5900	5
M8	B9	MgP, CSA Ctrl Dry, Ctrl Ctrl Dry	4100	6800	5.4

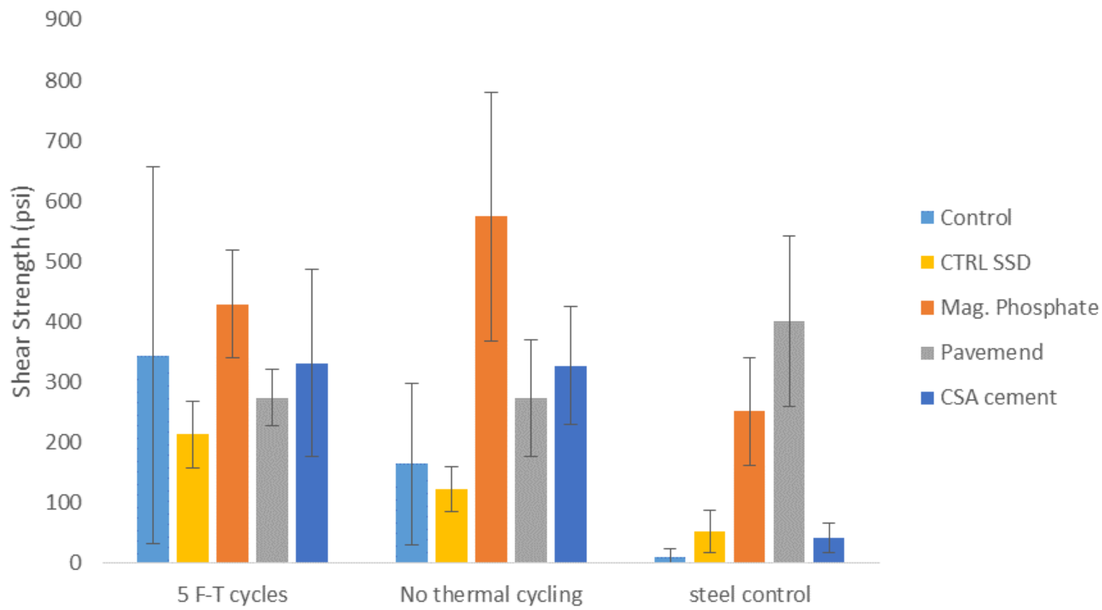

Figure 0.1 CRTL, CTRL SSD, MgP, PM, and CSA shear strength

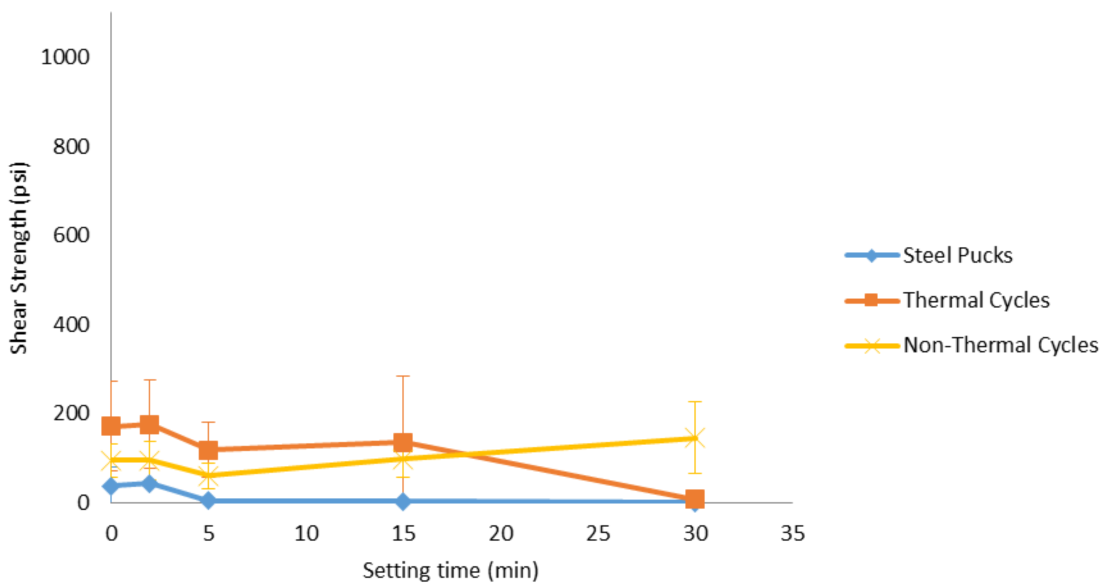

Figure 0.2 3-1 W/C grout shear strength

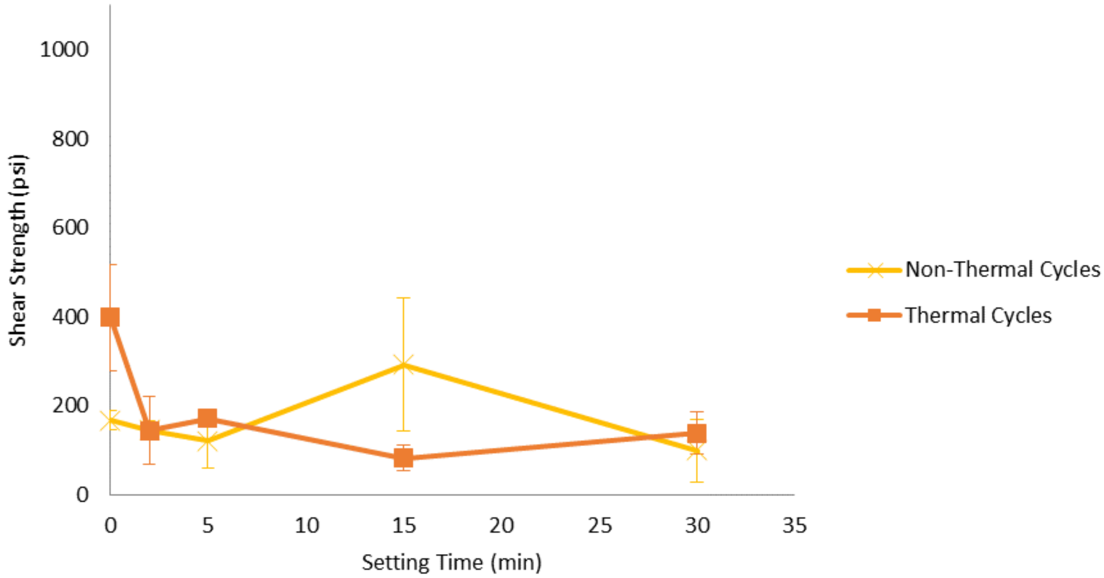

Figure 0.3 0.5 W/C grout shear strength

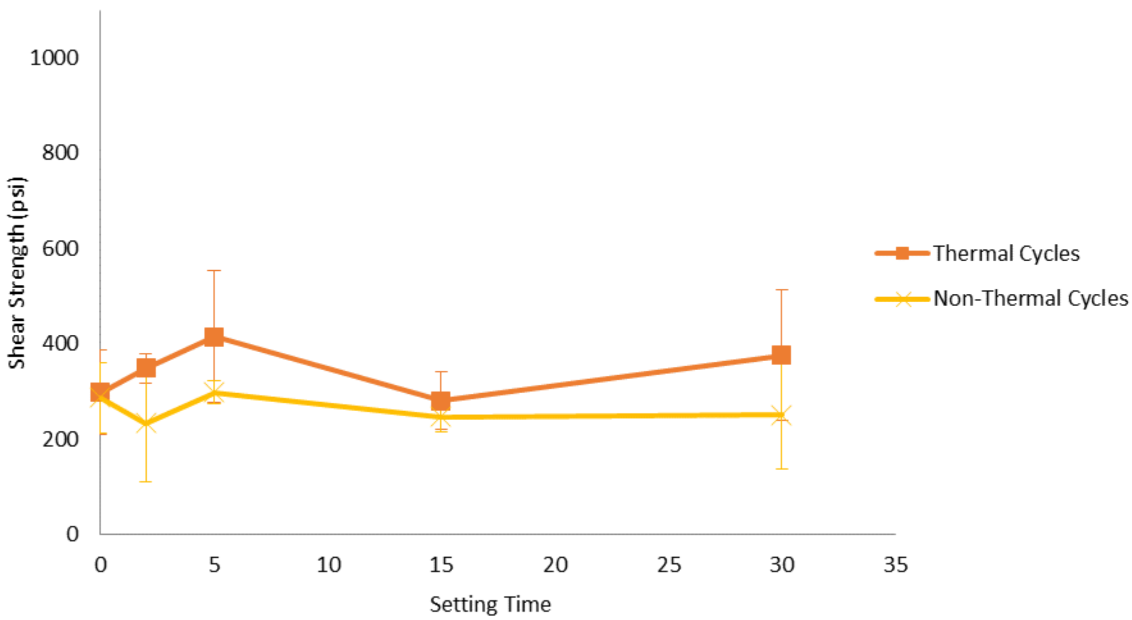

Figure 0.4 0.3 W/C grout shear strength

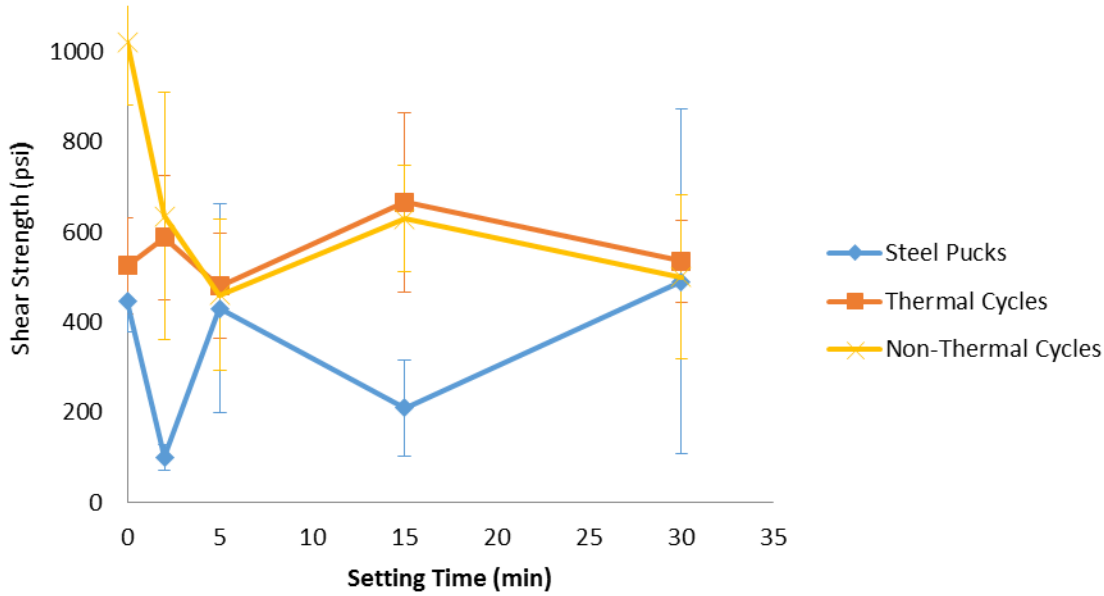

Figure 0.5 Epoxy agent shear strength

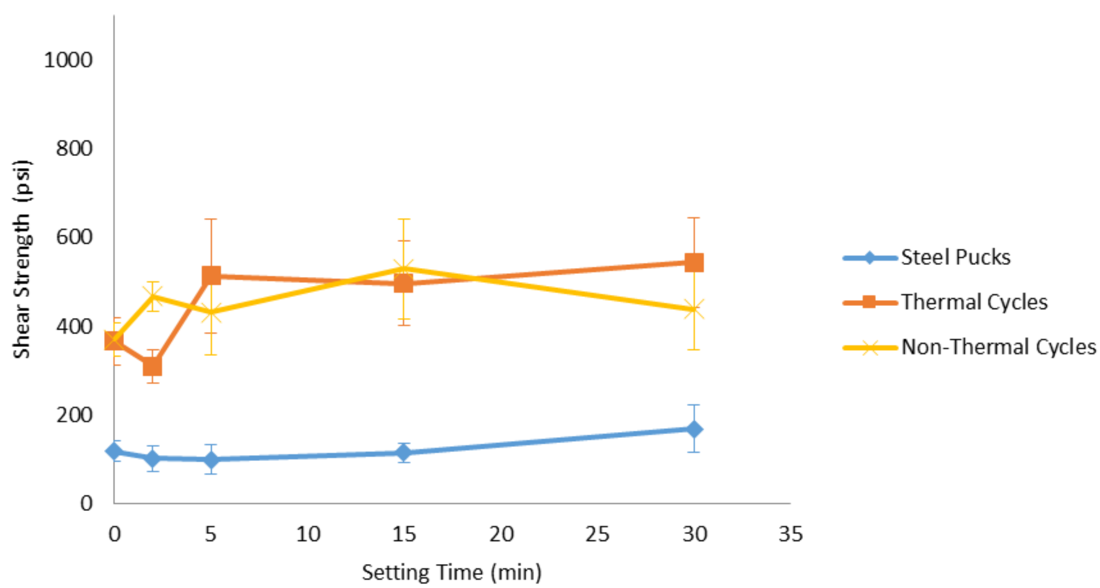

Figure 0.6 PVA agent shear stress

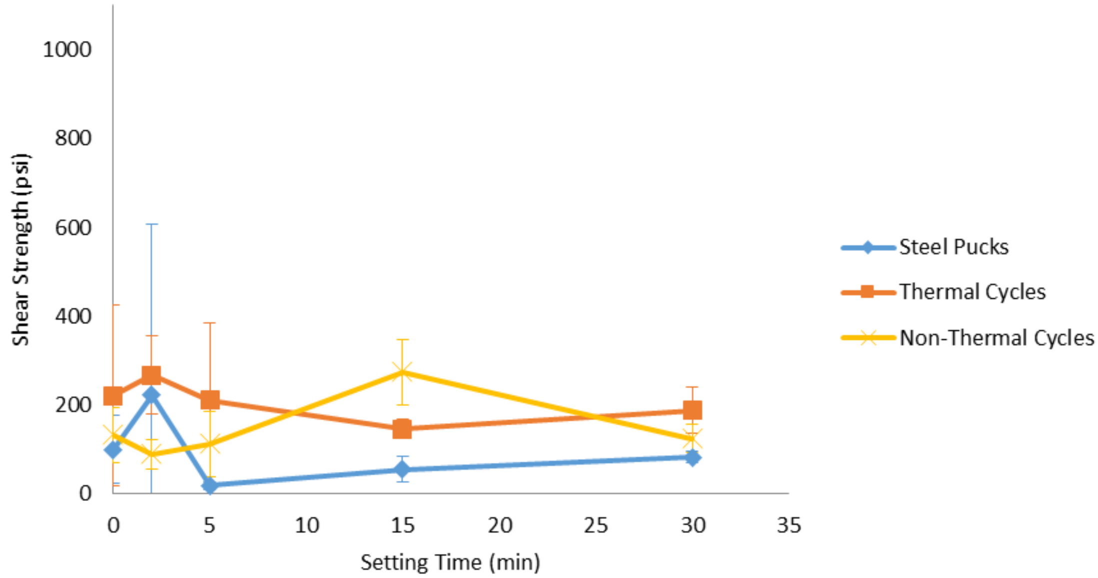

Figure 0.7 Acrylic agent shear stress

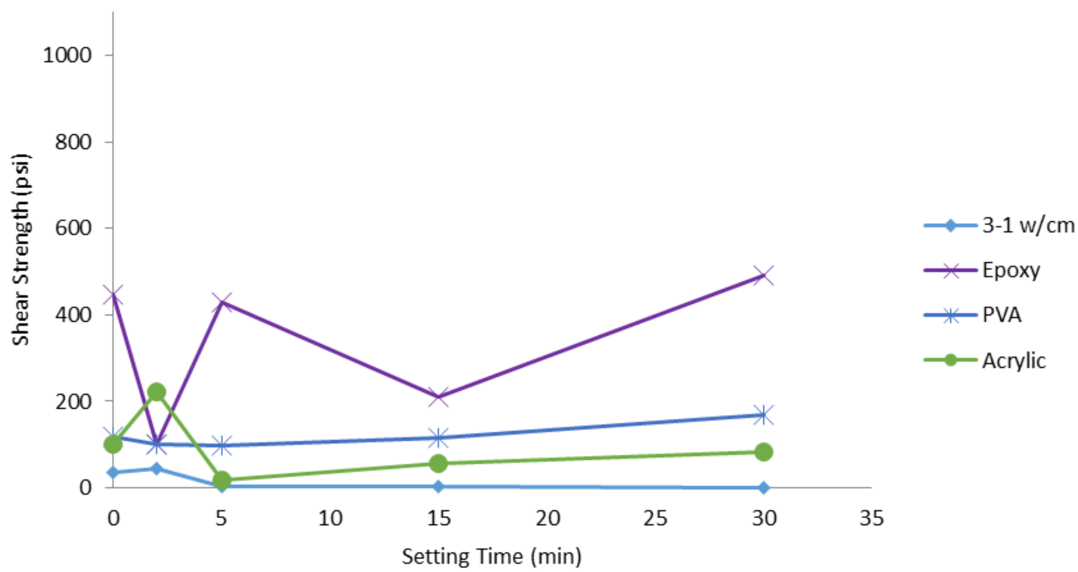

Figure 0.8 Steel control samples shear strength comparison

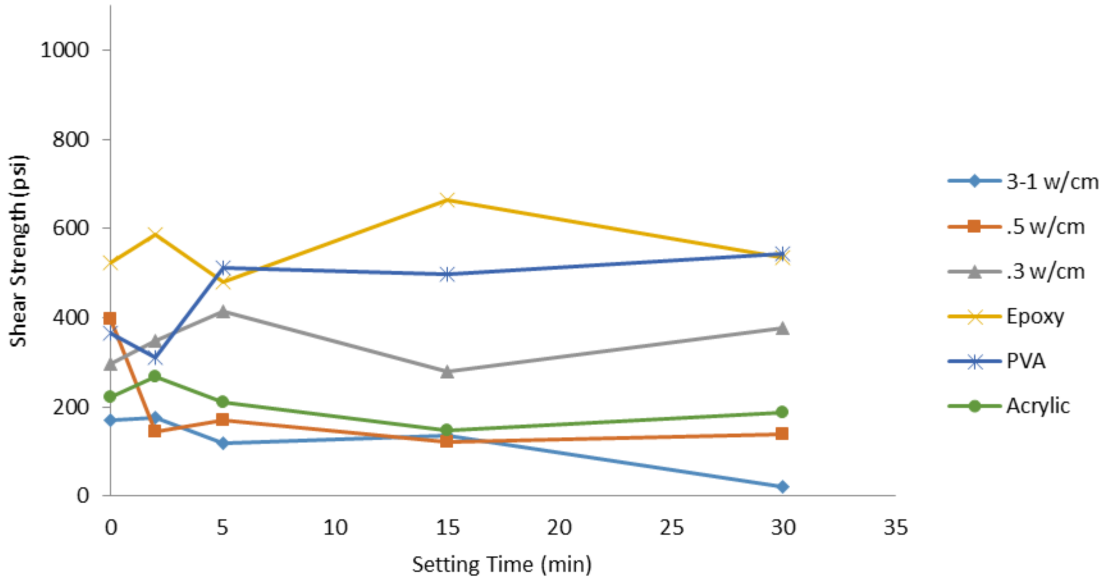

Figure 0.9 5 F-T thermal cycles shear strength comparison

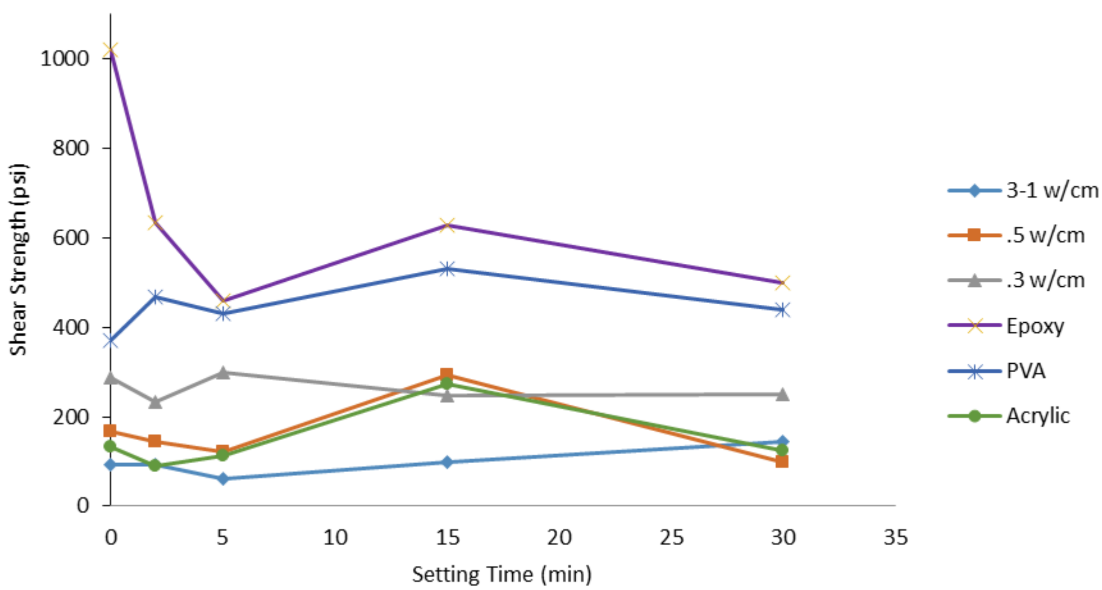

Figure 0.10 Non-thermal cycles shear strength comparison

All bonding agents and repair materials except the MgP experienced failure at the bond interface. The direct shear test caused a clean break at the bond interface between the repair material and the substrate concrete. The MgP experienced failure in the repair material with parts of MgP still attached to the substrate concrete.

5.2 Field Data

Tables 5.2 and 5.3 show the substrate concretes and repair materials compressive strength. The compressive strengths were calculated by averaging 3 compressive strength samples. Figure 5.11 shows the repair material temperature after placement. Figure 5.12 shows the pull-off tensile strength of the repair materials without bonding agents. Figure 5.13 and 5.14 shows the 7 day and 5 month pull-off strength for the concrete repair material when bonding agents were used. Pull-off test strengths reported are the average of the valid tests from the four pull-off tests performed for each repair material drying time. If no more than two sample strengths could be obtained from a setting time, the test was considered void. Appendix B contains the field pull-off data in tabular form.

Table 0.2 Field slab data

	Compressive Strength (psi)			
	7 day	28 day	air %	Materials Used
Slab 1	5550	5865	5.5	Ctrl, Ctrl SSD, MgP, CSA, PM
Slab 2	4417	4973	7.6	Cement Grouts, Epoxy agent, Latex Agents

Table 0.3 Repair material compressive strength

7 Day Repair Material Compressive Strength (psi)				
MGP	CSA	PM	RC1	RC2
3424	4896	8492	6630	6027

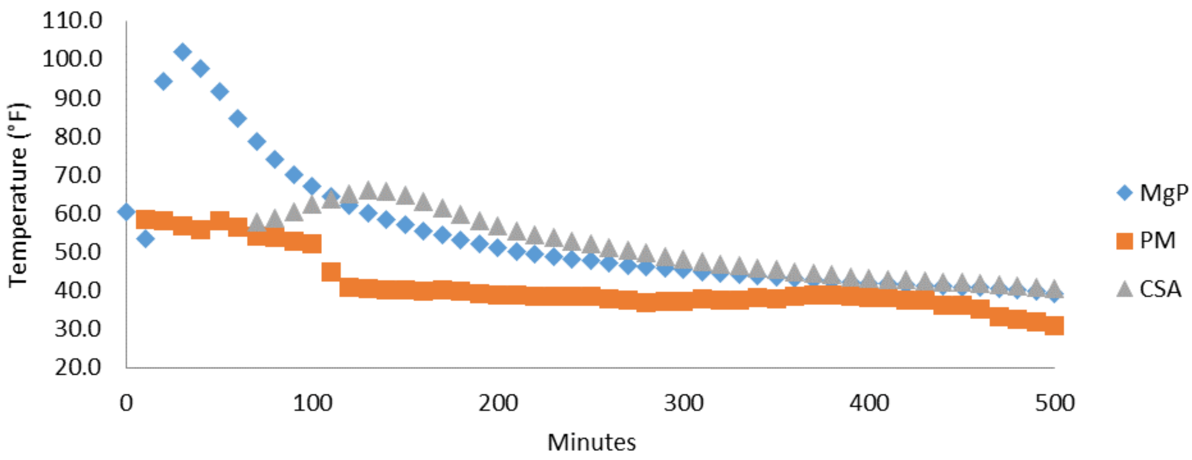

Figure 0.11 Rapid repair material temperature after placement

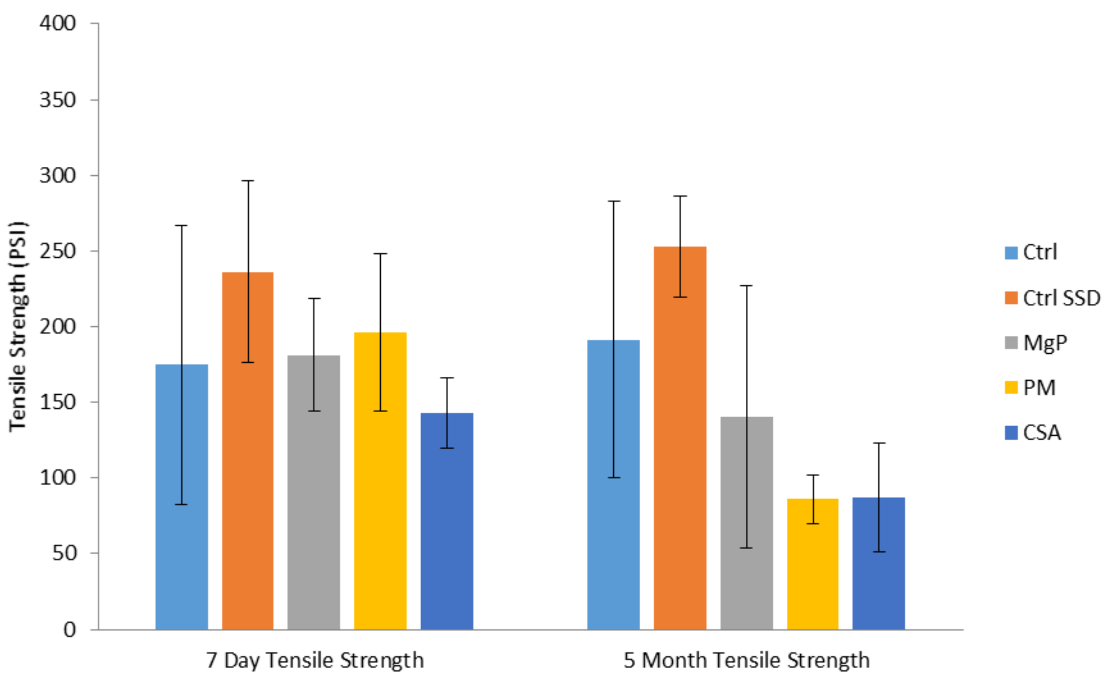

Figure 0.12 Repair material 7 day and 5 month tensile strength

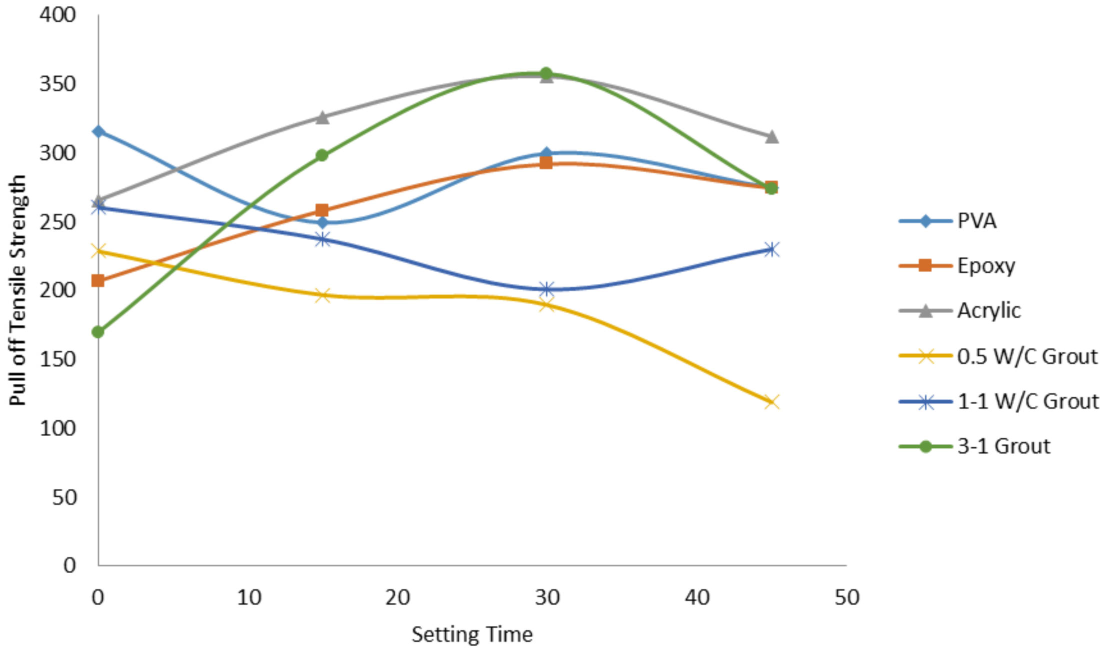

Figure 0.13 Bonding agent 7 day tensile strength

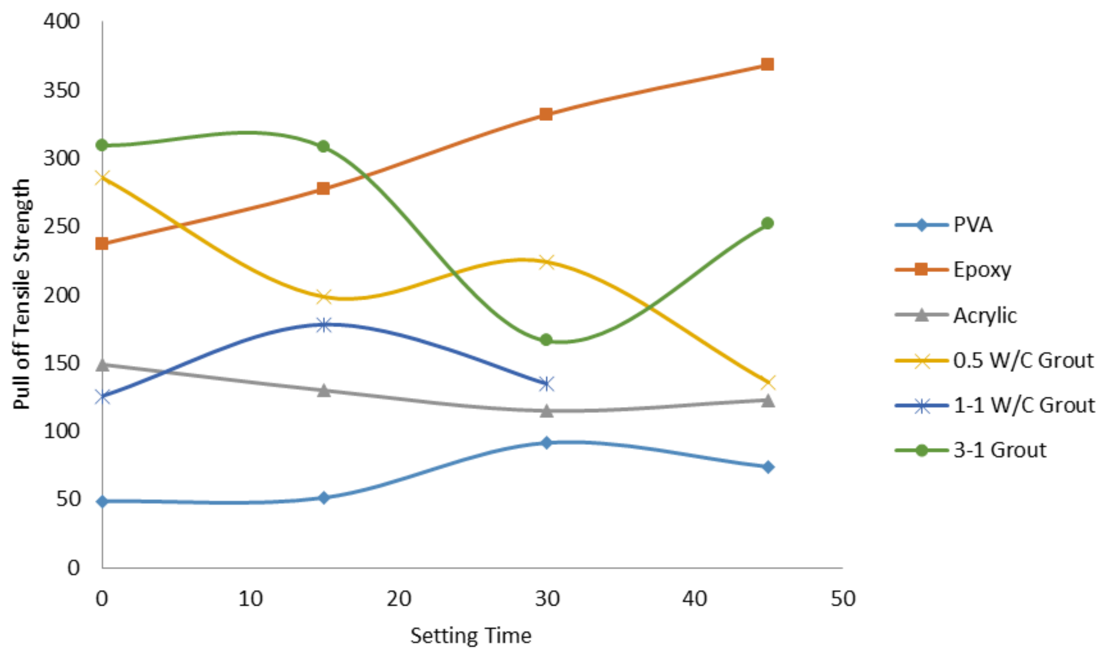

Figure 0.14 Bonding agent 5 month tensile strength

Chapter 6 Discussion

6.1 Laboratory Results

6.1.1 Rapid Repair Material

The samples that were subjected to freeze-thaw cycles without thermal cycling showed that the magnesium phosphate had the highest bond strength. The PM samples had higher bond strength with the steel samples and is known to bond well to steel substrates. This may be beneficial for repairs performed on continuously reinforced concrete pavements. The samples that did not undergo thermal cycles had the highest shear strength, with MgP having the highest shear strength of 570 psi. After the thermal cycles the MgP shear strength dropped to 420 psi. This indicated that MgP cements may lose bond during freeze-thaw cycles. PM had the similar shear strength to the CSA cement for both sample sets subject to thermal cycles and non-thermal cycles.

The rapid repair materials loss of bond due to the thermal cycles could originate from small thermal material differences between the repair materials and the existing concrete. The repair material could also trap water near the interface, causing deterioration during the freezing and thawing cycles. With the loss of bond strength that occurred with the five thermal cycles, the possibility of significant bond loss due to extreme weather events could be increased.

6.1.2 Controls with No Bonding Agents

Both of the control samples with dry and SSD surface conditions subject to thermal cycles had higher shear strength than the sets that were not subjected to the thermal cycling. Shear strengths for the control thermal and non-thermal samples were 340 and 160 psi. Shear strengths for the SSD samples were 210 psi and 120 psi respectively. The dry control samples did have higher shear strength than the SSD samples, but the standard deviation for the non SSD samples was 300 and 100 psi. Wetting the surface prior to repair material placement seemed to lower variability.

The increase in bond strength for both sets of data when the samples were subject to thermal cycles as opposed to the samples that were not, could be due to an acceleration of the cement hydration process at the bond interface that was caused by the oven being at 120°F for 22 hours during each freeze-thaw cycle.

6.1.3 Portland Cement Bonding Agents

Of the three portland cement grouts used, the samples with the highest shear strength were the 0.3 w/c grouts. The grout with the lowest shear strength in both the thermal and non-thermal sets was the 3-1 w/c grout. For all three w/c, the sets of samples that were subject to thermal cycles had higher shear strength than the non-thermal cycles. The 0.3 w/c grout shear strength was also more forgiving with respect to setting time, because as illustrated in figure 5.4, the shear strength never fell below 200 psi for either set. The 0.5 w/c grout was more susceptible to setting time because, as shown in figure 5.3, once 15 minutes of set time has been allowed, the shear strength fell below 200 psi. The 3-1 w/c grout was the most susceptible to setting time with bond strength rapidly dropping after 5 minutes of setting time, as illustrated in figure 5.2.

The increase in bond strength in between the samples that were put through thermal cycles could have also been from the acceleration of the hydration process caused by the oven. All of the cementations repair materials and bonding agents showed similar trends in increase in bond strength after the thermal cycles as opposed to the samples that were left in room temperature.

The decrease in bond strength as the waiting time increased for the high w/c could be caused by segregation of the water and cement during the waiting period. The lower w/c bond agents did not experience the same level of segregation, and even though they dried out some, they did not experience the same level of strength loss with waiting time.

6.1.4 Epoxy and Latex Bonding Agents

The epoxy samples that were subject to thermal cycles had lower strengths than the non-thermal cycles. The standards that the epoxies have to meet though ASTM C881 make it so that the epoxies behave similarly and develop high bond strengths as the results verify. This may be because epoxy bonding agents can have high coefficients of thermal expansion, creating stresses during the thermal cycling.

Of the two latex bonding agents used, the PVA agent had higher strength than the acrylic bonding agent. On average, both sets thermal and non-thermal PVA samples had strength of over 400 psi. The setting time had higher influence on the acyclic bonding agent, since the strength decreased as setting time increased. Since the PVA agent is reemulsifiable and no external water was introduced during laboratory testing, the latex film that was made between the repair material and the existing concrete was not altered with time and the bond strength remained consistent.

The cementitious latex grout agent that was made by using acrylic agent, water, and cement showed similar trend to the cement grouts. The fluids-solids ratio of the grout was 1, but the data showed that the agent had similar strengths to the 0.5 w/c grout. The latex polymers in the agent could have influenced the increase in strength and mirrored the results of the 0.5 grout.

6.2 Field Results

6.2.1 Rapid Repair Materials

For the 7 day pull-off test the three rapid repair materials had similar pull off strengths. Both the MgP and the PM had strengths over 180 psi, while the CSA cement strength was over

140 psi. When 5 month tests were performed, both the PM and CSA cement had strengths reduced below 100 psi and the MgP strength had strength reduced to 140 psi. As illustrated in figure 5.11, the rapid repair materials temperature after placement was low, possibly reducing strength development from table 5.3. The materials were placed in late fall so the cool temperature from the environment during placement could reduce the heat generation from the materials, thus having low strength gain with the materials. The CSA cement showed signs of surface cracks developing a day after placement, as shown in figure 6.1. The MgP cement had scaling visible on the surface after 5 months of outdoor exposure. The scaling could be an indication of poor frost durability and could have contributed to the large strength drop with time in the field.

Figure 0.1 CSA cement with surface cracks

6.2.2 Controls with No Bonding Agents

Both the 7 day and the 5 month pull-off tests had similar results. The control sample with a dry substrate surface had 7 day and 5 month strengths of 170 and 190 psi. The samples with

SSD conditions had strengths of 230 and 250 psi. The control samples' bond strength increased with the 5 months as the repair concrete strength increased after the initial 7 days. The control samples with no bonding agents and a dry substrate surface were able to obtain their strength because of the substrate surface being free of dirt, oils, or foreign substance that can behave as a bond breaker in the bond zone interface. The rough surface produced by needle scabling provided enough interlock to develop bond strength. Having a SSD surface on the existing concrete prevented the substrate concrete from absorbing too much moisture from the repair material into the existing concrete. Having a substrate surface that was saturated with pooling water could lower bond strength because the pooling water would reduce the w/c on the bond later (Courard 2013). For most non-structural concrete partial depth repairs, SSD conditions can be considered an acceptable substitute for the use of bonding agents.

6.2.3 Portland Cement Grouts

The 0.5 and 1-1 w/c grouts both had a pull-off strength of over 200 psi for the 7 day strength test. Both of the grouts showed strength decrease as setting time increased. The 3-1 w/c grout results were inconsistent since the lowest strength was over 150 psi and occurred with no wait time. The 3-1 data showed a strength increase to 250 psi after 15 minutes of wait time. It is possible that in field conditions, the drier substrate concrete with a larger concrete volume under the repair could have absorbed more water than in the laboratory tests, effectively lowering the grout w/c with time, without causing segregation.

For the 5 month strength test the 0.5 w/c grout had initial strength over 250 psi, but as setting time increased, the strength reduced below 200 psi. The 1-1 w/c grout had strengths that were consistently around 150 psi. The 1-1 grout strengths were lower than the 0.5 grout strengths. The 3-1 grout produced good bond strengths at 5 months.

The 0.5 and 1-1 w/c grouts had similar trends with bond strength loss as wait time increased. The grouts could have experienced excess moisture loss with time from absorption and evaporation. The loss in strength was more dramatic in the field testing because of the field environment effects during the grout application that allowed for more water to evaporate from the grout than the evaporation and drying that occurred in the laboratory testing.

6.2.4 Epoxy and Latex Bonding Agents

For the 7 day strength test the epoxy, PVA, and acrylic bonding agents had a consistent pull-off strength of over 250 psi. The strengths showed no trend as setting time of the agents increased. The three bonding agents had not been exposed to the extreme changing temperature effects and moisture that is experienced in northeast Kansas.

The 5 month tests showed that the epoxy still had a pull-off strength of over 250 psi for all setting times. The epoxy is the most consistent of all the bond agents examined and was shown to provide the highest bond strength.

The latex bonding agents experienced strength loss after the 5 months of weather exposure. The acrylic agent experienced significant strength loss over the winter period. The acrylic agent used was non-reemulsifiable, however, some reemulsion could have occurred. Additionally, the acrylic agent could have helped trap more moisture at the interface, causing some damage during freezing and thawing. The PVA bonding agent showed the lowest strength of 50 psi because it was reemulsibiable. When the field slabs were exposed to weathering the latex film at the bond interface broke down, lowering the bond strength.

Chapter 7 Conclusion and Recommendations

7.1 Conclusions

When comparing the control samples to one another, the field data suggests that samples with an SSD condition will have higher tensile pull-off strength than the dry substrate samples. When not using bonding agents, the SSD condition on the substrate concrete should be used to achieve higher bond strength than dry surface conditions. If portland cement grouts are to be used as an bonding agent, grouts with a w/c of 1 or less can provide an increase in bond strength. From the measured data from this project it can be stated that portland cement grouts are more susceptible to drying times. The grouts had a higher shear and tensile strengths if the repair material was placed before 15 minutes of wait time. Once the setting time had passed 15 minutes a trend of lowered bond strength could be observed. A problem encountered was that once the w/c was lowered below 0.5 the workability of the grout was lowered, making the grouts harder to work with and apply. Grouts with a w/c over 1 also showed the highest decrease in bond strength with respect to setting time compared to the other w/c grouts. If using a cementitious bonding agent, a w/c of 1 is recommended to give the best balance between workability, strength, and lower sensitivity to wait times.

The epoxy bonding agent had the best performance of the bonding agents tested. The epoxy agent had low sensitivity to wait time as long as the repair material was placed while the epoxy was still tacky. The acrylic and PVA bonding agent's bond strengths were higher when compared to the portland cement grouts in the laboratory testing and the initial 7 day pull off test. When the agents were subject to the 5 month pull off test, both latex bonding agents' strength had decreased below the cement grout's strength. The PVA bonding agent, which is the reemulsifiable agent, experienced the lowest bond strength of all the bonding agents used in the field after 5 months and is not recommended for use in pavements or in wet conditions.

The rapid repair materials’ shear strength during the laboratory testing was higher when compared to the control samples. The repair materials had 7 day pull-off strengths that were similar to control samples, but after 5 months of weathering, the bond strength of the repair materials dropped dramatically to almost a 50% reduction in strength. Rapid repair materials can set up fast, which is favorable in time sensitive conditions, but the 5 month bond strength results show poor bond development over time in freezing and thawing conditions.

7.2 Future Research

With the inadequate performance of the rapid repair materials used during the field testing, more in depth research should be performed on how the outdoor environment influences bond strength between the material and the existing pavement. A microstructural investigation of the bond interface would be beneficial.

During the field testing when examining bond strength, exposure to traffic on the partial depth should be examined to observe durability of the repair since this study only exposed the repair to thermal and environmental weathering.

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Appendix A Laboratory Data

Table Error! No text of specified style in document.**.1** Ctrl, Ctrl SSD, MgP, PM, CSA cement, strength and standard deviation

Shear Strength (PSI)					
	Control	Control SSD	Mag. Phosphate	Pavemend	CSA Cement
5 F-T Cycles	344	213	429	274	331
Non-Thermal	164	122	571	122	327
Steel Control	9	52	251	400	42
Standard Deviation					
5 F-T Cycles	313	54	90	46	155
Non-Thermal	134	37	207	97	98
Steel Control	15	35	89	101	25

Table Error! No text of specified style in document.**.2** 3-1 W/C shear strength and standard deviation

3-1 w/c Grout					
Setting Time Shear strength (psi)					
	0	**5**	**10**	**15**	**30**
5 F-T Cycles	171	175	118	135	7
Non-Thermal	94	94	60	97	145
Steel Substrate	37	43	3	2	-
Standard Deviation					
5 F-T Cycles	100	99	61	149	-
Non-Thermal	37	44	29	41	81
Steel Substrate	42	4	-	-	-

Table Error! No text of specified style in document.**.3** 0.5 grout shear strength and standard deviation

0.5 w/c Grout					
Setting Time Shear strength (psi)					
	0	**5**	**10**	**15**	**30**
5 F-T Cycles	398	143	170	81	138
Non-Thermal	167	145	120	292	98
Steel Substrate	-	-	-	-	-
Standard Deviation					
5 F-T Cycles	120	77	14	29	48
Non-Thermal	21	21	60	149	69
Steel Substrate	-	-	-	-	-

Table Error! No text of specified style in document.**.4** 0.3 grout shear strength and standard deviation

0.30 w/c Grout					
Setting Time Shear strength (psi)					
	0	**5**	**10**	**15**	**30**
5 F-T Cycles	297	348	414	280	376
Non-Thermal	287	233	298	246	251
Steel Substrate	-	-	-	-	-
Standard Deviation					
5 F-T Cycles	89	30	138	61	137
Non-Thermal	74	124	23	32	113
Steel Substrate	-	-	-	-	-

Table Error! No text of specified style in document.**-5** PVA shear strength and standard deviation

PVA Bonding Agent					
Setting Time Shear strength (psi)					
	0	**5**	**10**	**15**	**30**
5 F-T Cycles	366	310	513	497	544
Non-Thermal	94	94	60	97	145
Steel Substrate	371	468	432	530	439
Standard Deviation					
5 F-T Cycles	54	38	128	94	101
Non-Thermal	38	33	97	113	93
Steel Substrate	23	29	33	22	52

Table Error! No text of specified style in document.**-6** Epoxy shear strength and standard deviation

Epoxy					
Setting Time Shear strength (psi)					
	0	**5**	**10**	**15**	**30**
5 F-T Cycles	525	587	480	665	535
Non-Thermal	1020	635	460	629	500
Steel Substrate	446	101	430	210	490
Standard Deviation					
5 F-T Cycles	106	137	117	200	91
Non-Thermal	140	274	168	117	182
Steel Substrate	68	29	231	107	382

Table Error! No text of specified style in document.**-7** Acrylic shear strength and standard seviation

Acrylic Bonding Agent					
Setting Time Shear strength (psi)					
	0	**5**	**10**	**15**	**30**
5 F-T Cycles	221	267	211	146	187
Non-Thermal	133	89	112	274	124
Steel Substrate	100	222	18	55	82
Standard Deviation					
5 F-T Cycles	203	89	173	21	52
Non-Thermal	62	33	74	74	32
Steel Substrate	76	385	10	29	13

Appendix B Field Data

Table Error! No text of specified style in document.**-1** 7 Day bond failure location

	Type of Break 7-day											
	PVA				Epoxy				Acrylic			
Wait Time	0	15	30	45	0	15	30	45	0	15	30	45
Pull-Off Test 1	2	3	3	3	3	3	3	3	2	4	2	4
Pull-Off Test 2	3	3	2	3	3	3	3	3	2	3	2	3
Pull-Off Test 3	3	3	3	3	3	3	3	3	3	2	3	2
Pull-Off Test 4	3	2	2	2	3	3	3	4	3	4	4	2
	.5 W/C grout				1-1 Grout				3-1 Grout			
Wait Time	0	15	30	45	0	15	30	45	0	15	30	45
Pull-Off Test 1	1	2	1	2	2	1	2	2	3	2	4	2
Pull-Off Test 2	3	2	2	2	2	3	2	3	3	3	2	3
Pull-Off Test 3	1	1	1	2	1	1	2	3	3	2	2	2
Pull-Off Test 4	2	4	2	2	2	3	2	3	4	1	2	4

Table Error! No text of specified style in document.**-2** 5 month bond failure location

	Type of Break 5-Month											
	PVA				Epoxy				Acrylic			
Wait Time	0	15	30	45	0	15	30	45	0	15	30	45
Pull-Off Test 1	2	2	2	2	2	2	1	2	2	2	2	2
Pull-Off Test 2	2	2	2	2	2	2	2	3	2	2	2	2
Pull-Off Test 3	2	2	2	2	2	3	3	1	2	2	2	2
Pull-Off Test 4	2		2		2		2	3	2		2	
	.5 W/C grout				1-1 Grout				3-1 Grout			
Wait Time	0	15	30	45	0	15	30	45	0	15	30	45
Pull-Off Test 1	2	2	2	2	2	2	2		2	2	3	2
Pull-Off Test 2	2	2	2	2	2	2	2		3	2	3	2
Pull-Off Test 3	2	2	2	2	2	2	2		2	2	2	2
Pull-Off Test 4	2	2	2	2	2				2		2	

Table Error! No text of specified style in document.**-3** PVA and epoxy 7 day tensile strength

	7 Day Tensile Strength (PSI)							
	PVA				Epoxy			
Wait Time	0	15	30	45	0	15	30	45
	412	294	247	248	175	224	326	305
	344	186	318	188	205	295	239	272
	282	286	311	294	241	226	268	
Average Strength	316	250	300	275	207	258	292	275
Standard Deviation	80	51	36	76	27	39	46	29

Table Error! No text of specified style in document.**-4** Acrylic and 0.5 w/c grout 7 day tensile strength

	7 Day Tensile Strength (PSI)							
	Acrylic				.5 W/C grout			
Wait Time	0	15	30	45	0	15	30	45
	333	512	335	412	181	143	198	64
	267	232	184	198	258	166	96	188
	245	235	435	405	297	266	220	105
	218		469	233	179	213	245	
Average Strength	266	326	356	312	229	197	190	119
Standard Deviation	49	161	128	112	59	54	65	63

Table Error! No text of specified style in document.**-5** 1-1 grout and 3-1 grout 7 day tensile strength

	7 Day Tensile Strength (PSI)							
	1-1 Grout				3-1 Grout			
Wait Time	0	15	30	45	0	15	30	45
	107	224	162	200	120	324	267	358
	200	295	149	235	169	260	375	163
	464	198	207	335	220	280	320	341
	271	233	286	151		328	469	233
Average Strength	261	238	201	230	170	298	358	274
Standard Deviation	151	41	62	70	50	33	86	92

Table Error! No text of specified style in document.**-6** PVA and epoxy 5 month tensile strength

	5 Month Tensile Strength (PSI)							
	PVA				Epoxy			
Wait Time	0	15	30	45	0	15	30	45
	36	15	11	166	326	235	346	341
	87	118	120	24	236	209	335	427
	19	21	109	32	218	389	331	412
	53		126		169		316	294
Average	48.75	51	92	74	237.25	277.7	332	369
Standard Deviation	29	58	54	80	66	97	12	62

Table Error! No text of specified style in document.**-7** Acrylic and 0.5 w/c grout 5 month tensile strength

	5 Month Tensile Strength (PSI)							
	Acrylic				.5 W/C grout			
Wait Time	0	15	30	45	0	15	30	45
	124	162	87	169	307	201	135	122
	149	62	198	68	307	162	215	209
	175	166	75	132	329	233	218	77
	148		100		201	198	329	56
Average	149	130	115	123	286	199	224	136
Standard Deviation	21	59	56	51	58	29	80	67

Table Error! No text of specified style in document.**-8** 1-1 grout and 3-1 grout 5 month tensile strength

	5 Month Tensile Strength (PSI)							
	1-1 Grout				3-1 Grout			
Wait Time	0	15	30	45	0	15	30	45
	120	15	70		309	224	158	288
	47	407	166		364	404	256	291
	184	113	169		294	296	119	176
	152				271		132	
Average	126	178	135		310	308	166	252
Standard Deviation	59	204	56		37	91	62	66