DEVELOPMENT OF HIGH-PERFORMANCE RAPID PATCHING MATERIALS FOR PAVEMENT REPAIR

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16. Abstract

Concrete pavements exhibiting severe distresses which require patching are commonly observed in the concrete pavement in Nebraska. Due to the requirements of opening pavement to traffic after placing the rapid patching materials, it is essential for that concrete to achieve high early strength. To ensure this, a high cement content and chloride-based accelerators are currently used in the Nebraska Department of Transportation (NDOT) Portland cement-based rapid-patching materials. Besides its associated high cost, high cement content tends to result in a less stable mix with high shrinkage, high heat of hydration, and high cracking potential. In addition, using chloride-based accelerators has adverse effects on concrete durability. Also, the effect of the low ambient temperature has a considerable impact on the strength gain and needs to be assessed to estimate the traffic opening. Therefore, this project studied the performance of rapid patching materials for three different aspects: reducing cement content through optimizing aggregate gradation, replacing conventional calcium chloride with a non-chloride accelerator, and partial replacement of type I/II or type III cement with type IP cement. Fresh, early-age, mechanical, durability performance and constructability were evaluated on each of the developed mixture design. The performance of developed mixes at low ambient temperature (50 and 60°F) was also evaluated. Overall, it appears that, with the optimized aggregate gradation, mixes with reducing cement content by up to 100lb/yd^3 together have good constructability and can meet the general requirements, which were confirmed from the evaluation of key parameters, including early-age compressive strength, modulus of rupture, bond strength, surface resistivity, drying shrinkage, and alkali-silica reaction (ASR) resistivity. The non-chloride-based accelerator showed promising behavior as an alternative accelerator. The developed mixes exhibit satisfy early-age and 28-day compressive strength, modulus of rupture, and bond strength. The free shrinkage can be reduced by up to 30% with the lower cement content. The tendency of ASR deterioration can be reduced significantly by replacing 50% Type III cement with Type IP cement. Finally, as expected, when experiencing a low ambient temperature, strength growth can be delayed and employing PR3 mixes will be a more viable option to reduce the traffic closure durations.

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ABSTRACT

Concrete pavements exhibiting severe distresses which require patching are commonly observed in the concrete pavement in Nebraska. Due to the requirements of opening pavement to traffic after placing the rapid patching materials, it is essential for that concrete to achieve high early strength. To ensure this, a high cement content and chloride-based accelerators are currently used in the Nebraska Department of Transportation (NDOT) Portland cement-based rapid-patching materials. Besides its associated high cost, high cement content tends to result in a less stable mix with high shrinkage, high heat of hydration, and high cracking potential. In addition, using chloride-based accelerators has adverse effects on concrete durability. Also, the effect of the low ambient temperature has a considerable impact on the strength gain and needs to be assessed to estimate the traffic opening. Therefore, this project studied the performance of rapid patching materials for three different aspects: reducing cement content through optimizing aggregate gradation, replacing conventional calcium chloride with a non-chloride accelerator, and partial replacement of type I/II or type III cement with type IP cement. Fresh, early-age, mechanical, durability performance and constructability were evaluated on each of the developed mixture design. The performance of developed mixes at low ambient temperatures (50 and 60°F) was also evaluated. Overall, it appears that, with the optimized aggregate gradation, mixes with reducing cement content by up to 100lb/yd^3 together have good constructability and can meet the general requirements, which were confirmed from the evaluation of key parameters, including early-age compressive strength, modulus of rupture, bond strength, surface resistivity, drying shrinkage, and alkali-silica reaction (ASR) resistivity. The non-chloride-based accelerator showed promising behavior as an alternative accelerator. The developed mixes exhibit satisfy early-age and 28-day compressive strength, modulus of rupture, and bond strength. The free shrinkage can be reduced by up to 30% with the lower cement content. The tendency of ASR deterioration can be reduced significantly by replacing 50% Type III cement with Type IP cement. Finally, as expected, when experiencing a low ambient temperature, strength growth can be delayed, and employing PR3 mixes will be a more viable option to reduce the traffic closure durations.

CHAPTER 1 . INTRODUCTION

1. 1 Background

Concrete pavement distresses resulting from freezing/thawing (F/T) deterioration, alkali-silica reaction (ASR), and chemical attacks may cause different forms of deterioration, including scaling, cracking, breaking, chipping, and fraying. Concrete pavements exhibiting severe distresses such as transverse cracks, shattered slabs, and corner breaks require patching are commonly observed in concrete pavements in Nebraska. Due to the opening requirement of the pavement to traffic after placing the repair concrete, it is essential to achieve high early strength. To ensure high early strength, the Nebraska Department of Transporation (NDOT) current patching mix (i.e., PR mixes in the NDOT specification 1002.02) requires a minimum cement content of 752 or 799pcy for PR1 and PR3 mixes respectively. Besides the associated high cost, the high cement content tends to result in a less stable mix with a high drying shrinkage, high autogenous shrinkage, high heat of hydration, and cracking potential. The mixes also exclude the use of fly ash, which makes it vulnerable to various deteriorations, particularly ASR. An example of premature failure of pavement repair can be found in Figure 1. In order to reduce the material cost and premature failures of pavement repair, patching materials that develop early strength and are durable is needed.

Figure 1-1. Example of premature failure of patching material

This study is conducted to improve the current rapid patching materials. The research team particularly focused on mix design in terms of aggregate gradation, cement type and content, water-to-cement ratio (w/c), and incorporation of proper chemical admixtures to achieve sufficient early-age strength that is comparable to the current NDOT PR mixes, yet more durable and resistant to durability issues.

1. 2 Research Objectives

The goal of this study is to develop cost-effective and durable high-performance rapid patching materials for full-depth concrete pavement repair. An experimental assessment was conducted to improve rapid patching concrete mixtures by reducing cement content through optimization of aggregate gradation and evaluating new materials as replacement of current Nebraska DOT.

To achieve this goal, two specific objectives of this study are selected as:

1. Develop cost-effective and more durable patching materials that provide sufficient early strength (a minimum 3,000psi compressive strength within 8 hours) for proper traffic opening;
2. Ensure the overall performance of the developed patching materials satisfy NDOT requirements, including fresh, hardened, and durability properties.

1. 3 Organization of the Report

The report is divided into seven chapters. Chapter 1 is an introduction, where the general background and main objectives are provided. A literature review is presented in Chapter 2, which includes a summary of commonly used materials and design, as well as DOT practices for rapid patching. Chapters 3 and 4 include the main experimental program and results covering different mixes developed through the project. Chapter 6 covers the constructability of developed patching mixtures through patching lab-scale slabs. Cost-effectiveness and feasibility study are discussed in Chapter 6. Chapter 7 summarizes all conclusions and provides recommendations for future studies.

CHAPTER 2 CHAPTER 2. BACKGROUND

2. 1 Introduction

The development of concrete with high early strength has always been a challenge for concrete researchers and practitioners. One of the most common applications of this type of concrete is in pavement repair material, as limited traffic opening time urges pavement contractors to use concrete with sufficient compressive or flexural strength gain in early hours (Dornak et al. 2015; Cramer et al. 2017). It is reported that the range of minimum compressive and flexural strengths at different traffic opening times in different states is approximately between 2,000 to 3,500 psi and 300 to 500 psi, respectively (Ghafoori et al. 2017).

This chapter is to summarize the previous and current methods which are common in the concrete pavement full-depth repair with the emphasis of taking different cement types and content, water to cementitious ratio, aggregate gradation, and chemical accelerators into account.

Several approaches are used to design a concrete mixture with proper early-age mechanical properties. Often, Type I and Type III Portland cement and other special cements such as regulated set cement, rapid hardening cement, calcium aluminate cement, and magnesium phosphate cement, with or without accelerators, as well as prepackaged proprietary cement mixtures, are routinely employed (Kosmatka and Wilson 2011; Dornak et al. 2015). Type I Portland cement, along with an accelerator, is commonly utilized for traffic openings within 6 to 8 hours, while Type III Portland cement is used for shorter openings. Further, proprietary cement mixtures can target even earlier openings. Although Type III Portland cement demonstrates better performance in early-strength gain, certain issues lead to Type I cement being used more often in practice. For example, autogenous shrinkage is significantly higher in systems prepared with finer cement, as in the case of Type III cement, which in turn results in a higher chance of shrinkage cracking. Moreover, it is common knowledge that Type III cement needs higher w/c rather than Type I cement which results in a higher shrinkage rate during early ages (American Concrete Pavement Association 1989).

Accelerators are widely used for the reduction of setting times or to expedite early strength gain. They typically target to accelerate aluminate phase (C_3A) reactions in hydration processes (Cheung et al.; Todd et al.). Among various accelerators, Calcium Chloride is the most well-known one, but the presence of Chloride causes hazardous issues such as a tendency toward corrosion in reinforcing bars or excessive slab cracking (mostly due to autogenous shrinkage). As a result, most states' pavement specifications prohibit the use of this type of admixture (Aggoun et al.; Myrdal 2007). Therefore, a number of non-Chloride accelerators (mostly based on inorganic salts) were developed to replace Calcium Chloride (Aggoun et al.; Justnes and Nygaard 1996). Calcium Nitrate was found to be an effective alternative that could favorably accelerate early compressive strength, yet without major durability concerns. Also, Calcium Nitrate was described as a corrosion inhibitor and antifreeze agent. However, its initial cost-to-performance ratio is relatively higher than that of Calcium Chloride (Aggoun et al. 2008; Karagöl et al. 2019).

As the opening to traffic times are different in each state due to various traffic loads or temperature ranges (Collier 2016), each state has its own patch design description; accordingly, a review of their specifications has been gathered in the next sections.

2. 2 Commonly Used Cement for Rapid Patching

High-early-strength can be achieved by employing different cement types as specified below:

2. 2. 1 Ordinary Portland Cement

Generally, Type I and III Portland cement are typically used in patching materials (Kosmatka et al. 2002). Based on ASTM C150, Type III cement is employed when high early strength is desired. However, Type I cement could be used with lower water to cement ratio to achieve high early strength (American Concrete Pavement Association 1989). Generally, Type I and III Portland cements have similar chemical properties, with the main difference that the latter is grounded finer and contains a higher amount of tricalcium aluminate (Kosmatka et al. 2002).

2. 2. 2 Regulated Set Cement

Regulated set (RS) cement is a Portland-based cement with the replacement of about 20 to 25% Calcium Aluminate phases with Calcium Fluor Aluminate, which reduces the setting times and helps to gain early strength (Kosmatka et al. 2002). One of the important applications could be in cold-weather concrete placement as the hydration heat generates immediately after the cement is mixed with water (ASTM C494 2015).

2. 2. 3 Calcium Aluminate Cements

One of the Calcium Aluminate Cement (CAC) employment is in early strength gain and repair applications (individual or combined with Portland Cement). The main compound in any CAC is monocalcium aluminate (CA) and typically comprises 40 to 60% of the cement. The second most available compound is Mayenite; this phase plays an important role in the decrease of setting times (Dornak et al. 2015). It is to be noted that w/c at a maximum of 0.40 should be used. Nevertheless, the unstable hydration products cause incremental of porosity and reduction of compressive strength (Kosmatka et al. 2002).

2. 2. 4 Magnesium Phosphate Cements

Magnesium Phosphate Cements (MPC) is a hydraulic cement. The main advantage of MPC is no need for water in curing and extremely low shrinkage ratios rather than Portland cement-based patches (Ding et al.; Li et al. 2014). However, lack of sufficient toughness, sensitivity in employing water content tolerance and above all, the prohibition of using Calcium-based aggregates in their designs limit the application of MPC in patching materials (Sommerville 2013).

2. 3 Commonly Used Accelerating Admixtures for Rapid Patching

Rather than using special cements, cement hydration reactions could be accelerated by employing different chemical substances. Based on ASTM C494, type C (Accelerating admixtures) and type E (water-reducing and accelerating admixtures) are applicable to be used in a patch mixture design. Accelerators affect the hydration process in two different steps. Set accelerating admixtures decrease the initiation of the transition of the mix from the plastic to the rigid state, while hardening accelerating admixtures increase the rate of development of early strength in the concrete, with or without affecting the setting time (Myrdal 2007). As it is shown in Figure 2-1, the heat of hydration compared in three different mixture designs. The Reference diagram specifies cement paste without the accelerator, while the others used hardening and set accelerators. The area beneath the stetting and reference diagram indicates that the heat of hydration is equivalent and the mixture with the difference that hydration reactions start in setting accelerator faster. However, the hardening accelerator resulted in a higher heat of hydration.

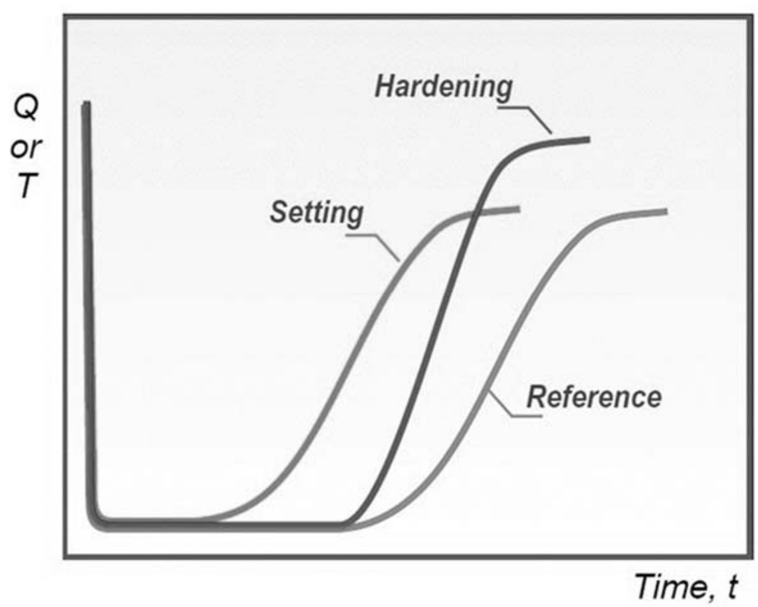


Figure 2-1. Rate of heat evolution Q (W/kg) or temperature T (°C) during hydration of cement (adopted from Myrdal 2007)

2. 3. 1 Calcium Chloride

Calcium Chloride ($CaCl_2$) is one of the most common accelerating admixtures for non-reinforced concrete and is described as the oldest, the cheapest and the most effective accelerator to the date for ordinary Portland cement-based concrete. $CaCl_2$ affects both the setting and hardening of OPC which makes it popular. However, its corrosive nature has prohibited it from being used in many specifications. Besides, there are crucial drawbacks such as slump loss, excessive slab cracking and extensive shrinkage (Myrdal 2007).

2. 3. 2 Calcium Nitrate

Calcium Nitrate ($Ca(NO_3)_2$), as a non-Chloride accelerator, has been used in chemical formulas, in many popular commercial accelerating admixtures (Myrdal 2007). However, based on previous studies (Justnes and Nygaard 1996; Myrdal 2007), Calcium Nitrate is less efficient than Calcium Chloride on hardening criteria.

2. 3. 3 Calcium Nitrite

The difference between Calcium Nitrite and Calcium Nitrate is only the oxidation state of Nitrogen. If the oxidation state of Nitrogen changes from 7 to 5, the Calcium Nitrite appears. Calcium Nitrite ($Ca(NO_2)_2$) has probably been the most popular non-Chloride setting accelerator in the USA industry since the late 1960s. However, the major challenge of Nitrites are toxic nature and not environmentally friendly.

2. 3. 4 Sodium Thiocyanate

The most recent non-Chloride hardening accelerating admixture is Sodium Thiocyanate (NaSCN). Unlike the Nitrates and Nitrites salts, it influences only the hardening. However, the combination of Sodium Thiocyanate and Calcium Nitrate could provide the one-day strength as same as Calcium Chloride. The disadvantages of Sodium Thiocyanate are: more expensive (in comparison with the Chloride-based and other non-Chloride accelerators), hazardous properties, and the high amount of alkalis introduced into the concrete and the corresponding potential for alkali-silica reaction.

Also, regarding the pavement repair application, Sodium Thiocyanate usage makes the patch more susceptible to deterioration issues such as ASR (Myrdal 2007). Table 2-1 summarized the advantages and disadvantages of the above-mentioned accelerators.

Table 2-1 Summary of advantages and disadvantages of inorganic-based accelerators common in concrete repairing (Myrdal 2007)

Accelerator Type	Advantages	Disadvantages
Calcium Chloride	Low cost/performance ratio	Extensive shrinkage and cracking Corrosion hazards
Calcium Nitrate	Low durability issues (due to the absence of chloride ion) Effective inhibitor against chloride-induced corrosion	More of a set accelerator than hardening one High cost/performance ratio
Calcium Nitrite	Low durability issues (due to the absence of chloride ion) Popularity	Toxic and environmentally unfriendly High cost/performance ratio Less effective than chloride-based accelerators
Sodium Thiocyanate	Low durability issues (due to the absence of chloride ion)	High amount of alkalis introduced into the concrete High cost/performance ratio

2. 4 Proprietary Blends Used for Rapid Patching

As a substitute for traditional methods of rapid pathing, many commercial blends have emerged due to their reliability and ease of use, and have attracted many contractors and agencies. There are a variety of blends such as silica-fume cements, alkali-activated blends, and gypsum-modified Portland cements, etc. Although these products shorten the opening to traffic significantly, there are still durability concerns about their long-term performance (Zuniga 2013). In addition, the extremely short setting time could result in construction issues. This type of patching material could be a good option for emergency repairs, in which strength gain needs to be less than four hours (Ghafoori et al. 2017).

2. 5 DOT Rapid Patching Practices

2. 5. 1 Nebraska DOT Rapid Patching Practice

The Nebraska Department of Transportation (NDOT) current practice has the target of minimum compressive strength of 3000 psi at 4-8 hours by using different types of Portland cements and Calcium Chloride as an accelerator. Related details are shown in Table 2-2, with the PR1 and PR3 concrete classes specific for the concrete pavement repair.

Table 2-2. Current Nebraska Department of Transportation mix designs

Class of Concrete (1)	Base Cement Type	Total Cementitious Materials Min. lb/cy	Total Aggregate		Air Content % Min.-Max. (2)	Coarse Aggregate (%)	Water/Cement Ratio Max. (3)	Required Strength Min. psi
			Min. lb/cy	Max. lb/cy				
47B**	IP/IS/IT*	564	2850	3150	6.5 - 9.0	-	0.45	3500
47B***		564	2850	3150	6.0 - 8.5	-	0.45	3500
47BD		658	2500	3000	6.0 - 8.5	30+3	0.42	4000
47B-HE		752	2500	3000	6.0 - 8.5	30±3	0.40	3500
BX(4)		564	2850	3150	6.0 - 8.5	-	0.45	3500
47B-OL****		564	2850	3200	5.0 - 7.0	30±3	0.36	4000
PR1	I/II	752	2500	2950	6.0 - 8.5	30±3	0.36	3500
PR3	III	799	2500	2950	6.0 - 8.5	30±3	0.45	3500
SF(5)	I/II	589	2850	3200	6.0 - 8.5	50±3	0.36	4000

Current accelerator usage specifications, per NDOT current patching practice, the Calcium Chloride for use in PR concrete shall be either:

1. A commercially prepared solution with a concentration of approximately 32 percent by weight.

2. A contractor prepared a solution made by dissolving 4.5 pounds of Grade 2 or 6.2 pounds of Grade 1 Calcium Chloride per gallon of water to provide a solution of approximately 32 percent by weight.

The 7.4 pounds of water in each gallon of the solution shall be considered part of the total water per batch of concrete. The Calcium Chloride solution shall be added, just prior to placement, at a rate of 0.375 gallons/100 pounds of cement (1.4 lb. Calcium Chloride per 100 lb. cement). Class A Flaked or Pellet Calcium Chloride shall be added at a rate not to exceed 2.0 percent of the weight of the cement for Grade 1 or 1.6 percent of the weight of the cement for Grade 2. Grade 1 Calcium Chloride purity is between 70 and 90 percent, and Grade 2 Calcium Chloride is between 91 and 100 percent.

It should be noted that for the concrete of Class PR3, Calcium Chloride shall be thoroughly mixed into the concrete before placement and the minimum mixing time is two minutes. For Class PR1 Concrete, Calcium Chloride shall be added first, and then the concrete mixed at least two minutes or as required by the manufacturer. Next, the Type F high range water-reducer admixture is added, and the concrete is mixed an additional five minutes. Figure 2-2 presents a typical process of chemical admixtures being measured and introduced into the ready-mixed truck during a PR mix preparation at the job site.

Figure 2-2. Typical process of chemical admixtures being prepared and introduced into a mixing truck at the job site

2. 5. 2 Common Rapid Patching Methods in Other DOTs

California Department of Transportation (Caltrans) uses three different mixture designs for rapid strength concrete (RSC). The first RSC mix contains specialty or proprietary cement mixtures that meet the opening strength requirements for traffic within 2 to 4 hours. The second RSC uses Type III cement with a non-Chloride accelerator that can meet opening strength requirements within 4 to 6 hours. The third mixture uses a Type III cement with a lower dosage of a non-Chloride accelerator that can meet opening requirements within 12 to 24 hours. Caltrans' opening to traffic strength requirement is 2,000psi compressive strength and 500psi flexural strength respectively, regardless of the different traffic opening times of the three different types of RSC (Zayed et al. 2018). Florida Department of Transportation specifies a 3,000psi compressive strength within their opening to traffic at 8 hours. The cement amount varies between 840 to 900 lb/yd^3, and both Chloride and non-Chloride accelerators are allowed (Cramer et al. 2017). Kansas Department of Transportation (KDOT) specification indicates that the most common patching method employed in the state is to reach the minimum cementitious content of 650 lb/yd^3 with Calcium Chloride. The required compressive and flexural strength in KDOT is 1,800 psi and 360 psi within 4 to 6 hours (Porras 2018). There are DOTs that allow usage of proprietary blends. For example, Missouri Department of Transportation uses rapid set concrete patch materials which commercially named Western Materials product (MO) and CTS product (CO) (Darter 2017). A summary of the cement type and amount specified by different agencies is shown in Table 2-3.

Table 2-3. Cement type and amount specified in different DOTs

State	Cement type	Amount of cement [lb/yd^3]
California	Proprietary Cement Mixtures	N/A
	Type III Portland Cement	
Colorado	Type I / III Portland Cement	Minimum 660
Florida	Type II Portland Cement	800-950
Illinois	Calcium Aluminate Cement	Minimum 675
	Rapid Hardening Cement	600-625
Kansas	Type I/III Portland Cement	Minimum 658
Maryland	Type III Portland Cement	870-915
Nebraska	Type I/II Portland Cement	Minimum 752
	Type III Portland Cement	Minimum 799
New Jersey	Type I/II Portland Cement	Minimum 799
North Carolina	Type I/II Portland Cement	Minimum 754
	Regulated Set Portland Cement	Minimum 612
	Rapid Setting Cement	Minimum 651
Washington	Type I/II Portland Cement	Minimum 705
	Type III Portland Cement	Minimum 750
Wisconsin	Type I/III Portland Cement	840-900

According to a survey conducted by the research team, as shown in Figure 2-3, a few states including Iowa, Indiana, Ohio as well as Nebraska, permit the contractors to use Calcium Chloride in their design. Most of the states such as Texas, California, Michigan, Minnesota, Washington, and Orgon do not allow to use it, mostly because of its corrosive characteristics.

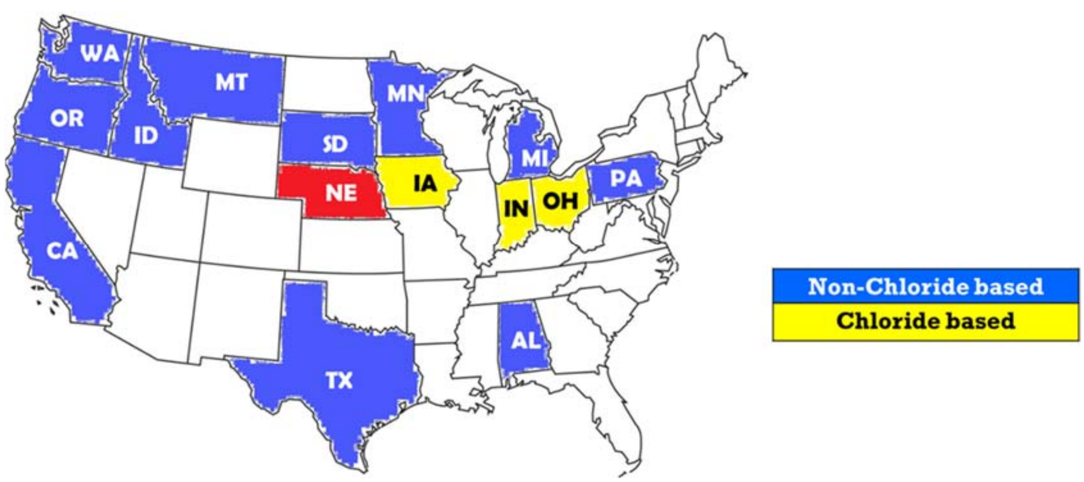


Figure 2-3. Employment of Chloride or Non-Chloride accelerators in different states

2. 5. 3 Performance Requirements by DOTs

Each DOT has its own performance requirement because of diverse reasons such as average traffic load or ambient temperature. Generally, DOTs necessitate minimum compressive strength per each distinguished traffic opening requirement. For example, Florida Department of Transportation (DOT) specifies a minimum compressive strength of 2,200 and 3,000 psi for 6 and 24 hours, respectively. A survey summary of required concrete properties and opening to traffic allowance specified per each DOT can be found in Table 2-4.

Table 2-4. Slump, Air Content, Compressive Strength and Traffic Opening Details from different DOTs

State	Opening to traffic [h]	Minimum compressive strength [psi]	Slump [in]	Air content [%]
California	2-4	2000	N/A	N/A
Florida	6	2200	1.5-4	1-6
Georgia	4	2000	N/A	N/A
Illinois	4-8	3200	2-8	4-6
Iowa	10	3500	4	6
Maryland	4	2000-2500	N/A	N/A
Minnesota	N/A	3000	N/A	N/A
Missouri	N/A	2000	N/A	N/A
Nebraska	4-8	3000	N/A	6-8.5
North Carolina	4-6	N/A	N/A	6
Utah	48	4000	N/A	N/A
Washington	N/A	2500	N/A	N/A
Wisconsin	8	3000	N/A	5-7

2. 6 Low-Temperature Performance

Achieving high-early-strength could be affected by different parameters that delay or accelerate the strength gain. Low ambient temperature is one of those. Low temperature retards the hydration reactions and significantly could disrupt the early strength gain. The 40°F (4.44°C) is defined as the threshold of freezing-temperature by ACI 306R-16 (ACI 2016) and it is the minimum temperature for hydration reactions. If the temperature goes under the 40°F (4.44°C), additional measures should be employed to increase the hydration rate and concrete attains a strength of 500psi before freezing (Karagöl et al. 2019). For a concrete patch that is exposed (prior to loading), there is a requirement of protection until achieving 500 psi strength. Also, concrete should not be allowed to freeze prior to achieving 3,500 psi (ACI 2016).

In extreme weather conditions, if a regular concrete mix design is employed, there would be various modes of protection for low-temperature concreting operations such as applying plastic sheets, insulation blankets, electric heating pad, and temporary shelters, etc. However, the surcharges might be remarkable in comparison with ordinary concrete placement. Moreover, modifying the mixture design introducing an anti-freeze admixture can be helpful. A number of researchers have studied the application of different anti-freeze admixtures in low-temperature concrete operation (Karagöl et al. 2019; Korhonen 2006; Morrical and MacDonald 2010; Polat 2016).

Generally, higher heat of hydration causes higher concrete temperature, which helps to achieve the desired curing temperature (Kosmatka et al. 2002). For this reason, materials or methods which provide higher heat of hydration are favorable to accommodate such as using higher cement content or Type III Portland cement. However, this remains a controversial issue, as the number of permeable voids could be increased, which will impair the concrete microstructure in freeze/thaw cycles.

2. 7 Rapid Patching Materials Improvement

Rather than special construction materials, to reach desired early-age strength in concrete mixtures, the most common approach is to use a high cement content. However, this remains a controversial issue, as the high cement content in concrete can also cause a higher degree of shrinkage and cracking, which could lead to high maintenance costs. Porras (2018) claimed that high cement content would result in high permeability, which was confirmed by Ghafoori et al. (2017) as it was observed that high cement content resulted in higher absorption and volume of permeable voids in 24 hours and 28 days for Type III cement-based concrete. In addition, increasing cement content resulted in higher drying shrinkage rates and lower frost resistance (Ghafoori et al. 2017).

Moreover, the reduction of cement content can also be beneficial to reduce other adverse effects (high permeability, shrinkage, and cracking) without compromising key properties such as strength. Previous studies (Wassermann et al. 2009; Yurdakul 2010) have shown that there is a positive effect of reduced cement content on chloride penetration resistance, total porosity, and capillary absorption. It is postulated in such a way due to the more porous nature of the cement paste compared to the aggregates. Thus, when cement content is reduced, and the volume fraction of aggregates to paste is increased, less permeable concrete can be achieved. In terms of the strength, previous research also showed that compressive strength seems to be independent of the cement content, once the minimum amount of paste is provided to fill the voids and bind aggregates (Wassermann et al. 2009; Yurdakul 2010; Mamirov 2019).

Optimizing the particle packing of the aggregate matrix is the key approach to reduce cement content in the mixture as the aggregate skeleton occupies approximately 70-80% of the concrete mixture by volume. Particle packing optimization targets to achieve the lowest porosity within the aggregate skeleton. According to De Larrard (1999)'s study, particle packing is mainly affected by three factors: particle size distribution, particle geometry, and the compaction method. Due to the focus of this study, particle packing optimization focused mostly on optimizing aggregate gradation. Due to the focus of this study, particle packing optimization focused mostly on optimizing aggregate gradation. Figure 2-4 demonstrates the mechanism of reducing cement content when aggregate gradation is better optimized. With the amount of the void within the aggregate skeleton reduced, the required amount of cement paste to fill those voids is reduced accordingly.

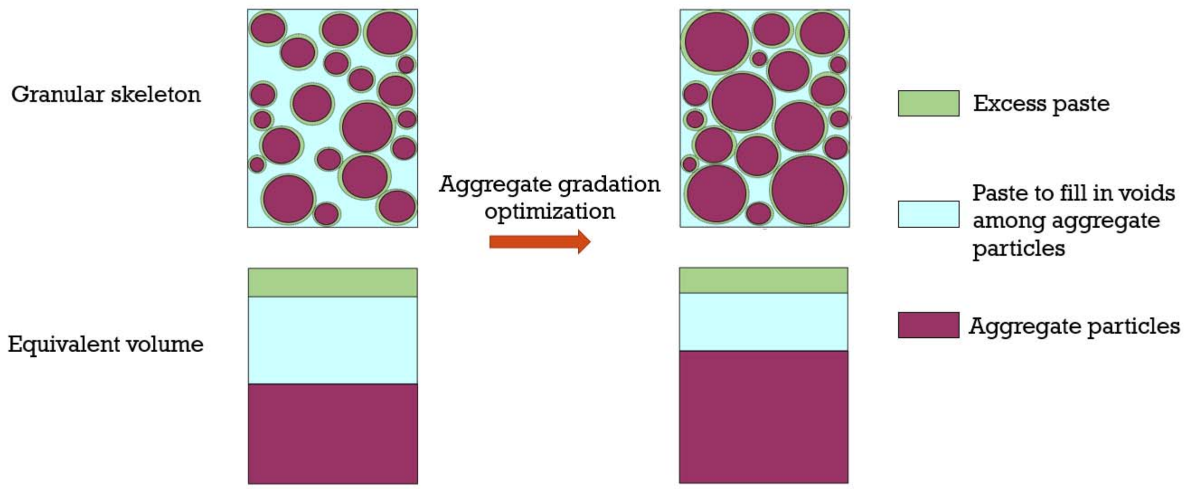


Figure 2-4. Illustration of cement content reduction with optimized aggregate gradation.

2. 8 Summary

Based on the literature survey, there is a clear need to further investigate the combined effects of different accelerators with different types of cement for a more tailored rapid patching concrete mixture in the state of Nebraska. Also, because chloride-based accelerators have adverse effects on concrete durability, non-chloride-based accelerators should be evaluated for specific mixture designs. The overall performance of developed mixtures should be assessed through a comprehensive evaluation of constructability, mechanical behavior, and long-term performance evaluation to diminish durability issues (such as ASR, and shrinkage, etc.) and reduce maintenance costs. Finally, low ambient temperature effects on patching materials' early age properties should also be evaluated. All the above-mentioned performance testing could help to achieve rapid patching materials with higher sustainability and lower long-term costs.

CHAPTER 3 EXPERIMENTAL PROGRAM

3. 1 Introduction

This chapter presents selected raw materials, mixture designs, and testing methods that were employed in different stages or phases of the project.

3. 2 Material

3. 2. 1 Cement and Cementitious materials

Types I, III, and IP Portland cement with Blaine fineness of 420, 680, and 440 m^2/kg were used respectively. Based on Nebraska DOT specification, Type I and III Portland cements are applicable in patches mixture designs. Additionally, as type IP cement can mitigate durability issues such as ASR, to study the effect of partial replacement of Type I and III cements with IP cement, a Type IP Portland-Pozzolan cement with 25% blended class F fly ash content also included in the study.

3. 2. 2 Aggregate

Crushed limestone (LS) with a nominal maximum aggregate size of 1 in. (25 mm) was used as coarse aggregate. Sand and gravel (SG) combination with a fineness modulus of 3.78 was used as fine aggregate.

3. 2. 3 Chemical Admixtures

Daraccel and Daraset 400, both commercially available in the United States and satisfy ASTM C494 Type E and C specification, were selected as Chloride (CL) and one non-Chloride (NCL) accelerator, respectively. Plastol 6200 EXT was used as a high-range water reducer (HRWR), and MasterAir AE 200 and MasterAir AE 90 was employed as the air-entraining agent (AEA).

3. 3 Mixing Procedures

The mixing procedure followed the ASTM C192 (Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory) (ASTM 2018). In summary, coarse aggregates, approximately half of the mixing water with AEA were added to the mixer. Then, after 30 seconds of mixing, the fine aggregate and cement were added. The mixer was turned on again, and the remaining water was added incrementally. After the three minutes of mixing, the concrete was allowed to rest for another three minutes, with the opening covered with a moist towel to prevent undesirable evaporation. After the resting, accelerator (either Chloride or non-Chloride based) was added to the mixer and mixed for an extra two minutes. Note that accelerators were not added to the batch at the early stage it could cause slump loss after a quick period. The practice also complies with NDOT specification that accelerators should be added to concrete before placement and mixed for at least two minutes. Finally, HWWR was added to the batch and mixed for another five minutes.

3. 4 Testing Methods

3. 4. 1 Fresh concrete properties

Slump

As an evaluation of workability, the concrete slump test as shown in Figure 3-1 was conducted immediately after mixing in accordance with the ASTM C143 (ASTM 2015).

Figure 3-1. Slump test example

Setting time

Initial and final setting times were determined by measuring the penetration resistance test per ASTM C403. Accordingly, one 6 in.×6 in. cylinder was filled with a mortar obtained from filtering fresh concrete by a No. 4 sieve. The penetration resistance was measured at 15 to 30-minute intervals. An Acme Penetrometer as shown in Figure 3-2 was used. The initial and final set times were defined as the times to reach a penetration resistance of 500 and 4,000 psi, respectively.

Figure 3-2. Setting time testing apparatus

Air Content

As shown in Figure 3-3, the air content of freshly mixed concrete was measured per ASTM C231 (ASTM 2017). Based on the NDOT requirement, 6-8.5% air content was recommended.

Figure 3-3. Air content test setup

Unit weight

Unit weights of the fresh concrete were measured for each mixture design based on ASTM C138 (ASTM 2017), see Figure 3-4.

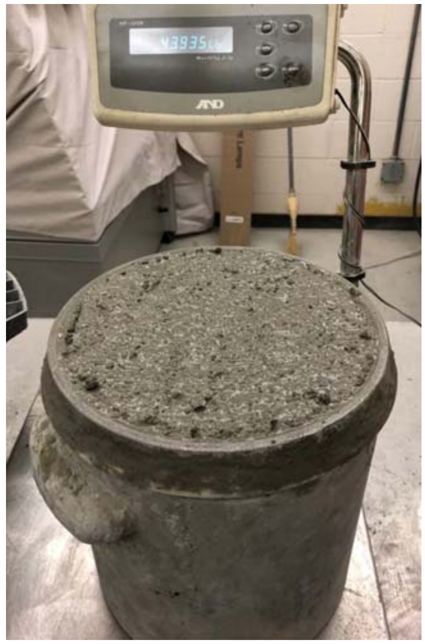


Figure 3-4. Unit weight measurement

3. 4. 2 Specimen Casting and Curing

All of the testing specimens were prepared per ASM C192 for different testing purposes. They were stored at room temperature at 73°F prior to demolding (see Figure 3-5) and then stored in a standard curing room with 100% R.H. and 75°F until testing.

Figure 3-5. Testing cylinders for compressive strength test

3. 4. 3 Early-Age Properties

The determination of concrete maturity data is important, as it can be used for concrete strength estimation at different ages for the prediction of traffic opening. Based on the ASTM

C1074 (ASTM 2019), the time-temperature factor (TTF) (or equivalent age) was monitored in the two insulated 4 in. x 8 in. cylinders. As shown in Figure 3-6, the specimens were implanted with thermocouple wires with the tips positioned at the center of specimens connected to the maturity meter instrument. Data were recorded every 30 minutes until 48 hours after batching. Estimating the compressive strength can be done by developing the relationship between the TTF and compressive strength. In this study, TTF was calculated as follows:

$$M(t) = \sum (T_a - T_o)\, \Delta t \tag{1}$$

Where M(t) is the TTF at age t (degree-days or degree-hours), Δt is the time interval (in hours), T_a is the average concrete temperature during time interval (°C), and T_o is the datum temperature (°C). The time interval and datum temperature were assumed at 0.5 hours and -10°C (14°F), respectively.

Figure 3-6. Maturity meter test setup

3. 4. 4 Mechanical Properties

Compressive Strength

Compressive strength uniaxial test in accordance with ASTM C39 (ASTM 2018) was conducted with a Universal Testing Machine (see Figure 3-7), at 6 and 12 hours, and 28 days to track strength growth. Three of 4x8 inches cylinders were tested at the specified ages and the average value was reported.

Figure 3-7. Compressive strength test setup

Modulus of Rupture

The flexural strength of concrete was measured in accordance with ASTM C78 (ASTM 2018) with casted concrete beams at a dimension of 6 in. x 6 in. x 20 in. At the desired age of the casted concrete beam, a Forney beam testing machine with a capacity of 30 kips, see Figure 3-8 was used.

Figure 3-8. Flexural Testing Apparatus

The modulus of rupture is calculated as follows:

$$R = \frac{PL}{bd^2} \tag{2}$$

Where R is the modulus of rupture in psi, P is the maximum applied load by the testing machine in lbf, L is the span length in inch, b is the average width in inch, and d is the average depth in inch.

Modulus of Elasticity

Modulus of elasticity was measured per ASTM C469 (ASTM 2014) with 4 in.x8 in. at the age of 28 days. The strain-measuring equipment was attached to specimens and then placed into the uniaxial compressive strength test setup. The modulus of elasticity measurement needs two points in the stress-strain graph. Point one stress should be read when the specimen has a strain of 50 millionths and point two strain should be read when the applied load is equal to 40 % of the ultimate load which is the maximum load in the test. The Modulus of elasticity is calculated as follows:

$$E = (S_2 - S_1)/(\varepsilon_2 - 0.000050) \tag{3}$$

Where the E is the modulus of elasticity, S_2 is the stress corresponding to 0.4 of ultimate load, S_1 is the stress corresponding to the strain of 50 millionths, and ε_2 is the strain produced by stress S_2.

Splitting Tensile Strength

The splitting tensile strength was determined per ASTM C496 (ASTM 2017). It is calculated as follows:

$$T = 2P/\pi ld \tag{4}$$

Where T is the split tensile strength in psi, P is the maximum applied load in lbf, l is the length in inch, and d is the diameter in inch.

Note that for this study, the test was used to evaluate bonds between the rapid patching materials and a standard 47B substrate concrete. Specimens at a dimension of 4in.x 8in. were cores collected from the constructed slabs. More details can be found in Chapter 5.

3. 4. 5 Durability Properties

Miniature Concrete Prism test (MCPT)

In order to evaluate the Alkali Reactivity of aggregates, the Miniature Concrete Prism test (MCPT) was conducted per AASHTO T-380 (AASHTO 2019). Since the ASR causes the expansion of the altered aggregate by the formation of an expansive gel, length change can reveal the ASR existence in samples. Three specimens per each mixture design were casted, and after 24 hours of curing the initial length was recorded. Then, the specimens were transported to a container with sufficient Sodium Hydroxide solution to be totally immersed. Measurements were taken at 3, 7, 10, 14, 21, 28, 42, and 56 days, subsequently. The length change was calculated and the results were reported.

Surface and Bulk Resistivity

Concrete cylinders at a dimension of 4 in. × 8 in. were tested for the surface and bulk resistivity test per AASHTO TP95 (AASHTO 2011). The resistivity tests can be used to estimate the permeability of hardened concrete nondestructively which indicates the concrete ability to resist chloride ion penetration. A four-point Wenner probe with a 1.5 in. probe spacing was employed to apply an alternating current through the outer pins, and the potential difference was measured between the inner pins. The procedure was carried out on statured surface dried (SSD) specimens, whereas repeated for four different directions of cylinder perimeters, and the average values were reported.

Figure 3-9. Surface and bulk resistivity test setup

Wet/Dry Cycling

The wet and dry cycling test was conducted in accordance with the NDOT specification. Three 3 in. x 4 in. x 16 in. concrete beams were casted per each mixture design and cured for 28 days. Then, they were transferred to wet/dry tank to be tested for repeated cycles of wet/dry, which includes a complete submersion of specimens in water at 70-75ºF for a period of 8 hours, followed by drying in heated air (at 120ºF) for a period of 16 hours. The specimens were evaluated every 28 days until a total of 548 days to identify any types of surface cracks (e.g. map cracking). Also, the weight change in both air and water were measured and reported.

Figure 3-10 Wet/Dry specimens after 28 days of curing

Free Shrinkage

Free shrinkage testing was conducted according to ASTM C157 (ASTM 2017). Monitoring the length changes in concrete bars reveals the tendency for volumetric expansion or contraction. Three of 3 in. x 3 in. x 11.25 in. concrete specimens were casted per each mixture design and were cured for 28 days. Then, they were moved to the environmental chamber with 73°F temperature and 50% relative humidity. The length measurement was conducted at 0, 1, 3, 7, 14, and 28 days after 28 days of curing.

Figure 3-11. Free shrinkage specimens

3. 4. 6 Cold Temperature Ambient Curing

As the concrete pavement patching is often executed during the mild temperature seasons and the traffic opening should still be short, there is a need of thorough examination of low-temperature effect on the performance of concrete patching materials with different types and contents of cement, and accelerator which still meet the technical requirement of NDOT’s rapid patching mixtures (i.e., 3,000 psi of compressive strength). As a part of the research project, the research team conducted an experimental assessment with the focus of monitoring strength development in early hours after placement with the curing at 50°F and 60°F, representing typical temperatures in the spring or fall season. Based on the early ages and mechanical properties, six of developed mixes were selected, which included three PR1 and three PR3 mixes. These mixes had satisfied the NDOT requirements and were expected to perform better in the lower temperatures.

After mixing per ASTM C192, casted 4 in. x 8 in. specimens were placed in an environmental chamber with the desired temperature and 50% relative humidity. All specimens were stored in an environmental chamber, CSZ's Z-Plus 16 cu. ft., until the testing age of 6-hr, 12hr, 24-hr and 48-hr. For 28-day strength evaluation, specimens were stored in the environmental chamber for 96 hours then transferred to a standard curing room with 75±2°F and 100% relative humidity till the date of planned testing. Maturity analysis was performed by monitoring the temperature change until 96 hours.

Figure 3-12. Environmental chamber for cold temperature simulation

CHAPTER 4 EXPERIMENTAL DESIGN AND RESULTS

4. 1 Introduction

This chapter summarizes the designs and results from the three different phases of the experimental studies. In Phase 1, after specifying the mix design parameters, various mix designs were prepared to evaluate the impact of design parameters such as cement content, and accelerator type and dosage. In Phase 2, the eight developed mixture designs with promising properties were selected to assess the long-term properties as well as additional early-age and mechanical properties. Finally, in Phase 3 study, different curing temperatures were selected to evaluate the performance of concrete in different concrete placement temperatures.

4. 2 Mixture Design Development (Phase I)

Table 4-1 shows the summary of phase 1 mix designs, which included a total of 28 mixes. The mix identification is based on four parameters, namely, cement type (PR1 or PR3), cement factor (800 to 600 lb/yd^3), aggregate blend (SG70 or SG55), accelerator types (CL or NCL) and amounts (40 or 60 fluid oz/cwt, where cwt represents a 100 lb of cement). For example, with Mix 5, the mix ID of "PR1-C600-SG55-CL60" referring to a Type I cement mix, with approximately 600 lb/yd^3 of cement, aggregate blend with SG composed of 55% of the total aggregate mass, and 60 fluid oz/cwt of the chloride-based accelerator, was used. A mix ID with an additional letter R at the end refers to mixes with a reduced w/c compared with the reference mix. The 28 mixes can be broken into seven groups. To determine the lowest amount of cement needed for the PR1 and PR3 mixes, Group A (Mixes 1 to 5) and Group C (Mixes 9 to 13) were prepared with a reduction of cement factor of approximately 50 lb/yd^3 in each step. To study the feasibility of using a non-chloride-based accelerator, Group B (Mixes 6 to 8), Group E (Mixes 18 to 20), Group F (Mixes 21 and 22) were prepared with similar cement content reduction. Another group (Group D, Mixes 14 to 17) was prepared to evaluate the effect of reduced w/c on PR3 mix performance. Finally, Group G (Mixes 23 to 28) was prepared to evaluate the contribution of Type IP cement.

Table 4-1. Phase 1 mix proportions.

Mix No.	Group	Mix ID	Cement Type	W/C	Agg. Blend	Cement	Water	LS	SG	CL	NCL	HRWR	AEA
					%	lb/yd³				fl oz/cwt			
1	A	PR1C750SG70CL60	Type I	0.31	70-30	753	204	864	2016	60	0	5	1.0
2		PR1C750SG55CL60			55-45	753	209	1296	1584			5	1.0
3		PR1C700SG55CL60			55-45	703	195	1334	1631			5	1.0
4		PR1C650SG55CL60			55-45	653	179	1373	1678			7.5	1.0
5		PR1C600SG55CL60			55-45	603	161	1415	1730			8	1.0
6	B	PR1C750SG55NCL60			55-45	753	204	1301	1590	0	60	7	1.0
7		PR1C700SG55NCL60			55-45	703	191	1337	1634			9	1.0
8		PR1C650SG55NCL60			55-45	653	179	1373	1678			9	1.0
9	C	PR3C800SG70CL60	Type III	0.41	70-30	800	296	780	1820	60	0	5	1.0
10		PR3C800SG55CL60			55-45	800	299	1170	1430			6	1.0
11		PR3C750SG55CL60			55-45	750	281	1213	1482			5.5	1.0
12		PR3C700SG55CL60			55-45	700	259	1260	1540			5.5	1.0
13		PR3C650SG55CL60			55-45	650	240	1303	1592			6	1.0
14	D	PR3C800SG70CL60R		0.36	70-30	800	258	810	1890			5	1.0
15		PR3C800SG55CL60R			55-45	800	261	1215	1485			6.5	1.0
16		PR3C750SG55CL60R			55-45	750	241	1260	1540			5.5	1.0
17		PR3C700SG55CL60R			55-45	700	224	1301	1590			6	1.0
18	E	PR3C800SG70NCL40		0.41	70-30	800	306	780	1820	0	40	6	1.0
19		PR3C800SG55NCL40			55-45	800	309	1170	1430			6.5	1.0
20		PR3C750SG55NCL40			55-45	750	288	1215	1485			7	1.0
21	F	PR3C700SG55NCL40R		0.36	55-45	700	237	1296	1584	0	40	5	1.0
22		PR3C650SG55NCL40R			55-45	650	220	1337	1634			6	1.0
23	G	PR1C753SG70CL60_25% IP	Type IP/I	0.31	70-30	176 + 565	201	2027	869	60	0	5	1.0
24		PR1C753SG70CL60_50% IP			70-30	353 + 377	200	2027	869			5	1.0
25		PR1C753SG55CL60_50% IP			55-45	353 + 377	200	1598	1307			5	1.0
26		PR1C753SG70CL60_75% IP			70-30	529 +188	198	2034	872			5	1.0
27		PR3C800SG55CL60_50% IP	Type IP/III	0.36	55-45	375 + 400	246	1507	1233			3	1.0
28		PR1C753SG70CL60_100% IP	Type IP	0.31	70-30	705	194	2041	875			5	1.0

*1 lb/yd³ = 0.593 kg/m³; 1 fl oz/cwt = 62.64 ml/100kg; W/C, LS, SG, CL, NCL, HRWR, and AEA denote water-to-cement ratio, Limestone, Sand and Gravel, Chloride-based accelerator (Darracel), Non-Chloride based accelerator (Daraset400), High Range Water Reducer (Plastol ext 6200), and Air Entraining Admixture (AE 200), respectively.

Note that Mixes 1 and 9 are the two reference mixes, as specified by NDOT (2017). To achieve sufficient early strength, a low w/c of 0.31 was used for all Type I cement (PR1) mixes. Because of the high fineness and water demand of Type III cement, a higher w/c of 0.41 was used for the Type III cement (PR3) mixes, except for Group D and F, in which a w/c of 0.36 was adopted to study the effect of different w/c on early-age strength gain and set time. To eliminate the effect of accelerator dosage, the CL and NCL accelerator dosages were fixed at 60 fluid oz/cwt (3,750 ml/100 kg of cement) (Group A, B, C, D, G) and 40 fluid oz/cwt (2,500 ml/100 kg of cement) (Group E and F), respectively, as per the recommended dosages from the producer. A higher dosage of NCL had to be used in PR1 mixes (Group 2) to achieve sufficient early-age strength. In phase 1, the amount of air content was not taken into account, so the AEA dosage was fixed at 1 fluid oz/cwt of AEA200. The HRWR dosage was adjusted in each mix to obtain mixes with appropriate workability.

Unit weight, slump, setting times, compressive strength, and maturity analysis were conducted on different mixture designs to identify the most proposing designs.

4. 2. 1 Fresh and Early-Age Concrete Properties

Fresh and early-age concrete properties can reveal important information per each mixture design. Concrete needs to have appropriate workability to ensure appropriate placement. Due to the high cement content and accelerator usage, rapid patching materials tend to have much shorter setting times (less than 1 hour in some cases) in comparison with regular concrete mixes (such as 47B).

As shown in Tables 4-2 to 4-4, slump values were found to be between 1.75 in. and 5.5 in. Unit weights were found to be between 145 and 154 lb/yd3. No clear trend is observed regarding the impact of cement type, content, and accelerator types on slump and unit weight of concrete.

Table 4-2. Fresh concrete properties of phase 1 mixes (PR1 mixes, Groups A & B)

Mix No.	Mix ID	Unit weight (pcf)	Slump (in)	Initial Setting (min)	Final Setting (min)
1	PR1C750SG70CL60	147.6	3	115	167
2	PR1C750SG55CL60	151.0	2.25	109	162
3	PR1C700SG55CL60	150.9	1.5	140	181
4	PR1C650SG55CL60	153.6	1.5	146	184
5	PR1C600SG55CL60	152.7	2	129	162
6	PR1C750SG55NCL60	152.0	2	83	122
7	PR1C700SG55NCL60	152.1	1.75	92	132
8	PR1C650SG55NCL60	153.8	2	96	147

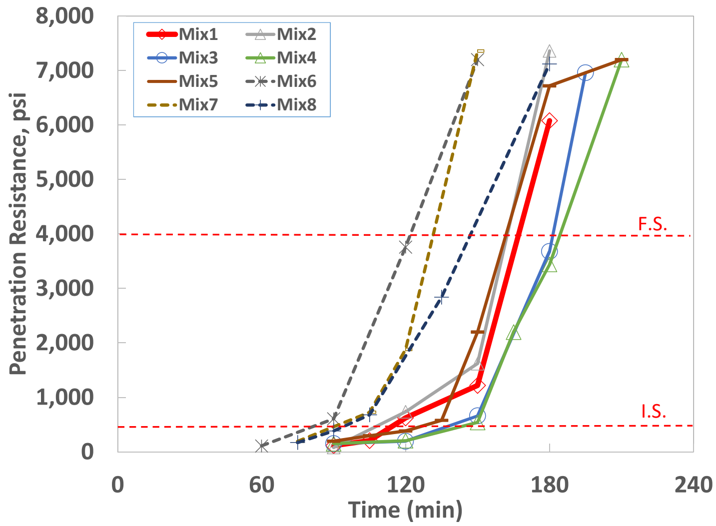
Table 4-3. Fresh concrete properties of phase 1 mixes (PR3 mixes, Groups C to F)

Mix No.	Mix ID	Unit weight (pcf)	Slump (in)	Initial Setting (min)	Final Setting (min)
9	PR3C800SG70CL60	146.9	4	175	212
10	PR3C800SG55CL60	147.0	3.75	170	225
11	PR3C750SG55CL60	147.0	5.5	182	224
12	PR3C700SG55CL60	149.4	3	184	229
13	PR3C650SG55CL60	152.5	2	185	233
14	PR3C800SG70CL60R	149.8	1.75	103	156
15	PR3C800SG55CL60R	149.9	2.25	98	150
16	PR3C750SG55CL60R	152.0	2	118	161
17	PR3C700SG55CL60R	152.1	2	124	164
18	PR3C800SG70NCL40	145.1	4	117	172
19	PR3C800SG55NCL40	148.1	3.5	106	161
20	PR3C750SG55NCL40	149.3	2.25	107	158
21	PR3C700SG55NCL40R	150.4	1.75	91	136
22	PR3C650SG55NCL40R	151.8	2.25	93	139

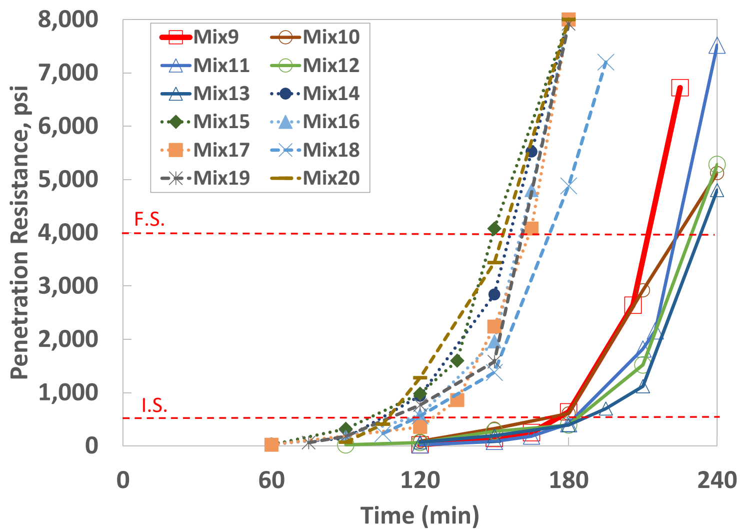
Table 4-4. Fresh concrete properties of phase 1 mixes (IP cement mixes, Groups G)

Mix No.	Mix ID	Unit weight (pcf)	Slump (in)	Initial Setting (min)	Final Setting (min)
23	PR1C753SG70CL60_25% IP	151.1	3.5	118	167
24	PR1C753SG70CL60_50% IP	148.3	2.75	119	169
25	PR1C753SG55CL60_50% IP	150.7	2	113	163
26	PR1C753SG70CL60_75% IP	150.0	2.25	121	167
27	PR3C800SG55CL60_50% IP	148.9	4.75	125	164
28	PR1C753SG70CL60_100% IP	149.9	2.25	122	170

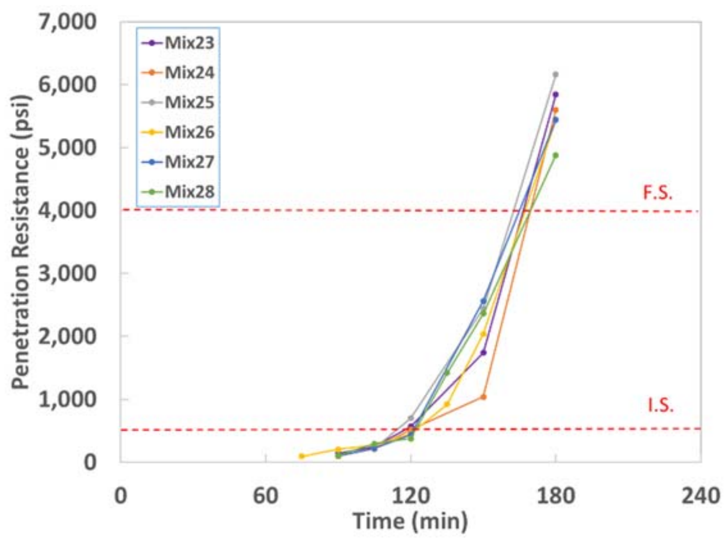
Setting times were determined by the penetration resistance test, per ASTM C403. Initial and final setting are times to reach a penetration resistance of 500 and 4,000 psi, respectively, which are identified as initial set (I.S.) and final set (F.S.) in Figure 4-1. As shown in Figure 4-1, with similar cement content, the non-chloride accelerator selected in the study resulted in a higher penetration resistance at the same age, compared to the chloride-based accelerator mixes. Also, as expected, the reduced w/c also leads to a higher penetration resistance at the same age.



(a) PR1 mixes, Groups A & B



(b) PR3 mixes, Groups C to F



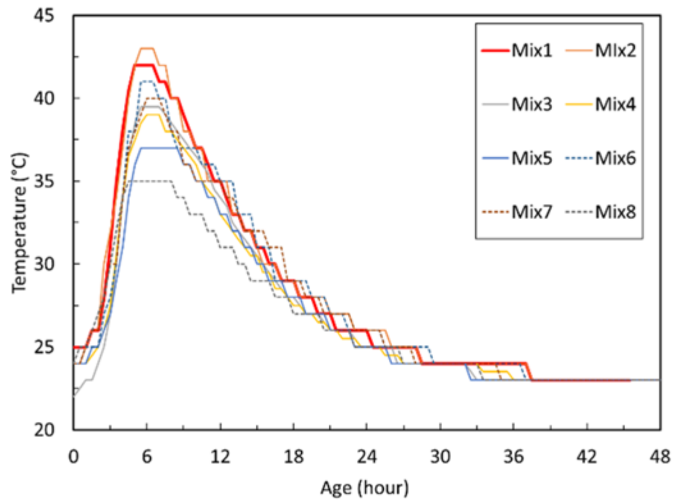
(c) IP cement mixes, Groups G

Figure 4-1. Results from penetration resistance tests for setting times

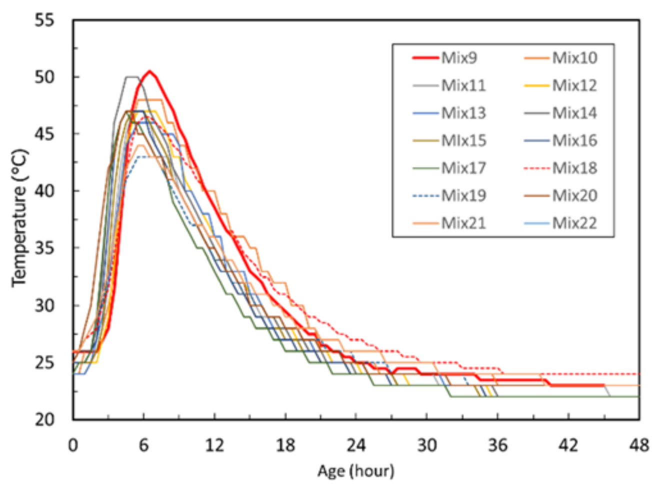
As can be seen in Figure 4-1, the initial setting time was between 83 and 185 minutes, and the final set times were between 122 and 233 minutes. Note that the windows between the initial and final sets were found to be relatively small compared with conventional concrete.

4. 2. 2 Temperature Evolution

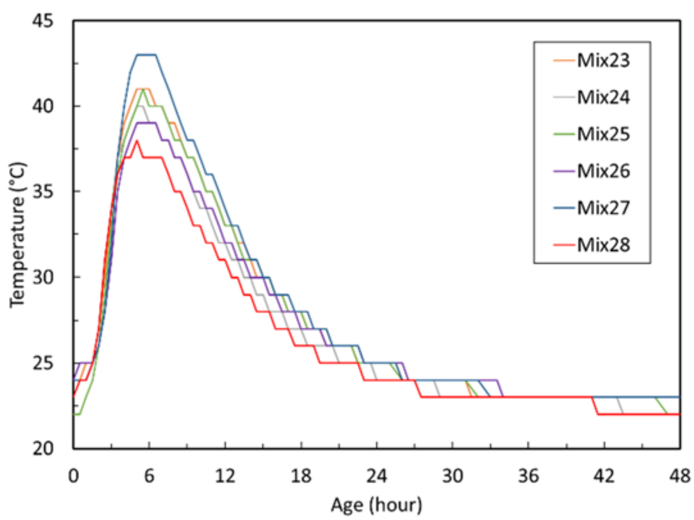
Results from the temperature development based on the maturity tests are shown in Figure 4-2. The area beneath the diagram indicates the heat of hydration during a certain period. As expected, it was observed that a lower amount of heat is generated with PR1 mixes and with the reduction of cement content.



(a) PR1 mixes, Groups A & B



(b) PR3 mixes, Groups C to F



(c) IP cement mixes, Groups G

Figure 4-2. Time-Temperature development

4. 2. 3 Mechanical Properties

In phase 1 of experimental studies, the compressive strength of all mixes evaluated to identify appropriate mixes which satisfy the NDOT strength requirement. The compressive strength all Phase I mixes in all the seven groups included in the study was evaluated at 6 and 12 hours and 28 days, and the results are presented in Figure 4-3. The recommended opening to traffic strength criteria at 3,000 psi and 3,500 psi are also identified in each of the graphs.

As shown in the figure, most mixes included in the study reached approximately 2,000 to 3,000 psi at 6 hours and 4,000 to 6,000 psi at 12 hours. As expected, significant strength gains were observed after 12 hours, and the strength at 28 days was all above 8,000 psi, which is higher than conventional concrete. The high cement content used in the rapid patching mixes likely contributed to the high strength.

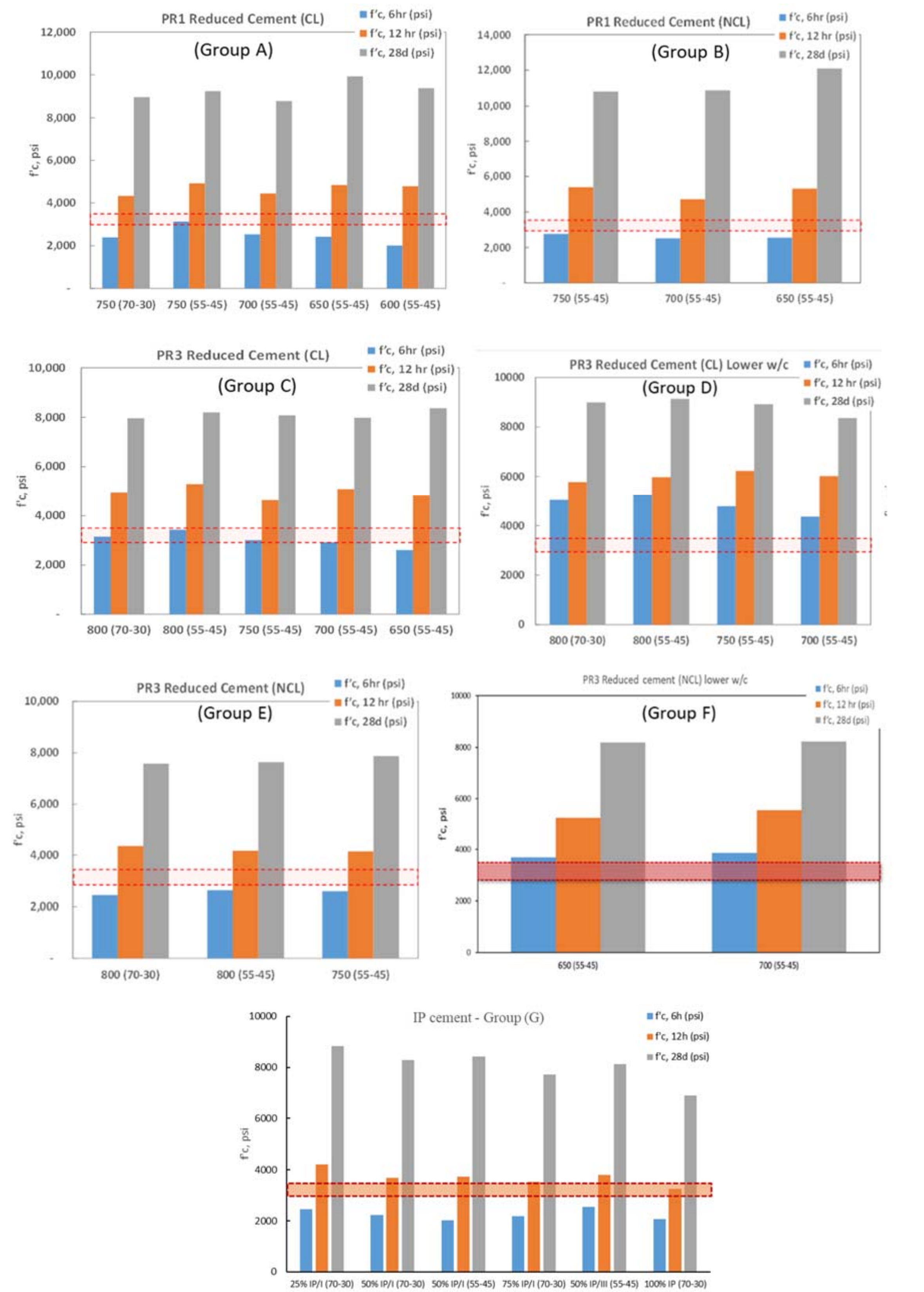


Figure 4-3. Comparison of compressive strength at different ages for phase i mixes

4. 2. 4 Maturity Analysis

The relationship between the calculated TTF and compressive strength was used to estimate the time for concrete to reach the recommended traffic opening strength of 3,000 and 3,500 psi. The best-fit curves were determined by the regression analysis for ages between the 6 and 12 hours, and the results are shown in Figure 4-4.

While there is no clear trend observed in regard to the impact of cement type, content, and the type of accelerators, the results were found to be consistent with those shown in Figure 4-4, i.e., that all mixes reached 3,000 psi of compression within 7 hours and 3,500 psi within 9 hours. Note that, as Group D and F mixes reached 3,500 psi before 6 hours, and as there is no strength data prior to 6 hours, the estimated times were all less than 6 hours and marked at 6 hours for both 3,000 psi and 3,500 psi in Figure 4-4.

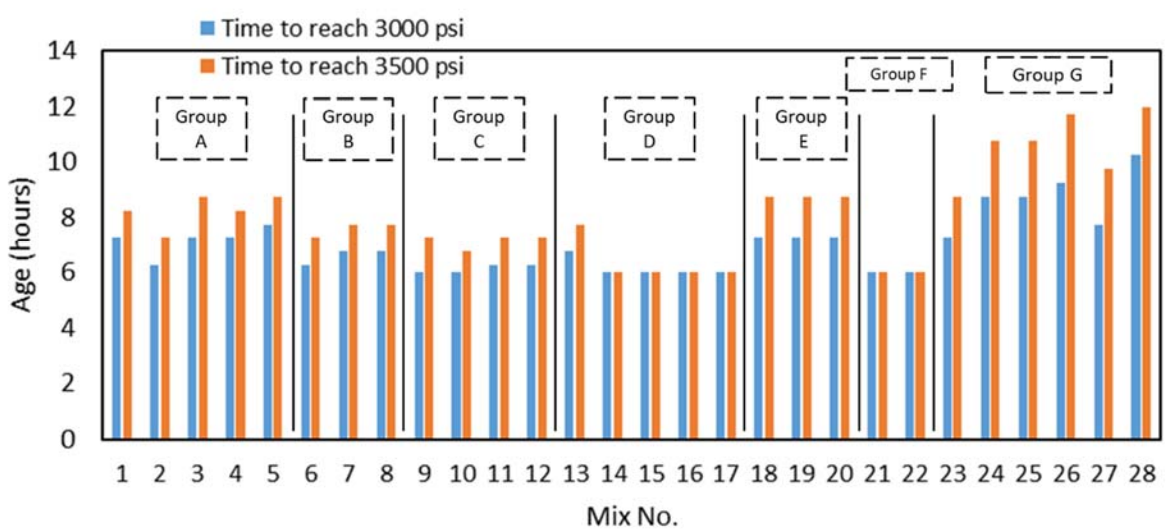


Figure 4-4. Estimated time for concrete to reach 3,000psi and 3,5000psi.

4. 3 Performance Evaluation (Phase 2)

Based on the results of mixture design development (phase 1), eight promising mixes were selected to conduct the additional fresh, early-age, mechanical, and durability tests. The selected mixes showed promising properties and fulfilled NDOT strength requirements. The summary of phase 2 mix designs can be seen in Table 4-5. Similar to phase 1, the mix identification is based on four parameters, namely, cement type (PR1 or PR3), cement factor (800 to 600 lb/yd^3), aggregate blend (SG70 or SG55), and accelerator types (CL or NCL) and amounts (40 or 60 fluid oz/cwt, where cwt represents a 100 lb). In this phase of experiments, the air content of mixtures was adjusted to the NDOT requirement (6-8.5%). MasterAir AE90 (as an air-entraining admixture) was used to reach desired air content. The HRWR dosage was adjusted in each mix to obtain mixes with appropriate workability. Plastol Ext 6200 was used as the HRWR like phase 1 of experiments.

Eight mixes can be broken into two groups of PR1 and PR3 based mixes. In each group, the first mix indicates the control mix design which has the regular amount of cement content, aggregate gradation and chloride accelerator similar to NDOT specification. The second mix employs a 100 lb. lower cement content and optimized aggregate gradation. The third mix is similar to the second mix but with non-chloride-based accelerator. Finally, in the fourth mix, the effect of Type IP cement was evaluated. 25% and 50% replacement of Type IP cement were selected in PR1 and PR3 mixes, respectively. Mechanical, fresh, and early-age testing for this phase of experiments include Slump (ASTM C143), Unit Weight (ASTM C138), Pressure Air Test (ASTM C231), Modulus of Elasticity (ASTM C469), Modulus of Rupture (ASTM C78), and Heat of Hydration (ASTM C1702). Durability testing matrix comprises Miniature Concrete Prism Test (AASHTO T380), Freeze and Thaw Cycling (ASTM C666), Wet and Dry cycling (NDOT specification), Free Shrinkage (ASTM C157), Restrained Shrinkage (ASTM C878), and surface and bulk resistivity tests (AASHTO TP95).

Table 4-5. Mix design for performance evaluation phase (phase 2)

Mix No.	Mix ID	Cement Type	W/C	Agg. Blend	Cement	Water	LS	SG	CL	NCL	HRWR	AEA
				%	lb/yd^3				fl oz/cwt			
1	PR1C750SG70CL60	Type I	0.31	70-30	753	204	864	2016	60	0	5	5.5
2	PR1C650SG55CL60			55-45	653	179	1373	1678			5.5	6.0
3	PR1C650SG55NCL60			55-45	653	179	1373	1678			6	6.0
4	PR1C753SG70CL60_25% IP	Type IP/I	0.31	70-30	176 + 565	201	2027	869			6	6.0
5	PR3C800SG70CL60R	Type III	0.36	70-30	800	258	810	1890			5	6.0
6	PR3C700SG55CL60R			55-45	700	224	1301	1590			5.5	6.5
7	PR3C700SG55NCL40R			55-45	700	237	1296	1584	0	40	5.5	6.5
8	PR3C800SG55CL60_50% IP	Type IP/III	0.36	55-45	375 + 400	246	1507	1233			5.5	6.5

*1 lb/yd3 = 0.593 kg/m3; 1 fl oz/cwt = 62.64 ml/100kg; W/C, LS, SG, CL, NCL, HRWR, and AEA denote water-to-cement ratio, Limestone, Sand and Gravel, Chloride-based accelerator (Darracel), Non-Chloride based accelerator (Daraset400), High Range Water Reducer (Plastol Ext 6200), and Air Entraining Admixture (AE 90), respectively.

4. 3. 1 Fresh and Early-Age Concrete Properties in Phase 2

The summary of the slump, unit weight, and air content results are shown in Table 4-6. The slump values were kept between 4-6 inches. Also, the air content percentage was adjusted between 6-8.5%.

Table 4-6. Fresh concrete properties for phase 2 mixes

Mix ID		Slump (in)	Unit weight (pcf)	Air content (%)
PR1-1	**PR1C750SG70CL60**	4.25	150.5	6.2
PR1-2	**PR1C650SG55CL60**	6.00	145.4	6.8
PR1-3	**PR1C650SG55NCL60**	5.25	147.2	7.0
PR1-4	**PR1C753SG70CL60_25% IP**	4.75	149.7	6.4
PR3-1	**PR3C800SG70CL60R**	5.25	145.0	6.0
PR3-2	**PR3C700SG55CL60R**	5.25	145.7	6.6
PR3-3	**PR3C700SG55NCL40R**	5.50	145.2	7.2
PR3-4	**PR3C800SG55CL60_50% IP**	6.00	144.8	8.0

In this phase of experiments, the heat of hydration was measured per each specimen by using an Isothermal Conduction Calorimetry for 48 hours. The rate of heat evolution and heat of hydration were calculated and shown in Figures 4-5 and 4-6. PR3 mixes displayed higher heat of hydration than PR1 mixes.

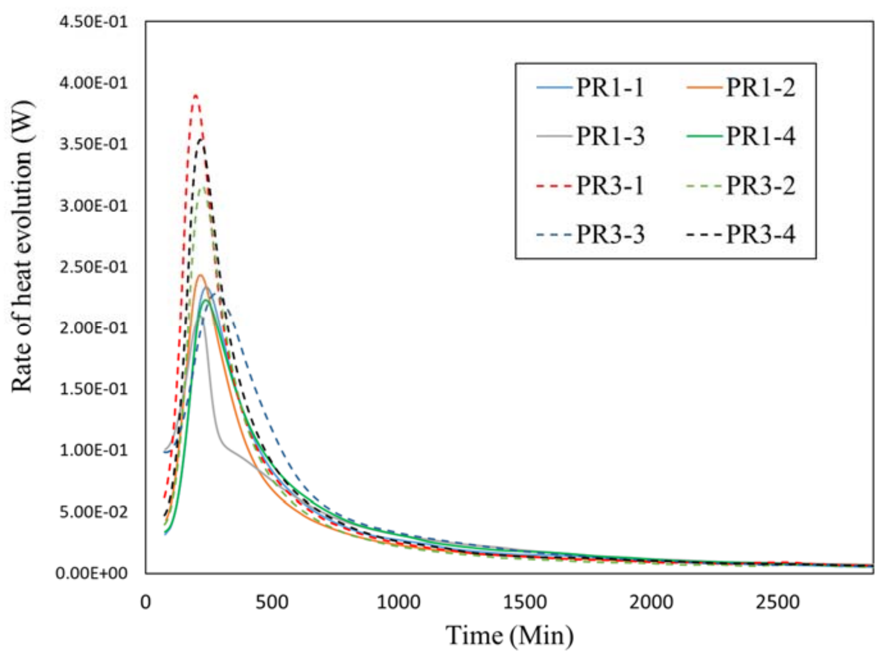


Figure 4-5. Rate of heat evolution for Phase 2 mixes.

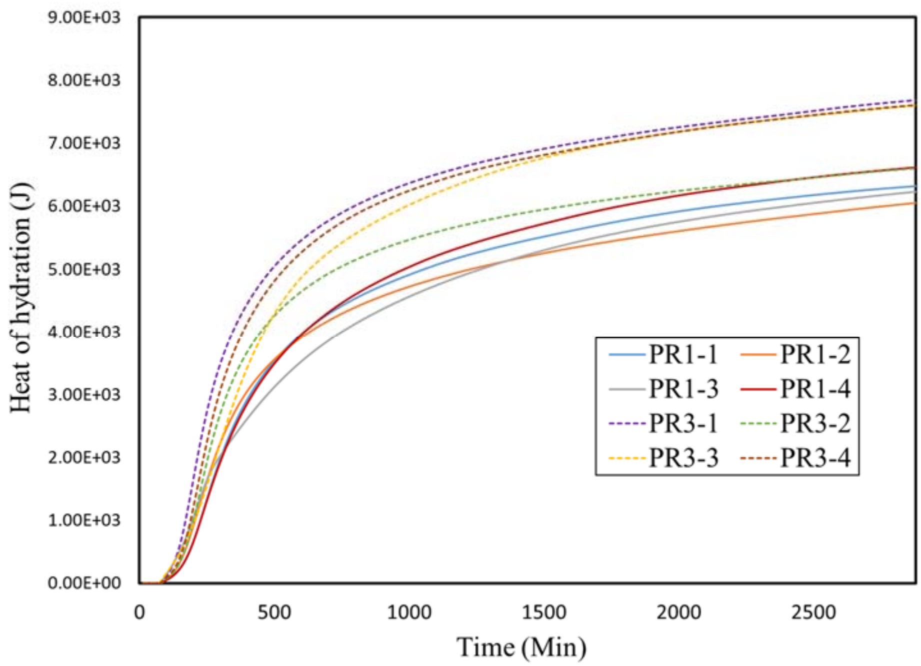


Figure 4-6. Heat of hydration for Phase 2 mixes.

4. 3. 2 Mechanical Properties

In this phase of experiments, modulus of elasticity and modulus of rupture were measured at the age of 28 days. Table 4-7 summarizes the mechanical properties of developed mixes.

Table 4-7. Mechanical properties of phase 2 mixes

Mix ID		Compressive Strength (psi)	Modulus of Elasticity (ksi)	Modulus of Rupture (psi)
PR1-1	**PR1C750SG70CL60**	8055	2335	740
PR1-2	**PR1C650SG55CL60**	8120	2240	720
PR1-3	**PR1C650SG55NCL60**	8905	3220	760
PR1-4	**PR1C753SG70CL60_25% IP**	8430	2350	590
PR3-1	**PR3C800SG70CL60R**	8895	2865	700
PR3-2	**PR3C700SG55CL60R**	8675	2545	610
PR3-3	**PR3C700SG55NCL40R**	8350	3010	725
PR3-4	**PR3C800SG55CL60_50% IP**	8110	2270	545

Modulus of Elasticity

The elastic moduli of the eight developed mixes are compared in Figure 4-7. It can be seen that non-chloride-based mixes (PR1-3 and PR3-3) display higher elastic modulus than the chloride-based mixes. Elastic modulus in mixes with 50 lb. less of cement content and optimized aggregate gradation slightly dropped compared to the reference mixes.

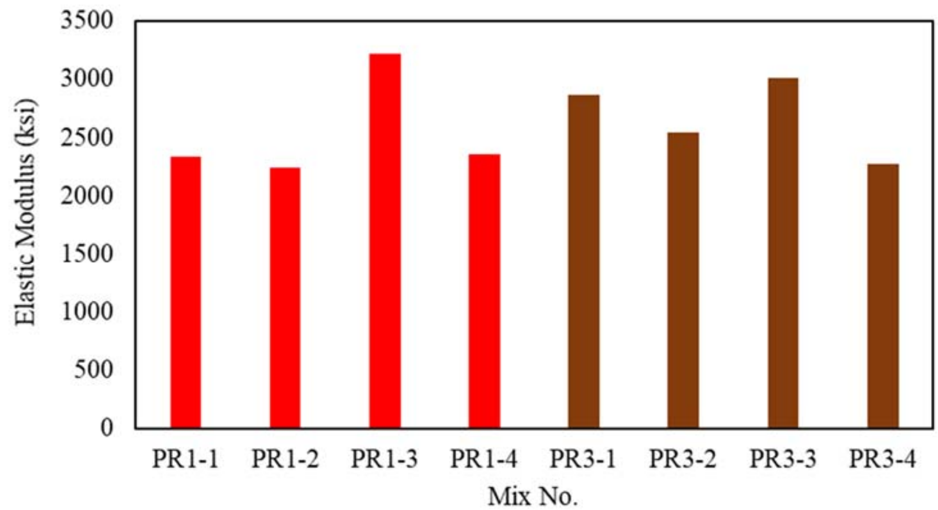


Figure 4-7. Modulus of Elasticity results for Phase 2 mixes

Modulus of Rupture

The moduli of rupture of the eight developed mixes are compared in Figure 4-8. It can be illustrated that the Type IP-based mixes (PR1-4 and PR3-4) show lower modulus of rupture in comparison with Type I- and Type III-based mixes. Also, the application of Calcium Nitrate improved the flexural strength of samples.

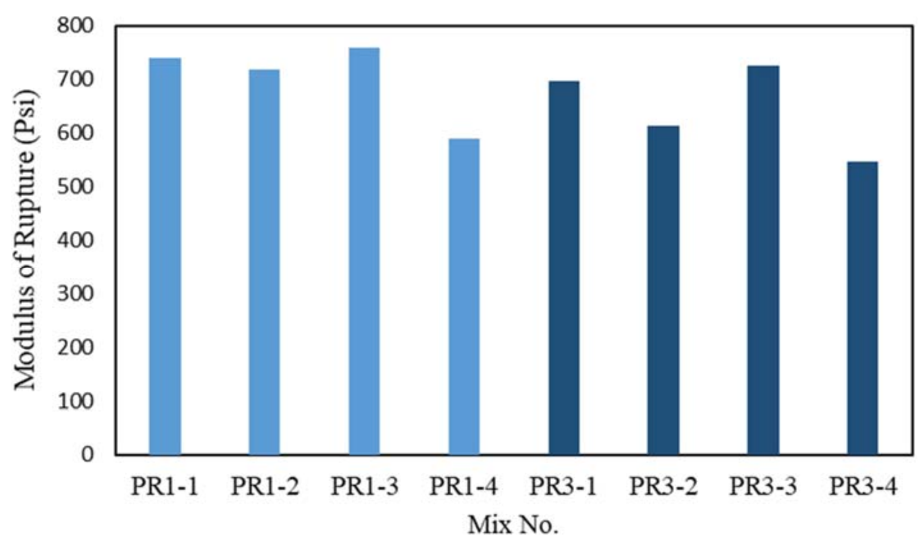


Figure 4-8. Modulus of Rupture results for phase 2 mixes

4. 3. 3 Alkali–Silica Reaction (ASR)

The ASR could be mitigated by employing supplementary cementitious materials (SCMs), such as fly ash. As a result, the Miniature Concrete Prism test (MCPT) was conducted on PR1-4 and PR3-4 mixes which consisted of Type IP cement with replacement percentage of 25% and 50%, respectively, and were compared with control mixes (i.e. PR1-1 and PR3-1 mixes). The 25%

and 50% replacement rates were determined by a preliminary study, which identified the maximum amount of IP cement that can be used to ensure sufficient early age strength. The summary of length change results is shown in Table 4-8.

Table 4-8. Miniature Concrete Prism test (MCPT) results for Phase 2 mixes

		(%) of Length Change			
	Mix No.	PR1-1	PR1-4	PR3-1	PR3-4
	Cement Type	Type I	75% Type I + 25% Type IP	Type III	50% Type III + 50% Type IP
Test Age	3 Days	0.01%	0.00%	0.00%	0.00%
	7 Days	0.01%	0.01%	0.00%	0.00%
	10 Days	0.01%	0.01%	0.01%	0.00%
	14 Days	0.02%	0.01%	0.01%	0.00%
	21 Days	0.03%	0.02%	0.03%	0.01%
	28 Days	0.05%	0.04%	0.06%	0.01%
	42 Days	0.09%	0.08%	0.14%	0.02%
	56 Days	0.13%	0.12%	0.18%	0.03%

As shown in Table 4-8, while 25% replacement with IP cement was not sufficient to mitigate ASR in PR1 mixes, a 50% replacement rate of IP cement to type III cement could potentially mitigate ASR in PR3 mixes. It can be illustrated that while the mix with Type III Portland cement resulted in a higher degree of reactivity than Type I-based mix, the PR3-4 mixture with 50% Type III and 50% Type IP displayed only 0.03% expansion rate after 56 days which is considerably smaller than PR3-1 mix with 0.18% expansion rate. However, it should be noted that the expansion rate is still higher than NDOT specification, in which expansion should not be greater than 0.020% at 56 days. On the other hand, according to AASHTO T-380, the expansion rate of 0.121–0.240% is considered as highly reactive, while the expansion rate of 0.031–0.040% is low/slow reactive. The high degree of reactivity typically displays the ASR distress in the field within five years, while low/slow degree of reactivity will be exhibited the distress beyond ten years. Results indicated that the 50% replacement of type III cement with Type IP cement (PR3-4 mix design) could potentially be a solution to ASR problem in patching designs.

4. 3. 4 Surface and Bulk Resistivity

The surface and bulk resistivity results are shown in Figure 4.5. Based on the AASHTO TP 95 chloride penetrability classification, chloride ion penetrability should be classified as high when surface resistivity is less than 12 kΩ*cm. As shown in the figure, all mixes were found to have surface resistivity below this mark and were classified as being highly susceptible to corrosion. The low resistivity is likely due to the high cement contents used in the rapid-patching mixes. However, the non-chloride-based mix design had the highest resistivity among the developed mixes (PR1-3 and PR3-3).

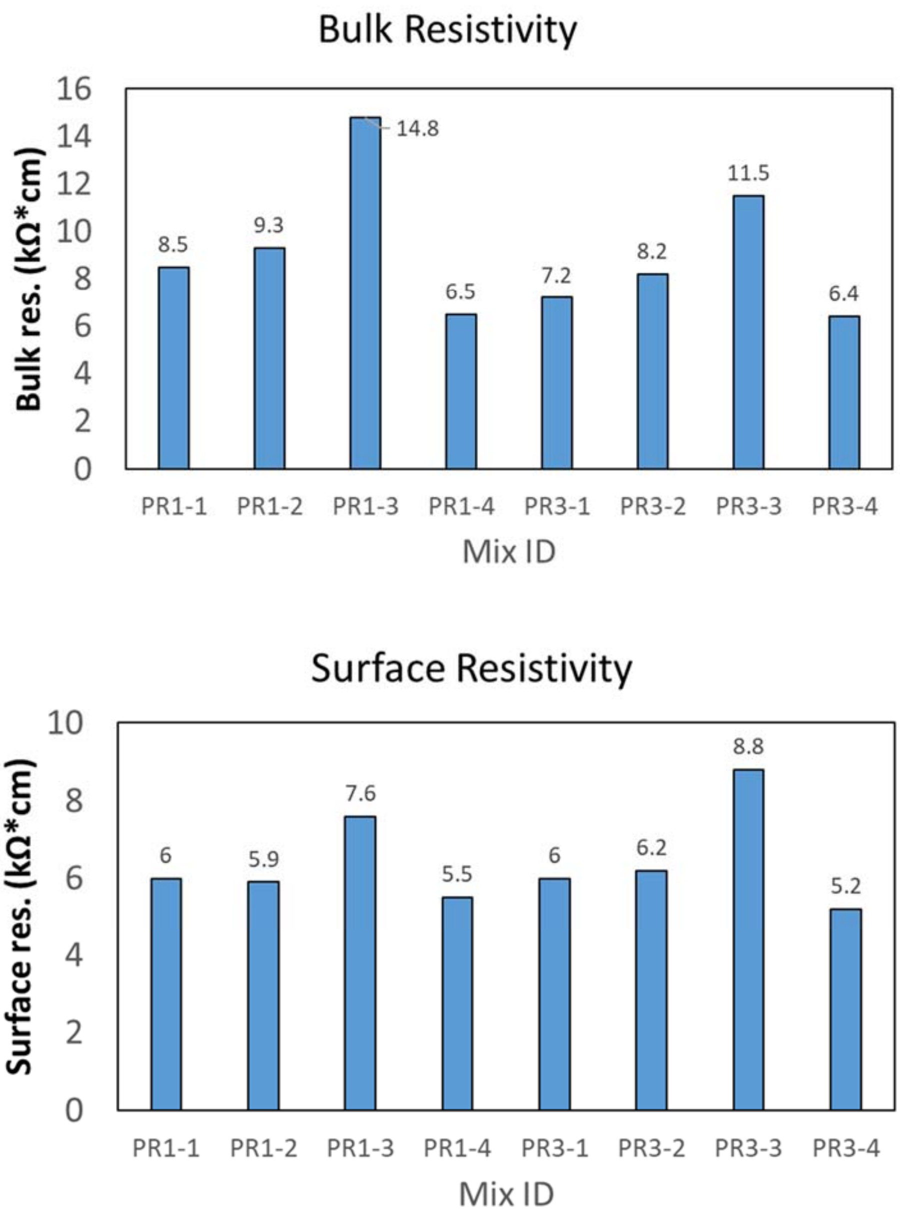


Figure 4-9. Surface and bulk resistivity results for Phase 2 mixes

4. 3. 5 Wet/Dry

Three wet/dry specimens per each mixture designs were placed into the wet/dry chamber to exam the deterioration resistance in wet/dry cycles. As the test requires a total of 548 days, this part of the experiments needs a longer time to show meaningful results and the results are not included in this report.

4. 3. 6 Free Shrinkage

The length change of three specimens per each mixture design was monitored for 28 days (after 28 days curing) to evaluate the effect of different cement types, cement content and chemical admixtures (Figure 4-8). The PR1-2 mix, which had the lowest cement content, displayed the lowest drying shrinkage. Generally, all rapid patching mixes showed higher shrinkage rates versus the regular 47B Nebraska state pavement mix which is because of the contribution of higher cement content.

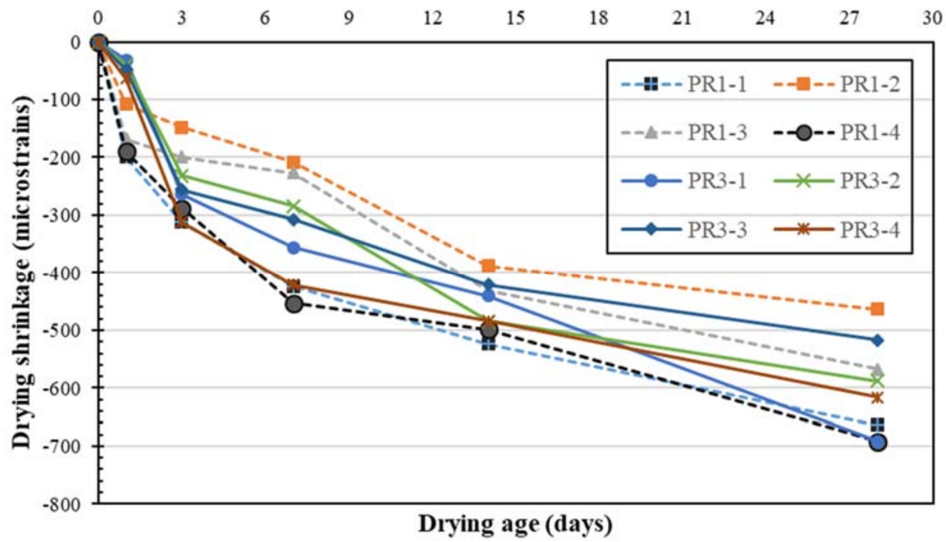


Figure 4-10. Free shrinkage results for Phase 2 mixes.

4. 4 Cold Temperature Performance (Phase 3)

As it is mentioned in Chapter 3, two lower ambient temperatures, 50°F and 60°F, were selected to evaluate the impact of curing at low temperatures on the performance of developed patching materials. Results from different experimental studies are collected in this section. The summary of phase 3 mix designs can be seen in Table 4-6. The mix identification is based on five parameters, namely, temperature (50, 60 or 75°F), cement type (PR1 or PR3), cement factor (800 to 600 lb/yd^3), aggregate blend (SG70 or SG55), and accelerator types (CL or NCL) and amounts (40 or 60 fluid oz/cwt, where cwt represents a 100 lb of cement). In Table 4-6, mixes which cured at 75°F (phase 1 experiments) are also included to compare mixes' properties clearly in the next sections; therefore mixes can be categorized in three different groups based on their curing temperature; mixes 1.1 to 1.6 represent 50°F, mixes 2.1 to 2.6 represent 60°F.

Table 4-9. Mix proportions for cold temperature performance evaluation (phase 3) mixes

Mix No.	Mix ID	Curing Temp.	Cement Type	W/C	Agg. Blend	Cement	Water	SG	LS	CL	NCL	HRWR	AEA
		°F			%	lb/yd³				fl oz/cwt			
1.1	50F_PR1C753SG70CL60	50	Type I	0.31	70-30	753	204	2016	864	60	0	5	1
1.2	50F_PR1C653SG55CL60				55-45	653	179	1678	1373	60	0	5.5	1
1.3	50F_PR1C653SG55NCL60				55-45	653	179	1678	1373	0	60	6	1
1.4	50F_PR3C800SG70CL60		Type III	0.36	70-30	800	258	1890	810	60	0	4	1
1.5	50F_PR3C700SG55CL60				55-45	700	224	1590	1301	60	0	4.5	1
1.6	50F_PR3C700SG55NCL40				55-45	700	237	1584	1296	0	40	4.5	1
2.1	60F_PR1C753SG70CL60	60	Type I	0.31	70-30	753	204	2016	864	60	0	5	1
2.2	60F_PR1C653SG55CL60				55-45	653	179	1678	1373	60	0	5.5	1
2.3	60F_PR1C653SG55NCL60				55-45	653	179	1678	1373	0	60	6	1
2.4	60F_PR3C800SG70CL60		Type III	0.36	70-30	800	258	1890	810	60	0	4	1
2.5	60F_PR3C700SG55CL60				55-45	700	224	1590	1301	60	0	4.5	1
2.6	60F_PR3C700SG55NCL40				55-45	700	237	1584	1296	0	40	4.5	1
3.1	75F_PR1C753SG70CL60	75	Type I	0.31	70-30	753	204	2016	864	60	0	5	1
3.2	75F_PR1C653SG55CL60				55-45	653	179	1678	1373	60	0	5.5	1
3.3	75F_PR1C653SG55NCL60				55-45	653	179	1678	1373	0	60	6	1
3.4	75F_PR3C800SG70CL60		Type III	0.36	70-30	800	258	1890	810	60	0	4	1
3.5	75F_PR3C700SG55CL60				55-45	700	224	1590	1301	60	0	4.5	1
3.6	75F_PR3C700SG55NCL40				55-45	700	237	1584	1296	0	40	4.5	1

*1 lb/yd3 = 0.593 kg/m3; 1 fl oz/cwt = 62.64 ml/100kg; W/C, LS, SG, CL, NCL, HRWR, and AEA denote water-to-cement ratio, Limestone, Sand and Gravel, Chloride-based accelerator (Darracel), Non-Chloride based accelerator (Daraset400), High Range Water Reducer (Plastol Ext 6200), and Air Entraining Admixture (AE 200), respectively.

4. 4. 1 Fresh Concrete Properties

The summary of slump and unit weight results are shown in Table 4-7. The slump values were between 2-3 inches. Changes in the slump and unit weight data show a reverse relationship between them. Moreover, it can be seen that there has been a slight increase in unit weight after the reduction of cement and using a higher proportion of limestone than sand and gravel.

Table 4-10. Fresh concrete properties of Phase 3 mixesat different temperatures.

Mix ID	Mix No.		Slump (in)		Unit weight (pcf)	
	50°F	60°F	50°F	60°F	50°F	60°F
PR1C753SG70CL60	**1.1**	**2.1**	2.25	2	149.5	150.1
PR1C653SG55CL60	**1.2**	**2.2**	2.5	2.25	150.8	150.6
PR1C653SG55NCL60	**1.3**	**2.3**	2.25	2	150.6	150.2
PR3C800SG70CL60	**1.4**	**2.4**	2.25	2.75	148.2	147.3
PR3C700SG55CL60	**1.5**	**2.5**	2.25	2.25	150.5	150.8
PR3C700SG55NCL40	**1.6**	**2.6**	2	2	149.8	150.0

4. 4. 2 Temperature Evolution

In order to understand how the hydration reactions delay due to low ambient temperature, specimens' temperatures were monitored. As shown in Figure 4-5, the temperature evolution of each mix design was monitored for 96 hours. The temperature record of each mix can be used to compare the mixes' peak temperature value and reaching time to it or heat of hydration (the area beneath each graph). The initial temperature of all mixes was the lab temperature (24±2°c). Moreover, Figure 4-5 shows how the heat of hydration diminished in each specific ambient temperature. However, the time to reach the peak temperature in all mixes was around 8 hours for all the cases.

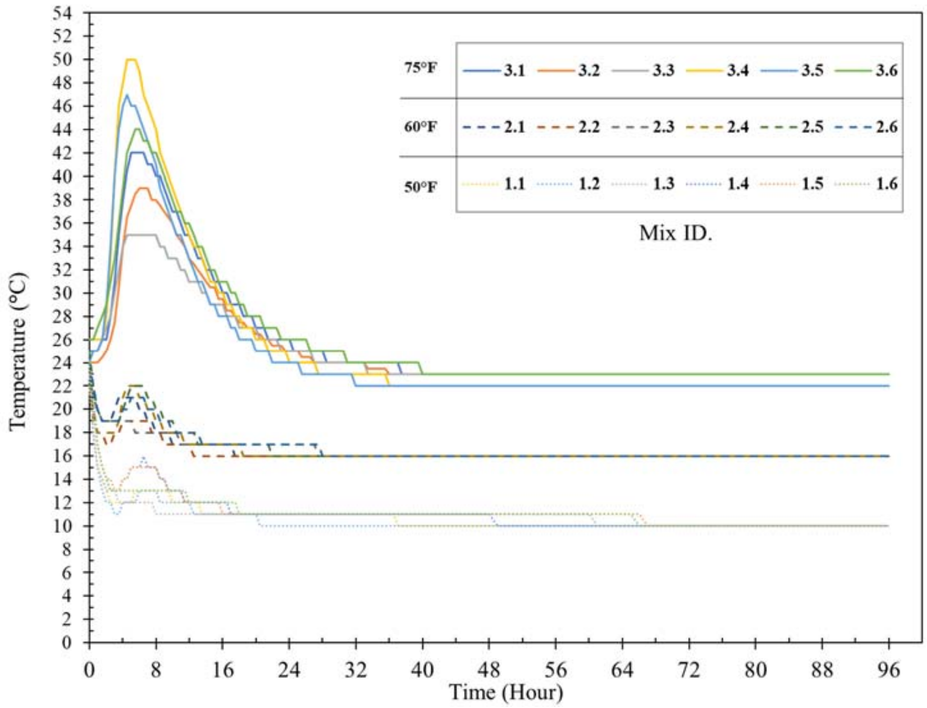


Figure 4-11. Temperature evolution of Phase 3 mixes at different ambient temperatures

4. 4. 3 Mechanical Properties

Figures 4-6 and 4-7 presents the results obtained from the compressive strength uniaxial test. The data are break down into two graphs for 50°F and 60°F ambient temperatures which simulated in the environmental chamber. The compressive strength value is the most important data in various DOT specifications of concrete pavement patching requirement so it is essential to assess in detail. Accordingly, the compressive strength (f'_c) was studied in different ages: 6, 12, 24 and 48 hours as well as 28 days. Ghafoori et al. (2017) reported that the range of required compressive strength at different traffic opening times in different states is approximately between 2,000 to 3,500 psi. It can be seen that reducing ambient temperature has delayed strength gain in all mixes. However, the impact was different for each mix because of various factors. It is interesting to note that the regular traffic-closure period cannot be employed even at 60°F ambient temperature. Another important finding was that in all three temperatures, the 28d strengths were all above 8000 psi which proves that the temperature drop occurrence during the curing (for a few days though) will not affect the ultimate strength.

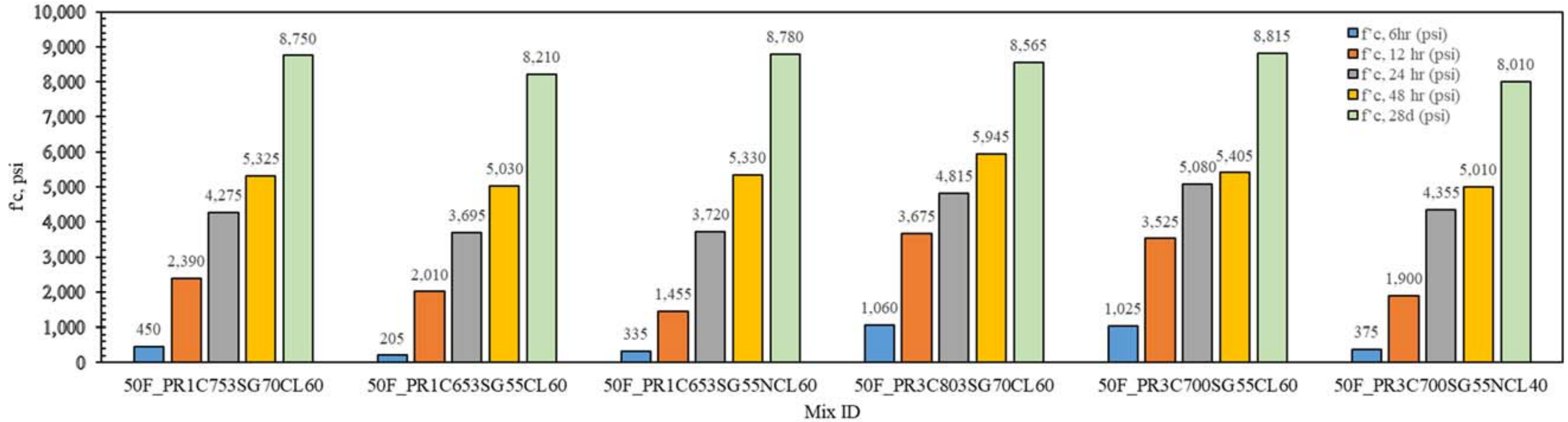


Figure 4-12. Compressive strength of Phase 3 mixes cured at 50°F.

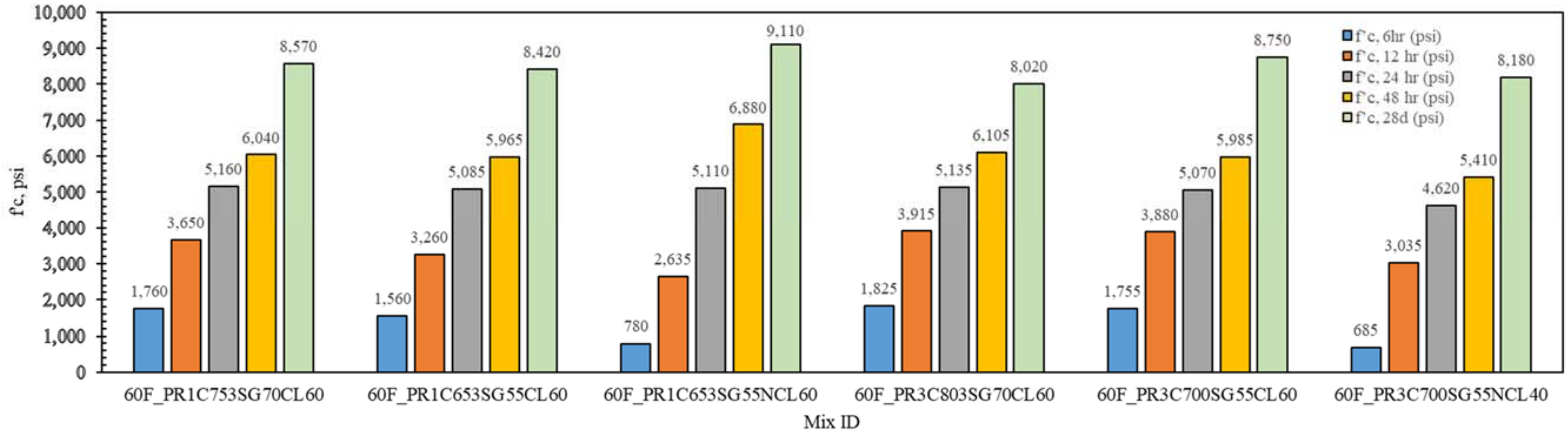
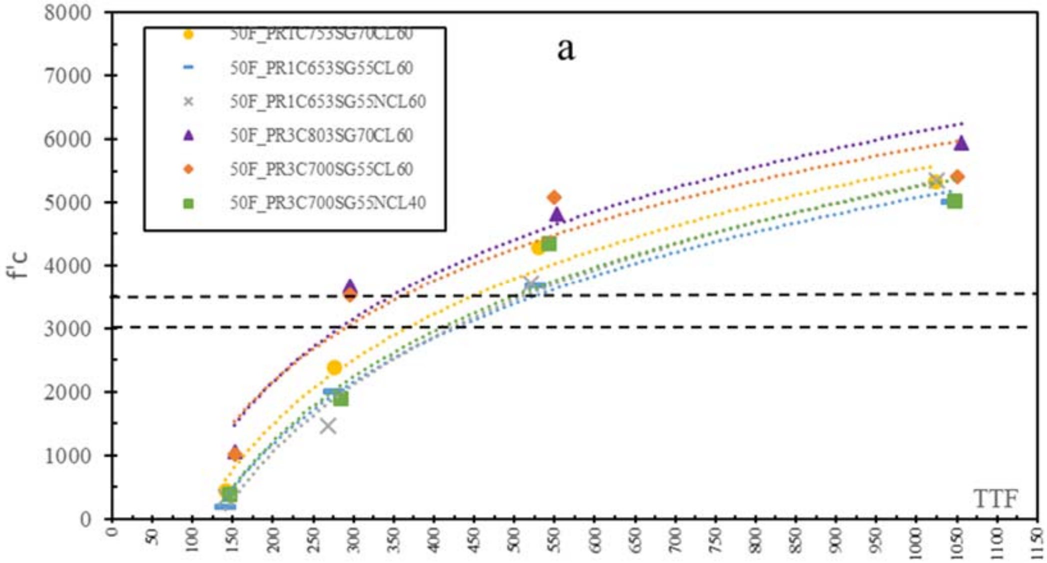


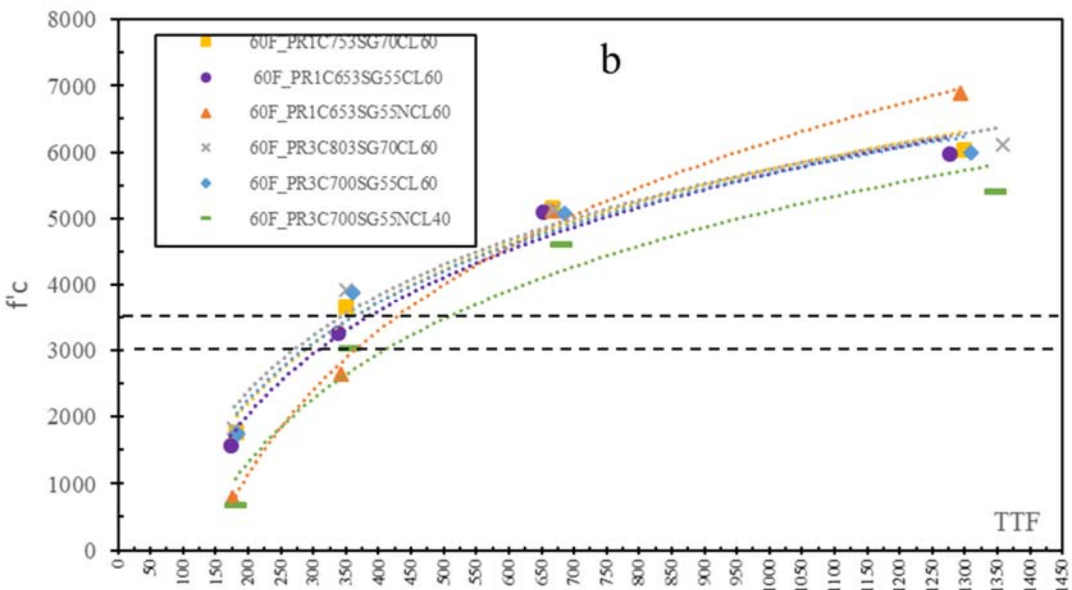
Figure 4-13. Compressive strength of Phase 3 mixes cured at 60°F.

4. 4. 4 Maturity Analysis

The compressive strength (f'c) versus Time-Temperature Factor (TTF) relationship for each mix is plotted in Figure 4-8. The best-fit curves were determined by the regression analysis for ages between the 6- and 48 -hours in 50 and 60°F.



a) 50°F



b) 60°F

Figure 4-14. Maturity analysis at different ages for ambient temperatures.

4. 4. 5 Opening to traffic estimation at different ambient temperatures

Based on the maturity analysis, opening to traffic estimation for each of the developed mixes is calculated in Table 4-9. It should be noted that the time estimation was determined per the time that the concrete mix achieves 3000 psi compressive strength.

Table 4-11. Low ambient temperature effect on the estimated time to reach 3000psi

Mix ID	Time to gain 3000 psi f'_c [hour]		
	50 °F	60 °F	75 °F
PR1C750SG70CL60	16-16.5	9.5-10	7-7.5
PR1C650SG55CL60	17-17.5	10.5-11	7-7.5
PR1C650SG55NCL60	18.5-19	12.5-13	6.5-7
PR3C800SG70CL60	11-11.5	9-9.5	6
PR3C700SG55CL60	11-11.5	9-9.5	6
PR3C700SG55NCL40	15.5-16	12-12.5	6

CHAPTER 5 CONSTRUCTABILITY AND PERFORMANCE EVALUATION

5. 1 Introduction

Debonding of the rapid patching material is a common problem in concrete pavement repair which means that the interface between the repair material and old concrete is of importance in the effectiveness of the repair patch (Guo et al. 2018; Zanotti et al. 2014). This chapter summarizes the construction procedure of slabs and patching materials based upon the identified eight developed repair mixes. The purpose of this experimental phase was to assess the constructability of developed mixes as well as the bonding behavior between the pavement slab (substrate) and patching materials.

5. 2 Slab Patching Test Setup

In order to evaluate the bonding performance of the selected eight patching mixes, two slabs of 8 ft. x 3 ft. x 0.75 ft. were constructed. Each of them had four openings with dimensions of 1 ft. x 1.5 ft. x 0.75 ft. to simulate actual full-depth repairs. The developed patching materials casted in the opening slots. The selected patching mixture designs can be found in Table 5-1. It should be noted that the selected mixes have similar designs to phases 2 and 3 of the experimental study. The slump test and compressive strength samples were also obtained from each patch mixture.

Table 5-1. Slab patching mix proportion (phase 4)

Mix No.	Mix ID	Cement Type	W/C	Agg. Blend	Cement	Water	LS	SG	CL	NCL	HRWR	AEA
				%	lb/yd^3				fl oz/cwt			
1	PR1C750SG70CL60	Type I	0.31	70-30	753	204	864	2016	60	0	5	5.5
2	PR1C650SG55CL60			55-45	653	179	1373	1678			5.5	6.0
3	PR1C650SG55NCL60			55-45	653	179	1373	1678			6	6.0
4	PR1C753SG70CL60_25% IP	Type IP/I	0.31	70-30	176 + 565	201	2027	869			6	6.0
5	PR3C800SG70CL60R	Type III	0.36	70-30	800	258	810	1890			5	6.0
6	PR3C700SG55CL60R			55-45	700	224	1301	1590			5.5	6.5
7	PR3C700SG55NCL40R			55-45	700	237	1296	1584	0	40	5.5	6.5
8	PR3C800SG55CL60_50% IP	Type IP/III	0.36	55-45	375 + 400	246	1507	1233			5.5	6.5

*1 lb/yd3 = 0.593 kg/m3; 1 fl oz/cwt = 62.64 ml/100kg; W/C, LS, SG, CL, NCL, HRWR, and AEA denote water-to-cement ratio, Limestone, Sand and Gravel, Chloride-based accelerator (Darracel), Non-Chloride based accelerator (Daraset400), High Range Water Reducer (Plastol Ext 6200), and Air Entraining Admixture (AE 90), respectively.

5. 2. 1 Slabs Construction

Plywood sheets were used to build the frameworks. Pictures from different stages of the slab construction can be found in Figure 5-1 to 5-4.

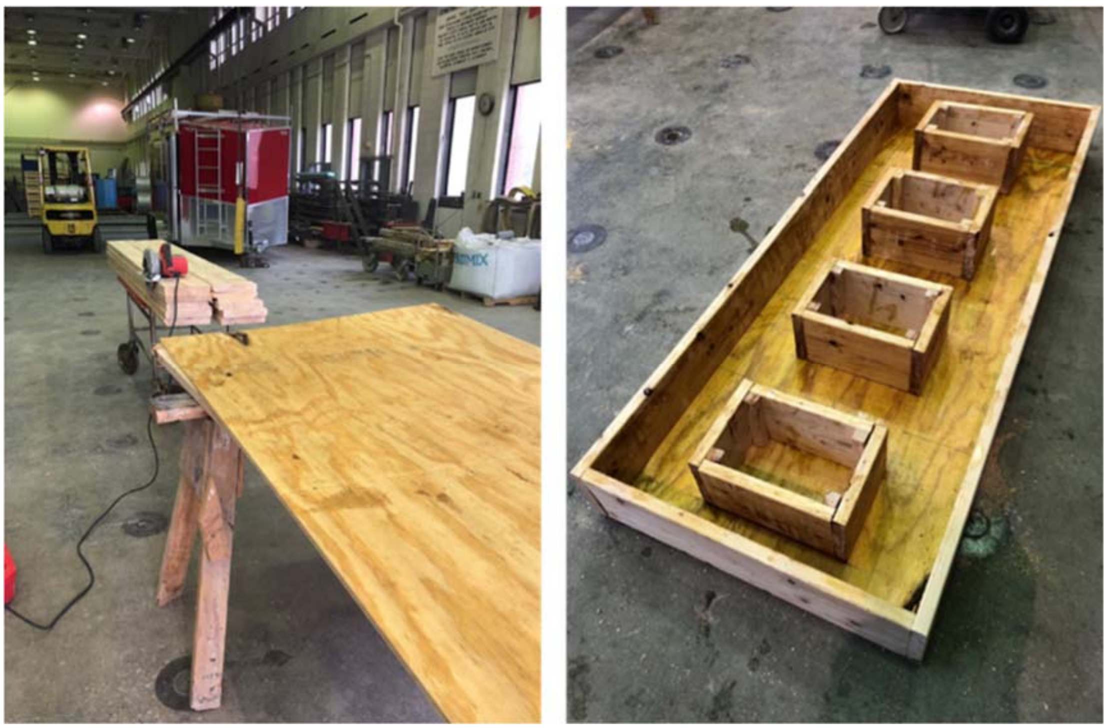


Figure 5-1. Formwork preparation.

After assembling the sheets of plywood, the frameworks were cleaned by air pressure prior to casting. Courard et al. (2014) reported that cleanliness has the highest degree of importance in the bonding strength among the substrate characteristics (even more than the surface roughness).

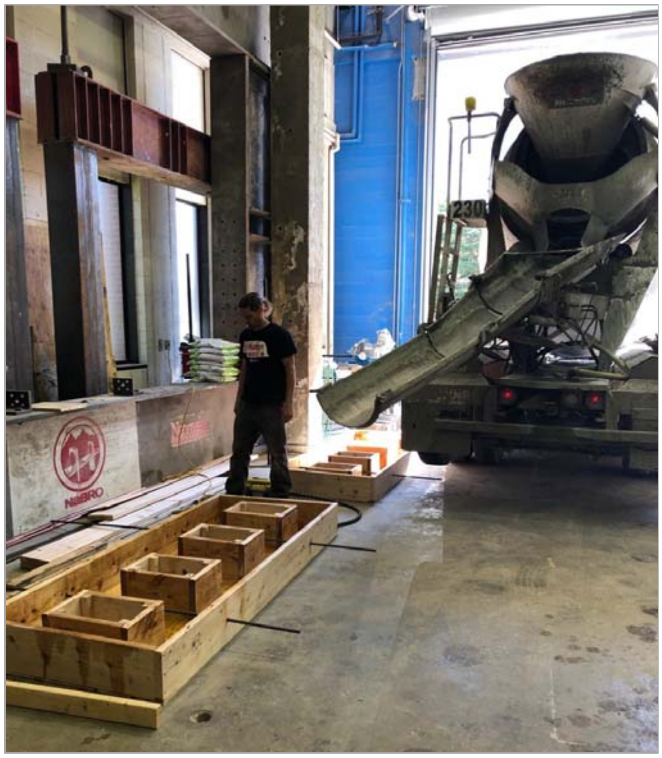


Figure 5-2. Concrete placement

After the formwork preparation, the concrete placement was carried out. A 47B state mixture, with w/c of 0.40 and a slump of 3 inches, was employed. Also, an internal concrete vibrator was used to consolidate the placed concrete. Surface finishing was done by using screeds and trowels.

Figure 5-3. Concrete slab after casting

The constructed slabs were cured with wet burlaps and plastic sheets for 28 days at an enclosed lab in the temperature of 24±3°C), Figure 5-4 (a). Premade patching slots surfaces were cleaned of any dust prior to the patching. The finished slab after demolding can be found in the Figure 5-4 (b).

(a) During moist curing **(b) Afer moist curing**

Figure 5-4. Slab curing.

5. 2. 2 Slab patching procedure

In summary, the patching materials were prepared in a lab mixer (similar to other phases of the project). Instantly after measuring the workability, the patch material was placed into the distinguished slot and consolidated with a concrete vibrator. Surface finishing was done by using screeds and trowels. The patching steps can be found in Figure 5-5. Also, thermocouples for maturity meter were installed to monitor the temperature evolution because of concrete hydration reactions. It was noted that the wires must be placed at the approximately mid-depth of the patch. An initial temperature reading was taken right after installation. Also, two thermocouple wires were used in each slot and the average values were presented. A picture of the finished patch right after the patching is shown in Figure 5-6.

Figure 5-5. Slab patching process

Figure 5-6. Finished patch surface

As shown in Figure 5-7, right after patching, the patch section was covered by wet burlaps and plastic sheets for 28 days to ensure appropriate curing.

Figure 5-7. Curing of casted patches

5. 3 Slab Performance Evaluation

The interface between substrate slab and patching materials were closely monitored until 90 days after patching, to identify if there is any premature cracking due to bonding. Results showed that no cracking was observed at the substrated slab-patching materials interfaces.

In order to evaluate the interface between substrate slab and patching materials, 4 inches diameter cores were taken from the interface between patching materials and substrate slabs for all the eight patch slots. The obtained cores mostly displayed good consistency, and no excessive shrinkage cracking was observed.

Figure 5-8. Coring process

5. 3. 1 Mechanical Properties

Splitting Tensile Strength

The ½ inch top and bottom of the cores were removed with a diamond saw to obtain 4x8 inches specimens which were tested according to splitting tensile strength test. The bond strength was qualified according to the splitting tensile into four categories (Table 5-2) of excellent, good, fair, and poor (Sprinkel et al. 2000). Table 5-3 presents the results achieved from the splitting tensile strength test. Also, an example of the fracture surface of tensile cores can be seen in Figure 5-9. The cores mostly were split through the bond.

Table 5-2. Bond strength qualification

Bond Strength (psi)	Quality
>400	Excellent
200-400	Good
100-200	Fair
0-100	Poor

Table 5-3. Results of splitting tensile strength

Mix No.	Splitting Tensile Strength (psi)	Quality
PR1-1	283.50	Good
PR1-2	204.80	Good
PR1-3	238.80	Good
PR1-4	351.90	Excellent
PR3-1	248.80	Good
PR3-2	398.10	Excellent
PR3-3	393.20	Excellent
PR3-4	322.70	Excellent

According to the split tensile strength test, the bonding quality in all eight patches was either good or excellent. The PR3 mixes displayed a slightly better interface, which might be due to the higher hydration rates of PR3-based mixes.

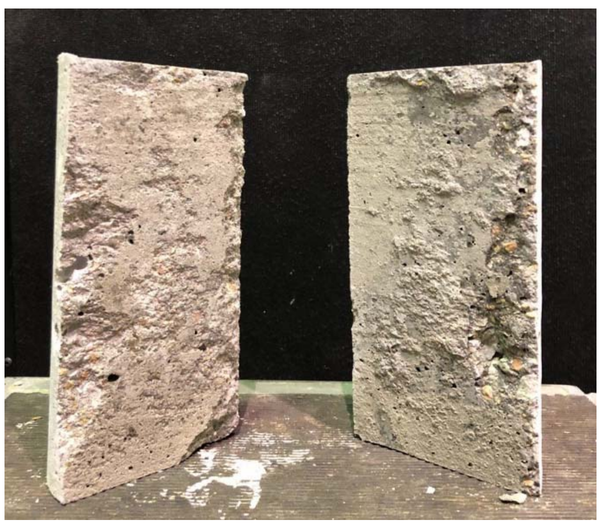
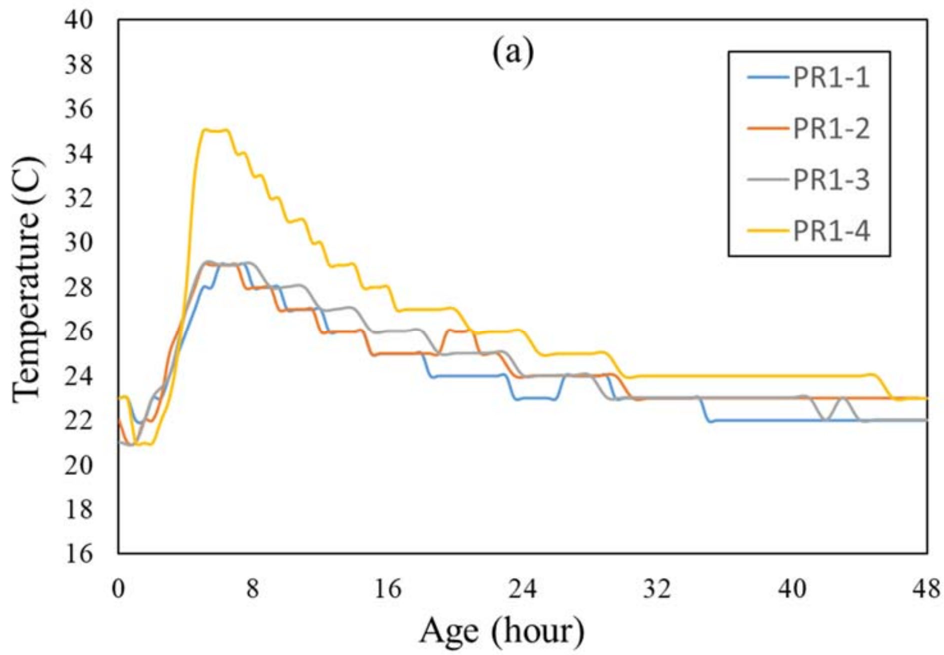


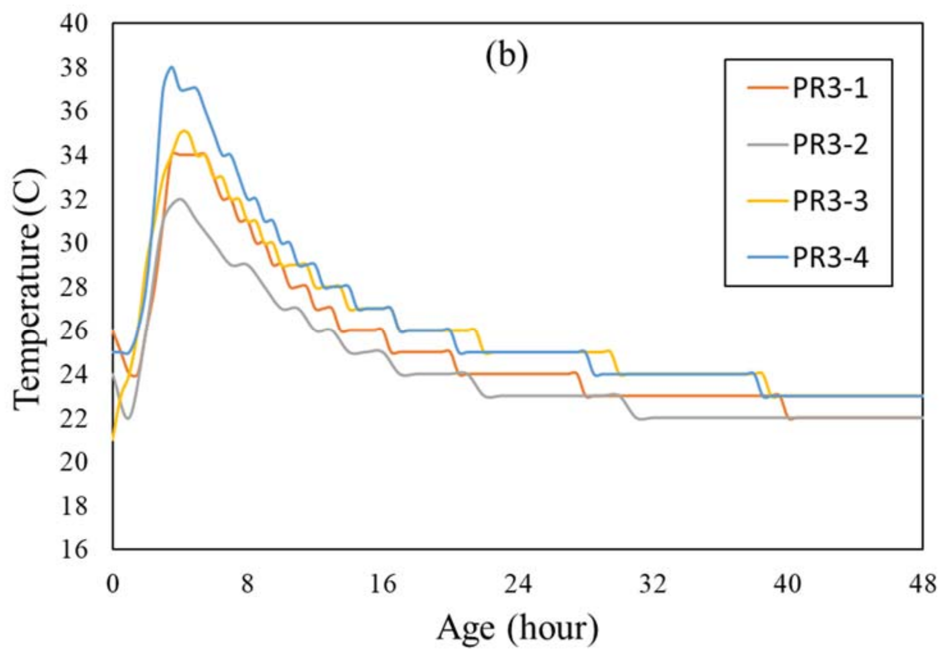
Figure 5-9. Typical fracture surface after splitting tensile test.

5. 3. 2 Temperature Evolution

As mentioned in the patching process section, two thermocouple wires were implanted in each patch slot to track the temperature evolution. The results are presented in Figure 5-10. It can be seen that PR3 mixes have higher peak temperatures in comparison with PR1 mixes. In other words, the PR3 mixes have released higher heat of hydration, which is consistent with phase 1 results. Another significant point is that the occurrence of peak temperature was not delayed, likely due to the uninsulated circumstance of slabs and for all mixes, the peak temperature took place within 8 hours of mixing time.



(a). PR1 mixes



(b). PR3 mixes

Figure 5-10. Temperature evolution of slab patches.

CHAPTER 6 COST-EFFECTIVENESS AND FEASIBILITY STUDY

6. 1 Cost-Effectiveness Analysis

With the identified materials sources and developed mixture designs, a cost analysis was performed based on material costs. The results are to be used to justify if the developed concrete mixtures are cost-effective to be implemented in different locations (i.e., west, central, and east) of the state of Nebraska.

6. 1. 1 Methodology

The unit cost of each specific material is shown the Table 6-1. It should be noted that this unit cost could be different at dissimilar periods or locations.

Table 6-1. Unit costs of materials

Material	Type	Unit cost ($)	Unit
Cement	Type I/II	130	ton
	Type III	145	ton
	Type IP	135	ton
Aggregates	Limestone	25	ton
	Sand & Gravel	18	ton
Chemical Admixtures	High Range Water Reducer	20	gal
	Air Entraining Agent	7	gal
	Accelerator	10	gal
Water	N/A	2.5	ton

6. 1. 2 Results

The base costs of developed mixes are calculated, and the results are shown in Table 6-2. It can be illustrated that reducing 100 lb. of cement per each cubic yard reduce the base cost notably. It should be noted that the transportation cost is not included in the cost estimation.

Table 6-2. Base costs of developed mixes

Mix ID	Cost ($/yd3)
PR1C750SG70CL60	121.59
PR1C650SG55CL60	113.30
PR1C650SG55NCL60	113.81
PR1C753SG70CL60_25% IP	122.07
PR3C800SG70CL60R	131.83
PR3C700SG55CL60R	122.92
PR3C700SG55NCL40R	111.88
PR3C800SG55CL60_50% IP	129.34

Based on the cost analysis, all the developed patching mixtures are cost-effective to be implemented in the state of Nebraska.

6. 2 Feasibility Study

The applicability of any repair mixes should be evaluated prior to implementation. Experimental results showed that the developed rapid patching mixes have an adequate early age strength, volume stability, compatibility with the substrate, and ease of placement that meet NDOT criteria. The major challenge that worth noting is the loss of workability (because of using accelerators) is more significant than conventional concrete mixes, which urge effectively taking advantage of the time when the concrete is in the fresh stage. While there is no clear trend fo the impact of mix design to slump loss rate, special attention is needed to ensure the rapid patching materials can be placed in a timely matter to eliminate any construction issues. Overall, the developed mixes could provide promising mixes that meet short traffic-opening requirements with lower maintenance costs.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORKS

7. 1 Conclusions

A comprehensive experimental study was carried out to investigate the properties of Portland cement-based rapid-patching materials by focusing substantially on the impact of cement reduction as compared with current Nebraska DOT rapid patching materials. The mixture designs employed different cement type and contents, conventional and optimum aggregate blends, w/c, and Chloride and non-Chloride-based accelerators to evaluate a variety of determinative factors. The results of this project will enable NDOT to produce a more durable, and cost-effective patching material. The following conclusions can be drawn:

- With the optimized aggregate gradation, reducing cement content by up to 100lb/yd^3 could still satisfy the NDOT specification, i.e., 3000 psi compressive strength within 8 hours.
- The developed mixes with reduced cement content exhibit good 28-day compressive strength, modulus of rupture, and bond strength.
- Based on the experimental study, besides a substantial drop in the heat of hydration peak temperature, cement reduction leads to a reduced drying shrinkage rate and an increase of surface resistivity, which could lead to more durable materials.
- Even with a slight decrease in early-age strength, results demonstrated that it is feasible to use a non-Chloride-based accelerator to produce rapid-patching materials with potential higher durability.
- With a replacement of 50% of Type III cement with IP cement, the potential issue of ASR in PR3 mixes can be effectively mitigated.
- When experiencing a low ambient temperature, strength growth can be delayed. Employing PR3 mixes could reduce the traffic closures' durations drastically in cold ambient temperatures compared to PR1 mixes.

7. 2 Future Works

Based on findings from this project, the following recommendations that could lead to further improvement of rapid patching materials can be made:

- Based on results from the low ambient temperature, achieving high-early-strength could be compromised drastically during cold seasons. As the repair job cannot be ceased during the winter, further adjustment of developed PRI and PRIII mixes, potentially with a higher dosage or different types of accelerators could be beneficial for spring/fall construction. Other types of rapid patching materials including non-Portland-based cement mixes that more feasible for cold weather repair should also be explored.

- Slag is expected to be more reactive than fly ash, and as a result, and might allows a higher replacement level, compared to fly ash. In order to achieve more durable repair patches, the use of type IS cement (Portland cement blended with slag cement) in PR mixes should also be evaluated.
- There is a need for defining emergency vs. non-emergency repair protocols which the former could meet super short traffic openings or low temperatures while the latter could be employed for regular applications with higher durability.

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