# Durable high early strength concrete

by

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# Abstract

Based on a 2017 report on infrastructure by the American Society of Civil Engineers, 13% of Kansas public roads are in poor condition. Furthermore, reconstruction of a two-lane concrete pavement costs between $0.8 and $1.15 million dollars per lane mile. High early strength Portland cement concrete pavement (PCCP) patches are widely used in pavement preservation in Kansas due to the ability to open to traffic early. However, these repairs done by the Kansas Department of Transportation (KDOT) deteriorate faster than expected, though, prompting a need for inexpensive, durable high early strength concrete repair mixtures that meet KDOT standards (i.e., a 20-year service life). This study developed an experimental matrix consisting of six PCCP patching mixture designs with varying cement content and calcium chloride dosage. The mixtures were subjected to isothermal calorimetry, strength testing, drying shrinkage, and various durability tests. The effects of cement content and calcium chloride dosage on concrete strength and durability were then investigated. In addition, the compressive strength development with time, the split tensile versus compressive strength relationship, and the shrinkage strain of the PCCP patching mixtures were compared to established relationships provided by the American Concrete Institute (ACI). Results showed a maximum 3% increase in total heat generated by various concrete paste samples in isothermal calorimetry testing. The minimum compressive strength of 1,800 psi required by KDOT could likely be obtained using any of the PCCP mixtures, regardless of the cement content or calcium chloride dosage used in the study. Furthermore, surface resistivity tests for mixtures containing calcium chloride could result in erroneous measurements. Only one mixture satisfied the maximum expansion and minimum relative dynamic modulus of elasticity required by KDOT. Some ACI relationships for shrinkage and strength development do not appear to be valid for high early strength PCCP patching mixtures.

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# Dedication

To my parents Ruben and Maria Del Refugio Porras Zaragoza. I could never have done this without your faith, support, and constant encouragement. Thank you for teaching me to believe in myself, in God, and in my dreams. I also dedicate this thesis to my sister Veronica, as well as to my brothers Ruben and Bradley Porras. Anything is possible if you believe in yourself and have the courage to pursue it.

# Chapter 1 - Introduction

## 1.1 Overview

According to a 2017 infrastructure report by the American Society of Civil Engineers (ASCE), 13% of the 140,654 miles of public roads in Kansas are in poor condition (ASCE, 2017). On average, Kansas motorists spend $500 per year in extra operating costs as a result of driving on roads in need of repair (ASCE, 2017). However, reconstruction of a two-lane concrete pavement costs between $0.8 and $1.15 million per lane mile (McLeod et al., 2014). The high costs associated with pavement reconstruction and vehicle repairs stress the need for inexpensive, durable concrete pavements in Kansas.

Pavement preservation is "a program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extends pavement life, improves safety and meets motorist expectations" (Geiger, 2005). Figure 1-1 shows three major phases in pavement preservation based on pavement condition. These phases include preventive maintenance, rehabilitation, and reconstruction. One common pavement preservation technique in Kansas is slab patching and/or slab replacement with high early strength Portland cement concrete pavement (PCCP). However, the Kansas Department of Transportation (KDOT) has noticed that PCCP patching repairs deteriorate quickly, requiring replacement less than 10 years after implementation. Although concrete patches are considered temporary, current limited funding has prompted the need for more permanent pavement patches that would last at least 20 years. This predetermined 20-year service life is half of the overall 40-year total life of concrete pavements in Kansas (McLeod et al., 2014).

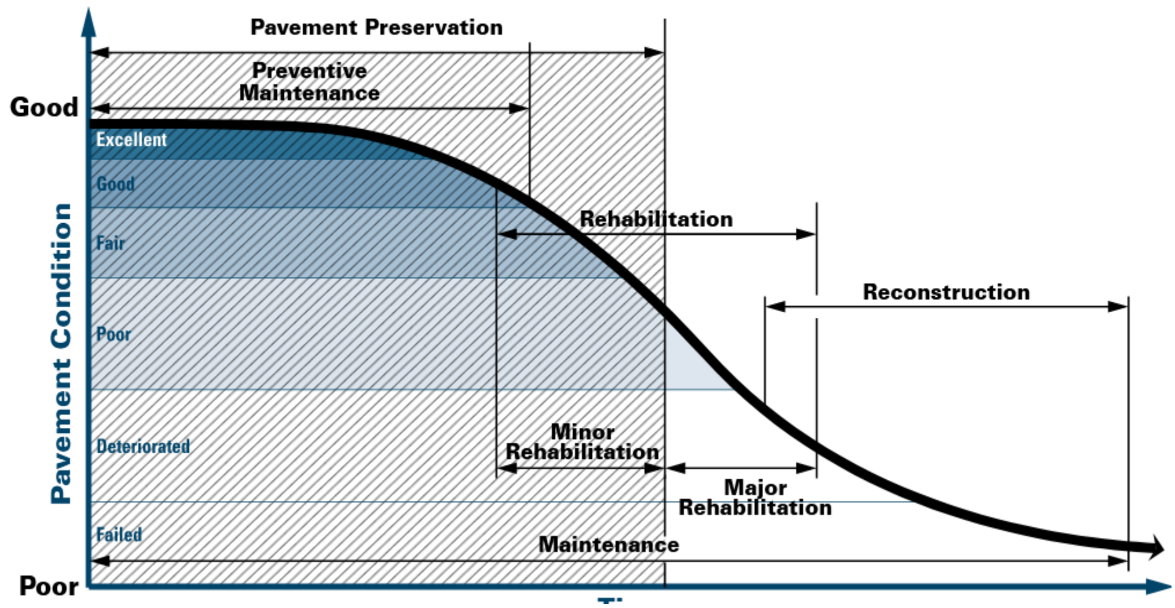


**Figure 1-1: Phases of Pavement Preservation (Peshkin et al., 2011)**

The use of high early strength PCCP repairs is essential when attempting to open a facility as quickly as possible to avoid traffic congestion, especially in urban areas. Urban areas in Kansas are situated in KDOT Districts I and V, as illustrated in Figure 1-2. To avoid traffic congestion, a contractor may close a lane of traffic after rush hour in the evening, remove the deteriorated concrete, and place the high early strength concrete in time for the roadway to be in service the following morning. The contractor must place the last patch between midnight and 2:00 a.m. to allow at least 4 hours of curing before the road is reopened to traffic. In Kansas, a facility is typically opened to traffic in District I at 5:00 a.m., and a closed facility in District V is typically opened at 6:00 a.m. Patches must achieve a minimum of 1,800 pounds per square inch (psi) in compressive strength or 380 psi in flexural strength before the road is opened to traffic, as listed in Section 833 of the Kansas Standard Specifications for State Road & Bridge Construction (KDOT 2015).

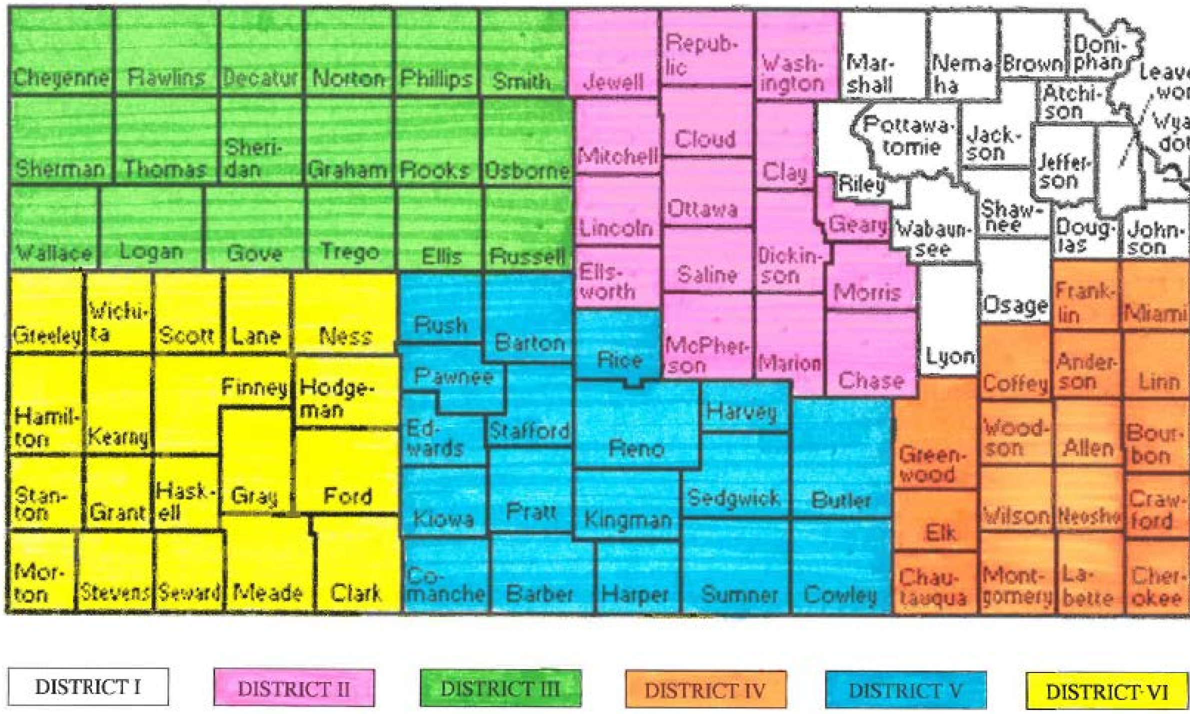


**Figure 1-2: KDOT Districts**

Rapid repair contracts contain a penalty provision if patches do not meet minimum compressive strength requirements when the road is opened. Although payment is rare, a penalty of $1,000 per half hour the patched pavement is closed has been enforced in District V (Turner, 2017). To avoid the penalty, contractors may increase the cement content unilaterally to accelerate the strength gain of PCCP patches. KDOT allows this practice in the Standard Specifications for State Road and Bridge, Section 501. Although the minimum specified cement content for air-entrained concrete pavement is 517 pounds per cubic yard (lbs/yd$^3$) (Section 403: *On Grade Concrete*), no upper limit exists for cement content, even for high early strength concrete. KDOT recommends adding 2% calcium chloride (by dry weight of cement) for pavements cured for 4 hours and 1% calcium chloride (by dry weight of cement) for pavements cured for 6 hours (KDOT, 2015). The additional cement and calcium chloride ensures the contractor will avoid the penalty for high early strength repairs. However, high levels of cement and calcium chloride may reduce pavement durability in patched PCCPs.

PCCP patches can show various modes of failures leading to rapid deterioration. Common failure modes include cracking of the repair material, cracking in the substrate concrete, and cracking at the interface between the substrate and the repair. These common failure modes are illustrated in Figure 1-3. This research focused on cracking in the repair material.

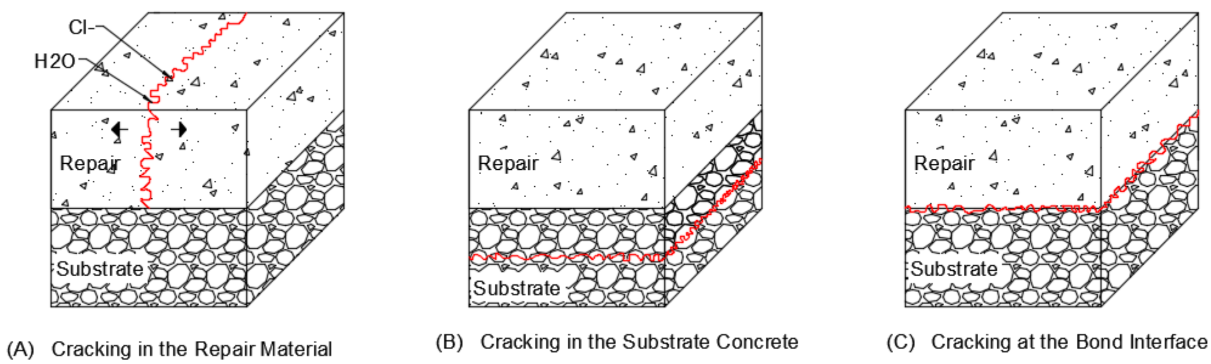


**Figure 1-3: Common Types of Failure Modes in Patching Systems**

Strength gain in PCCP patches is influenced by cement type and accelerator. The predominant cement type in high early strength repairs or patching is American Society of Testing Materials (ASTM) Type III Portland cement, which has finely ground particles that increase the surface area that encounters mixing water, resulting in an increased rate of hydration. Common accelerators to increase strength gain can be classified as chloride and non-chloride types. Calcium chloride, which is most often used as the main accelerator due to its low cost, reduces the initial set time from 3 hours to 1.5 hours when 1% calcium chloride (by dry weight of cement) is added to the mixture. An addition of 2% calcium chloride (by dry weight of cement) results in an initial set time as low as 1 hour (Mamlouk & Zaniewski, 2011). PCCP patching repair materials in Kansas can be subdivided into four categories:

- Type III Portland cement with a calcium chloride accelerator
- Type III with non-calcium chloride
- Type I/II with calcium chloride
- Type I/II Portland cement with a non-calcium chloride accelerator

Type III Portland cement with a calcium chloride accelerator was the main topic of this research because it is most commonly used for PCCP patching in Kansas. The primary objective of this study was to improve repair durability by redesigning PCCP patching mixes to expedite roadway openings. Specifically, this project was designed to identify optimal ratios of cement content and calcium chloride dosage to maximize durability of high early strength PCCP repairs.

## 1.2 Background

High early strength concrete minimizes traffic disruption by allowing the facility to open sooner. Concrete pavement repairs on high-volume routes in Kansas often use this type of concrete mixture. Typical high early strength concrete repairs have high cement content, low water-to-cementitious material (w/c) ratio, and chemical admixtures to achieve high early strength (Shanahan et al., 2016). Recent evaluation of these repairs indicated premature deterioration and a pavement life expectancy of only 5–10 years. Figure 1-4 shows prematurely deteriorated repair slabs in Garden City, Kansas. As financial resources become increasingly limited, high early strength repairs must provide longer pavement service life while satisfying current KDOT minimum strengths.

**Figure 1-4: Prematurely Deteriorated Slabs in Garden City, Kansas (photo courtesy of Rodney Montney, KDOT)**

## 1.3 Problem Statement

A concrete pavement can undergo up to two rehabilitation periods within an expected life cycle of 40 years (McLeod et al., 2014). High early strength concrete patching is a prevalent rapid repair technique used to limit traffic congestion during the rehabilitation periods, but high early strength concrete patching repairs done by KDOT are deteriorating at a faster rate than anticipated. City and county agencies in Kansas need guidelines for designing a concrete patching mix with a durable 20-year service life.

## 1.4 Objective

The primary objective of this study was to develop durable high early strength concrete mixtures that meet early strength requirements. Specifically, these mixtures must

- satisfy current KDOT minimum strengths for opening to traffic while complying with KDOT-desired fresh concrete properties, and
- provide a 20-year pavement service life.

This research project was divided into three phases. Phase one compiled background information, current uses, and existing research on the durability of high early strength concrete. Phase two developed and tested several preliminary concrete mix designs by optimizing the cement content and calcium chloride dosage to satisfy KDOT minimum strength requirements for opening to traffic. Once a preliminary concrete mix design satisfied the high early strength criterion, mix designs underwent durability testing. Phase three included evaluation of concrete properties of the optimized concrete mixes designed during phase two and composition of the final report.

## 1.5 Research Methodology

This study gathered PCCP patching mix designs from KDOT Districts I, II, V, and VI, but project scope limited the focus of this research to improving District V PCCP patching mix designs. KDOT performed isothermal calorimetry tests on the cement paste and admixtures used in the repair mixes to increase understanding of the effects of admixtures on reaction rate and heat generation. General trends were evident for District V PCCP patching mix designs that contained Type III Portland cement and calcium chloride. These trends formed the basis of the mixture design developed in this study. An experimental matrix for concrete mix design with varying cement content and calcium chloride dosage, as tabulated in Table 1-1, was developed to find optimal quantities of cement and calcium chloride dosage in a concrete repair mixture that passed durability testing. Durability tests included dry shrinkage (ASTM C157), surface resistivity (KT-79), scaling resistance (ASTM C672), and freeze-thaw (KTMR-22). ASTM C39 and ASTM C496 were used to track compressive and tensile strengths, respectively, at 4, 6, 24, and 72 hours, and at 7 and 28 days.

**Table 1-1: Optimal Cement Content and Calcium Chloride Matrix**

| Cement Content ($lb/yd^3$) | Liquid Calcium Chloride - 32% Concentration | |
|---|---|---|
| | 1% (by dry weight of cement) | 2% (by dry weight of cement) |
| 564 | PCCP ID 1 | PCCP ID 2 |
| 658 | PCCP ID 3 | PCCP ID 4 |
| 752 | PCCP ID 5 | PCCP ID 6 |

The cement content and calcium chloride dosages shown in Table 1-1 were obtained from various sources. A minimum cement content of 517 $lb/yd^3$ was initially identified in the 2015 version of Kansas standard specifications for air-entrained on-grade concrete pavement. During preparation of trial mixes, however, a concrete mixture with this cement content could not feasibly be produced without exceeding the manufacturer's recommendations for air-entraining and water-reducing admixture dosages. Furthermore, because the trial mixes did not reach the required minimum strength in the allotted time, a cement content of 564 $lb/yd^3$ for air-entrained concrete pavement was chosen, a quantity from the Kansas special provisions to the standard specifications, 2007 edition. The cement content of 658 $lb/yd^3$ (Table 1-1) was the required cement content for concrete patching originally stipulated in the 1990 Kansas standard specifications, and the cement content of 752 $lb/yd^3$ was suggested by Susan Turner, field engineering administrator of KDOT, as the common maximum cement content for concrete patching in KDOT District V. Calcium chloride contents of 1% and 2% were chosen based on KDOT specifications for this accelerator.

Small-scale trial mixes for all PCCP mixes were conducted to ensure adequate fresh concrete properties and the minimum strength of 1,800 psi in 6 hours.

### *1.5.1 District 5 General Trends*

After analyzing all PCCP patching mix designs received from District V, the following general trends were found to dictate mix design:

- 50/50% blend of fine and coarse aggregates
- unit weight range: 137–144 lb/ft$^3$
- w/c ratio range: 0.3–0.45
- mix designs with either type A or type F water reducer
- target air content: 6.5%
- cement content range: 658–752 lb/yd$^3$
- dosage of calcium chloride equal to 2% by dry weight of cement

### *1.5.2 Mixture Design*

In order to ensure consistency of the PCCP mixes and only vary the cement content and calcium chloride dosage, each PCCP mix ideally contained the following:

- 100% Type III Portland cement (Monarch)
- w/c ratio of 0.37 (based on average w/c ratio for District V)
- air content of 6.5% ± 1.5%
- slump of 2.5–5 in.
- 50% FA-A sand
- 50% CPA-4 limestone
- Daravair® 1400 air-entraining agent from GCP Applied Technologies
- ADVA® 140M type A/F water reducer provided by GCP Applied Technologies
- liquid calcium chloride with a 32% concentration from Scotwood Industries, Inc.

### *1.5.3 Optimization*

The optimal mix design for District V should have the lowest possible cement content and calcium chloride dosage with a minimum compressive strength of 1,800 psi at 6 hours maximum and a minimum of 4,000 psi compressive strength at 28 days. Ideally, the optimal mix should pass KTMR-22 test requirements as well as demonstrate high surface resistivity with a minimum of 9 kilo ohm-centimeter (kΩ-cm), minimal dry shrinkage, and minimal scaling.

## 1.6 Thesis Outline

This thesis includes seven chapters. Chapter 1 includes an introduction to the research. Chapter 2 consists of a literature review, including general requirements and methods to obtain high early strength. The literature review also examines a case study by the Strategic Highway Research Program (SHRP) that was performed on high early strength concrete, as well as factors affecting concrete durability and durability testing mechanisms. The materials used for all designed concrete mixes are discussed in Chapter 3, and Chapter 4 provides detailed explanations of experimental methods used in this study. Chapter 5 presents the results of this research, Chapter 6 includes analysis and discussion, and Chapter 7 contains conclusions and recommendations.

# Chapter 2 - Literature Review

## 2.1 History of High Early Strength Concrete in Kansas

High early strength concrete is a vital component of high-volume roadway repairs, especially as the current transportation infrastructure deteriorates. The rapid deterioration of repairs requires improved high early strength concrete repair mixes, which means enhancing durability while achieving required strength in the allotted time span. The Kansas Standard Specifications for State Road and Bridge Construction is a document provided by KDOT. KDOT periodically releases a new standard specifications book. The most recent releases occurred in 1990, 2007, and 2015. In response to problems or industry advances between releases, KDOT issues special provisions to update, correct, or modify standards in the specifications book (Turner, 2017).

### *2.1.1. KDOT Standard Specifications (1990 Version) and Special Provisions Released in 2003*

Section 812 of the 1990 KDOT standard specifications clearly stipulates characteristics of concrete patches. For example, maximum slump of concrete during patching is 2.5 in., and all patches must be air entrained. For an accelerated cure, standard specifications require a minimum 658 lb/yd$^3$ of Type III cement with a 1% or 2% grade II calcium chloride (by dry weight of cement), as approved by the engineer and added in solution form. Grade I contains 77% calcium chloride, and grade II contains 90% calcium chloride in flake, pellet, granular, or liquid form. KDOT considers the liquid solution of calcium chloride to be part of the mixing water. Minimum time after concrete patch placement is 6 hours before opening to traffic if 1% calcium chloride is used with an air temperature at or above 60$^o$F. If 2% calcium chloride is added, 4 hours are needed before opening the patch to traffic if the air temperature is at 60$^o$F or above. Extra precaution is needed if the ambient temperature falls below 60$^o$F. A curing membrane must also be placed on the patch as with normal strength concrete pavements (KDOT, 1990).

A table designated for patching repair was first established in the special provisions to the 1990 Kansas standard specifications. This table contained categories and requirements for concrete patching. The categories included aggregates, cement, calcium chloride, air-entraining admixture, water, and slump. This special provision, released in 2003, designated a liquid membrane-forming compound rate for curing concrete patches: 1 gallon per 150 sq. ft. applied to the finished patch. The provision also stipulated the minimum width and depth of sawed transverse and longitudinal joints and initially outlined the procedure for removing and cleaning patch areas, especially for partial-depth patches. For full-depth patches, the document required a minimum size of 6 feet in the longitudinal direction (KDOT, 2003).

### *2.1.2 KDOT Standard Specifications (2007 Version) and Special Provisions Released in 2011*

Although section 812 was changed to section 833 in the 2007 edition of the standard specifications, it was similar to special provisions of the 1990 standard specifications for section 812. For example, the section retained the table that included the minimum 658 lb/yd$^3$ of Type III cement for concrete with an accelerated cure and a maximum slump of 2.5 in. (i.e., *Table 833-1: Additional Requirements For Pavement Patching Air-Entrained Concrete*) (KDOT, 2007).

Special provisions to the 2007 standard specifications entailed significant changes. One of the principal changes was the removal of Table 833-1, which eliminated the minimum cement content and the maximum slump requirement for concrete patching (KDOT, 2011), thereby implying that the only minimum cement and/or slump requirement should come from section 401 (i.e., *Concrete*) in the special provisions to the standard specifications edition 2007. Section 401 established the minimum cement content to be 564 lb/yd$^3$ with coarse and fine aggregates, and the design air content was stipulated as a minimum of 4.0% behind the paver and a slump of 2.5 in. However, this slump could increase with use of a water reducer. Section 833 in the special provisions classified the concrete pavement patching section in the 2007 standard specifications into seven subsections: PCCP Patching Location, PCCP Patching Removal, PCCP Patch Preparation, PCCP Patch Concrete Placement, PCCP Patch Curing, Joints, and Opening to Traffic (KDOT, 2011). These changes indicated that the pavement patching section had begun to necessitate more construction requirements than previous material requirements.

### *2.1.3 KDOT Standard Specifications (2015 Version)*

The latest release of standard specifications occurred in 2015. Section 833 of this edition incorporated another subsection (i.e., *Finishing*) within the concrete pavement patching section. The 2015 standard categorized section 401 of the 2007 version into five sections, labeled as 401 through 405. Section 401 (i.e., *General Concrete*) designated the air content range to be 6.5 ± 1.5% and the slump range to be 2.5–5 in. However, section 403 (i.e., *On-Grade Concrete*) stipulated a maximum air content of 10% with immediate steps to reduce the air content whenever the air content exceeded 8%. In this 2015 edition, the minimum cement content for air-entrained concrete intended for on-grade concrete pavement changed from 564 lb/yd$^3$ to 517 lb/yd$^3$, including a minimum of 5% air content behind the paver (KDOT, 2015).

## 2.2 Approaches to Obtain High Early Strength in Concrete

Current high early strength concrete repairs are completed with traditional concrete ingredients according to common concrete practices. Various approaches are employed to obtain high early strength in concrete depending on the strength rate desired and budget. This research obtained high early strength concrete using a combination of the following:

- Portland cement
- low w/c ratio
- high cement content
- chemical admixtures
- specific mixing procedure

### *2.2.1 Portland Cement*

Portland cements used in high early strength concrete tend to be less expensive than proprietary rapid-hardening cements. The concrete producer can also control the accelerating and retarding characteristics using different admixtures to meet specific project requirements (Anderson et al., 2002). Portland cement can be categorized into five types depending on composition and blaine fineness, with Type III cement specifically produced for high early strength development. Lime, iron, silica, and alumina are the primary raw materials used to produce

Portland cement. These raw materials are calcinated in a kiln, and their molecular composition is restructured to produce four major compounds: tricalcium silicate, dicalcium silicate, tricalcium aluminate, and tetracalcium aluminoferrite. The silicates contribute to strength gain of concrete and, when hydrated, they produce calcium silicate hydrate. Type I and Type III Portland cement are relatively identical in composition, but they have unique fineness: 1,806 feet squared per pound ($ft^2/lb$) and 2,637 $ft^2/lb$, respectively. Finely ground Type III provides increased surface area that encounters water, leading to faster particle hydration and strength development (Mamlouk & Zaniewski, 2011).

### *2.2.2 Low Water-to-Cement Ratio and High Cement Content*

The w/c ratio is a factor that predominantly affects concrete strength. In 1919, Duff Abrams proved that concrete strength is inversely proportional to the w/c ratio when concrete is fully consolidated. Subsequent research, however, has shown that mixes with extremely low w/c ratios and high cement content (higher than 900 $lb/yd^3$) tend to retrogress in strength when large-sized aggregates are used, potentially due to excessive shrinkage of the cement paste, causing loss of the cement-aggregate bond (Neville, 1996). A high cement content and low w/c ratio are common in high early strength concrete (Shanahan et al., 2016).

Another factor affected by the w/c ratio is porosity; concrete with low w/c ratios tend to have low porosity. Kim et al. (2014) investigated the effect of w/c ratio on the durability and porosity of cement mortars with constant cement content. Their research indicated that when the w/c ratio increased from 0.45 to 0.60, the porosity increased up to 150% and compressive strength decreased by 75.6%, indicating significant changes in concrete properties associated with changes in the w/c ratio.

### *2.2.3 Chemical Admixtures*

Common chemical admixtures in PCCP repair concrete include air-entraining, water-reducing, and accelerating admixtures (Shanahan et al., 2016).

#### 2.2.3.1 Air-Entraining Agent

Air-entraining admixtures, which have been used since the 1930s, produce spherical bubbles that are evenly distributed throughout the fresh concrete. These air bubbles are also considerably smaller than natural air voids found in fresh concrete. Air-entraining admixtures are primarily used in concrete to provide a high degree of resistance to freezing and thawing. For protection against freezing and thawing, especially when chemical de-icers are used, the size and distribution of entrained air bubbles must be considered. The entrained bubbles must be closely spaced to avoid high internal hydraulic pressure in the paste and to allow water movement through the air voids. The Bureau of Public Roads conducted an extensive evaluation of air-entraining admixtures and found that many materials are capable of functioning as air-entraining admixtures (Klieger, 1966). Typical air-entraining admixtures include salts of wood resins, synthetic detergents, salts of sulfonated lignin, salts of petroleum acids, salts of proteinaceous materials, fatty and resinous acids, and organic salts of sulfonated hydrocarbons. Air-entraining admixtures can be liquid, powders, or semisolids. Factors such as cement content, w/c ratio, temperature, and mixing conditions also affect the air content in concrete mixtures (Klieger, 1966). These factors must be considered during mixing and placing of concrete to achieve the target air content. Various departments of transportation (DOTs) have set allowable minimum and maximum air contents for their concrete pavements (Ghafoori et al., 2017).

#### 2.2.3.2 Water Reducers

Water-reducing admixtures are commonly added to high early strength concrete mixes to reduce the quantity of mixing water required for a given consistency. In other words, these admixtures provide workability and reduce permeability while maintaining a relatively low w/c ratio, resulting in increased strength and durability. A variety of water-reducing admixtures can achieve a 5%–15% reduction in water content without adversely affecting concrete strength or durability. Water reducers are inexpensively made from lignosulfonates, hydroxylated carboxylic acids, or carbohydrates. They allow de-flocculation and dispersion of cement particles, resulting in an improved water-cement particle interaction, thus creating a more uniform lubrication and hydration process. However, high shrinkage rates, rapid slump loss, and increased bleeding are associated with many water-reducing admixtures (Nawy & Whitney, 2008).

#### 2.2.3.3 Accelerators

Accelerators are used in concrete repair applications to speed up the setting times for opening a pavement to traffic. Materials used as accelerators include silicates, fluorosilicates, thiocyanates, alkali hydroxides, calcium formate, calcium nitrate, calcium thiosulfate, potassium carbonate, sodium chloride, aluminum chloride, and calcium chloride (Nawy & Whitney, 2008). Calcium chloride, one of the first accelerators used in concrete, is the most common chemical accelerating admixture used in concrete repairs. The first use of calcium chloride in concrete occurred in 1873, and the first patent was issued in 1885 (Ramachandran, 1978). Calcium chloride has low cost, high availability, and predictable performance characteristics, making it a popular admixture in the concrete industry. Studies have shown that mixtures with a calcium chloride accelerator require less air-entraining admixtures to produce the desired air content.

The use of calcium chloride as an accelerator is a controversial topic, however, because although it increases concrete resistance to erosion and abrasion, it also contributes to a 10%–15% increase in dry shrinkage (Neville, 1996). In addition, the use of calcium chloride produces large bubbles with increased spacing, thus reducing the effectiveness of the air-entraining agent and making the concrete susceptible to freeze-thaw damage. Furthermore, calcium chloride increases the number of fine pores and decreases the number of coarse pores present in a concrete mixture, which leads to a discontinuous pore structure that is more susceptible to physical and chemical attacks. Research has shown that 28-day compressive and tensile strength slightly decreases in mixtures with calcium chloride compared to mixtures without this accelerator (Van Dam et al., 2005). The effects of calcium chloride should be carefully considered before usage, since a rapid chemical reaction occurs if calcium chloride directly encounters air-entraining admixtures, potentially adversely affecting the air content (Klieger, 1966).

Most DOTs use 1% or 2% calcium chloride (by dry weight of cement) as an accelerator in concrete repair applications. The calcium chloride dosage is based on a flake form of dehydrated calcium chloride with a concentration of 77% (Concrete Construction Publications Incorporated, 1976). The common form of calcium chloride used in high early strength repairs is a 32% concentrated liquid solution. Therefore, correct conversions should be considered to obtain accurate dosage when developing a mix design with calcium chloride.

### *2.2.4 Mixing Procedure for High Early Strength Concrete*

Dale Beggs, formerly with Ash Grove Cement Company in Overland Park, Kansas, suggested the following mixing sequence to avoid inhibiting air entrainment while using air-entertaining, water-reducing, and calcium chloride admixtures:

- Wet the mixer pan and wipe with a sponge to remove excess water;
- Add all of the coarse aggregate into the mixer;
- Add ¾ of the sand and ¾ of the water;
- Turn on the mixer for 15–20 seconds, and then add the air-entraining admixture that has been diluted with mixing water directly into the mix to increase dispersion;
- Mix for another 15–20 seconds;
- Add all the cement and the rest of the sand and water;
- Mix until a cohesive paste forms (usually another 15 seconds);
- Add the water reducer;
- Start the 3-minute mix, 3-minute rest, 2-minute mix cycle per ASTM C192; and
- Add the calcium chloride during the last 2-minute mixing period.

## 2.3 Current Practices

This study reviewed current practices of full-depth concrete pavement repairs using high early strength concrete from various state and highway agencies. The following sections summarize and describe of research findings.

### *2.3.1 Kansas (2015 Standard Specifications)*

KDOT standard specifications for high early strength concrete allow the use of Type I, II, or III Portland cement and blended hydraulic cements that include Portland-pozzolan cement, Portland blast-furnace slag cement, and ternary-blended cement if they meet requirements of strength development and timely opening-to-traffic expectations. Section 1700 of the specifications permit rapid-set concrete patching materials if they comply with ASTM C928: *Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs*. They must also have a minimum durability factor of 90% and expansion less than 0.10% at the end of 300 freeze-thaw cycles, according to ASTM C666: *Resistance of Concrete to Rapid Freezing and Thawing Procedure B*. The durability factor is defined as the relative dynamic modulus of elasticity (RDME), as defined in Section 4.3.7 Freeze-Thaw *Testing KTMR-22*, multiplied by the number of cycles at which the RDME was taken and then divided by the specified number of cycles at which the test is completed (ASTM, 2015a).

KDOT identifies prequalified Portland cements, blended hydraulic cements, and rapid-set concrete patching materials but does not specify minimum or maximum pounds of cementitious material per cubic yard for high early strength concrete. A maximum of 0.45 w/c ratio is stipulated for air-entrained on-grade concrete pavement (KDOT, 2015), and typical cement content for pavement and bridge deck repairs is between 600 and 900 lb/yd$^3$, with a w/c ratio between 0.4 and 0.5. (Ghafoori et al., 2017). A minimum of 4,000 psi 28-day compressive strength and a minimum of 9 kΩ-cm surface resistivity are also required for on-grade concrete pavements. Minimum surface resistivity does not apply to concrete patching materials when the pavement to be patched is more than 10 years old (KDOT, 2015).

KDOT specifications provide only a limited number of requirements for high early strength concrete. One requirement, unless otherwise specified in the contract documents, is to design high early strength concrete for pavement at a minimum of one of the contractor’s standard deviations above 2,400 psi at 24 hours. The second requirement is that the repair must be opened to traffic when a minimum flexural strength of 380 psi or a minimum compressive strength of 1,800 psi is achieved. Finally, an accelerating admixture may be used to expedite opening to traffic. A non-chloride accelerator must be used when the concrete under repair contains reinforcing steel;

otherwise, KDOT recommends adding 1% or 2% calcium chloride by dry weight of cement depending on cure time. A liquid membrane-forming compound is used for PCCP patch curing at a rate of 1 gallon per 100 sq. ft. applied to the finished patch (KDOT, 2015).

### *2.3.2 Illinois (2016 Standard Specifications)*

The Illinois Department of Transportation (IDOT) has developed five classes, labeled Pavement Patching (PP) 1–5, of concrete mix design criteria that depend on strength developed over a certain period. PP-5 must be used for a minimum compressive strength of 3,200 psi or a flexural strength of 600 psi at 4 hours. PP-5 consists of 675 lb/yd$^3$ of calcium aluminate cement, a w/c ratio of 0.32–0.40, slump ranging from 2 to 8 in., and air content of 4%–6%. A non-chloride accelerator, high range water reducer, and air-entraining agent from a qualified product list must be used for PP-5, and a mobile Portland cement concrete plant must be used to produce the patching mixture. If the same compressive strength and flexural strength is obtained at 8 hours instead of 4 hours, PP-4 mix design criteria must be used. PP-4 requires rapid-hardening cement (from the department's qualified product list) with a cement content ranging from 600 to 625 lb/yd$^3$. The PP-4 mix design also specifies a w/c ratio of 0.32–0.50, slump ranging from 2 to 6 in., and air content of 4%–6%. This mix design contains a high-range water reducer (HRWR) and an air entrainer. A retarder can be used to allow for haul time, but a retarder should not be used if a mobile Portland cement concrete plant is set up. PP-3 mix design criteria have the same compressive and flexural strength requirements as PP-4 and PP-5, but these strengths can be reached in 16 hours. The mix design criteria for PP-3 stipulates use of 735 lb/yd$^3$ of type III Portland cement, 100 lb/yd$^3$ of ground granulated blast-furnace slag, 50 lb/yd$^3$ of silica fume, a w/c ratio of 0.32–0.35, slump from 2 to 4 in., and air content of 4%–6%. If the air temperature is greater than 85$^o$F, then the Type III Portland cement can be replaced with Type I or II. PP-2 requires cement content ranging from 735 to 820 lb/yd$^3$ of Type I or II Portland cement, a w/c ratio of 0.32–0.38, slump from 2 to 6 in., and air content of 4%–6%. The compressive and flexural strength requirements are identical to the other classes; however, PP-2 has 24 hours to reach those strengths. A calcium nitrite-based non-chloride accelerator, HRWR, and air-entraining agent are used for classes PP-3 and PP-2. The accelerator should only be used if the air temperature is less than 55$^o$F for PP-2 concrete. PP-1 requires cement content between 650 and 750 lb/yd$^3$ of Type III Portland cement, a w/c ratio of 0.32–0.44, slump from 2 to 4 in., and air content of 4%–7%. The 3,200 psi in compressive strength or 600 psi in flexural strength can be reached in 48 hours. A calcium nitrite accelerator can be used with an air-entraining admixture if the air temperature is less than 55$^{o}$F for Class PP-1, but use of the accelerator must be clearly stipulated in the contract, and it must contain 1–2 quarts of solution per 100 pounds of cement for PP-1. If a calcium chloride accelerator is specified in the contract for PP-2, then 1.3–2.6 quarts per 100 pounds of cement is required. The calcium chloride used for Portland cement concrete patching must also be in liquid form with a minimum concentration of 32% (IDOT, 2016).

### *2.3.3 Iowa (2012 Standard Specifications)*

Iowa standard specifications for highway and bridge construction allow use of Type I or II Portland cements, Portland-Pozzolan cement, or Portland blast-furnace slag cement for pavement patching; Type III Portland cement can only be used for precast and prestressed concrete (Iowa DOT, 2012). The average cement content of these cement types is 825 lb/yd$^3$. Opening-to-traffic compressive strength ranges from 2,500 to 4,000 psi depending on the size and depth of repair, and depending on ambient temperature, strength requirements are reached in approximately 12–28 hours. Iowa reports a maximum w/c ratio of 0.4 for high early strength mixes, and maximum slump prior to the addition of calcium chloride solution should be 3 in. (Ghafoori et al., 2017). If a calcium chloride solution is not added, the maximum slump is 4 in., but the maximum slump of the mix prior to the addition of calcium chloride is 5 in. if a mid-range water reducer is used. The air content should be 5 ± 2% when calcium chloride is added and 6.5 ±1.5% when no calcium chloride is added. Iowa also stipulates that admixtures should not contain more than 1% chloride ions unless approved by the engineer, and the water in the calcium chloride solution should not be included when calculating the w/c ratio. The use of a commercial 32% calcium chloride solution is also suggested (Iowa DOT, 2012).

### *2.3.4 Nebraska (2007 Standard Specifications)*

The Nebraska Department of Transportation (NDOT) specifies that high early concrete pavement repairs must achieve a compressive strength of 3,000 psi before opening to traffic. If the ambient air temperature is above 60$^{o}$F, the minimum time before opening to traffic is 4 hours; however, if the ambient air temperature is between 41$^{o}$F and 60$^{o}$F, a curing time of 8 hours is expected before opening the repair to traffic (NDOT, 2007). Type III Portland cement is allowed for high early strength repairs; typical cement content is 799 lb/yd$^3$. Nebraska has a maximum w/c ratio of 0.45 and an air content ranging from 6% to 8.5% (Ghafoori et al., 2017). A 32% solution of calcium chloride is allowed for high early strength concrete and should be added just prior to placement at a rate of 0.375 gallons per 100 pounds of cement. Air-entraining agents and HRWRs are allowed, but the maximum allowable slump after the addition of all admixtures is 7 in. (NDOT, 2007).

### *2.3.5 American Concrete Pavement Association*

In 1995, the American Concrete Pavement Association (ACPA) published a guideline for full-depth repairs. In this guideline, typical strengths for opening to traffic, ranging from 2,000 to 3,000 psi in compression strength and 250 to 490 psi in flexural strength, depend on the repair length and the slab thickness. For Type III Portland cement with calcium chloride mixtures, a compressive strength of 2,000 psi can be reached 4–6 hours after placement. The guideline also recommends use of 1% calcium chloride (by dry weight of cement) when air temperatures exceed 80$^{o}$F, potentially requiring up to 2% when the air temperature is lower. For finishability, the slump in should be 2–4 in. Typical air contents range between 4.5% and 7.5% but can vary depending on patch location and maximum size of coarse aggregate used (ACPA, 1995).

### *2.3.6 Federal Highway Administration*

The Federal Highway Administration (FHWA) also has guidelines for full-depth concrete repair. They stipulate cement content between 658 and 846 lb/yd$^3$ for Type I or III Portland cement for typical full-depth repairs. The opening-to-traffic time of these repairs is 4–6 hours if a set accelerator is used; otherwise, opening to traffic is extended to 12–72 hours. The FHWA requires

6.5 ± 1.5% air content and a slump of 4–6 in. for full-depth repairs regardless of the mix design used. Guidelines for calcium chloride as an accelerator identically correspond to the guidelines from ACPA (FHWA, 2017).

## 2.4 General Requirements for High Early Strength Concrete

The University of Nevada at Las Vegas (UNLV) conducted a national survey from 2015 to 2016 to obtain information on high early-age strength repairs used in various DOTs. Eighteen DOTs responded to the survey, including KDOT. Survey results indicated that all responding states use high early strength concrete for pavement repairs. The major findings are summarized below.

### *2.4.1 Opening Times*

Opening-to-traffic times vary depending on the cement type used and geographic location of the repair. Opening times can be as low as 4–6 hours after pour when Type III Portland cement is used. Some DOTs also use proprietary cements such as Rapid Set, which can significantly decrease opening time to 2.5 hours. Opening times also depend on compressive or flexural strength requirements (Ghafoori et al., 2017).

### *2.4.2 Strength Requirement*

Compressive strength requirements range from 1,500 to 3,500 psi, with 3,000 psi being the most common minimum compressive strength when a repair is opened to traffic. Although the flexural strength requirement is not common, when required, the range is 380–600 psi (Ghafoori et al., 2017).

### *2.4.3 Drying Shrinkage*

Drying shrinkage requirements are also uncommon, but they range between 0.03% and 0.05% 28 days after placement when specified. Currently, Kansas has no drying shrinkage requirement (Ghafoori et al., 2017). However, the Illinois Tollway stipulates a length change requirement for high early strength cast-in-place concrete: the measured shrinkage must not be greater than 0.05% after 28 days of air drying when specimens are wet cured for 14 days prior to air drying (Illinois Tollway, 2013).

### *2.4.4 Cement Type*

High early strength concrete used for repairs may contain Type I, II, or III Portland cement, and the use of proprietary mixes is allowed in some states. Out of the 18 DOTs that responded to UNLV's survey, only Arkansas, Hawaii, and Iowa did not identify Type III Portland cement as the typical cement used by the agency. Alabama, Florida, Idaho, Illinois, and Virginia do not use proprietary mixes for high early strength concrete (Ghafoori et al., 2017).

### *2.4.5 Cement Content*

The cement content for pavement and bridge deck repairs using high early strength concrete is 600–900 lb/yd$^3$, with low pavement values ranging from 600 to 750 lb/yd$^3$ (Ghafoori et al., 2017).

### *2.4.6 Water-to-Cement Ratio*

The UNLV survey revealed that most states do not specify minimum w/c ratios, although some specify a maximum value. The maximum w/c ratio ranges from 0.40 to 0.45 (Ghafoori et al., 2017).

### *2.4.7 Air Content*

Air content control is essential in regions that have freeze-thaw weather cycles. States with freeze-thaw susceptibility have an air-content requirement between 5% and 8.5% (Ghafoori et al., 2017).

## 2.5 Origins of High Early Strength Concrete Opening-to-Traffic Criteria

High early strength concrete is commonly used for repairs because it allows the pavement to be opened to traffic as early as possible. DOTs determine the opening-to-traffic time by the repair strength gain rate at early ages. Although opening time is linked to strength gain, a 6-hour curing period is typically sufficient to prevent patch damage from traffic. In 1975, Joseph Ross, a concrete research engineer from the Louisiana Department of Highways, suggested that a maximum 6-hour repair curing is sufficient for a construction crew to complete a repair within an 8-hour shift (Ross, 1975).

Laboratory studies conducted in 1985 showed that 6-hour compressive strength of at least 2,000 psi is possible for mixes containing Type III Portland cement and 2% calcium chloride. However, these mixes had a high cement content with excessive shrinkage, and the slumps were so low that mixability, workability, and finishability were all negatively impacted. However, water-reducing admixtures, which are now common in high early strength concrete mix designs, were not mentioned in these studies. Field studies also revealed that 6-hour 2,000 psi compressive strength should not be a minimum range because strength gained within less than 6 hours was sufficient to prevent raveling, abrasion, deformation, and cracking when high early strength mixes were opened to traffic (Parker & Shoemaker, 1991).

## 2.6 Strategic Highway Research Program Study of High Early Strength Concrete

In 1993, the SHRP conducted a national study of high-performance concrete, or concrete with high early strength and durability against freezing and thawing. They focused on high early strength concrete as reported in Volume 2 SHRP-C-362, Volume 3 SHRP-C-363, and Volume 4 SHRP-C-364. The objective was to investigate the mechanical behavior and test high-performance concrete under prevailing field conditions. SHRP subdivided high-performance concrete based on cement type, minimum compressive strength, maximum w/c ratio, and minimum frost durability factors. High-performance concrete with Type III Portland cement and 2,000 psi compressive strength in 6 hours is the focus of this literature review on the SHRP study. The criterion of 2,000 psi compressive strength in 6 hours was chosen because, after repeated trials, SHRP could not effectively produce Portland cement-based concrete with 3,000 psi compressive strength at 4 hours with acceptable performance characteristics without calcium chloride (Zia et al., 1993). SHRP determined compressive strength using 4 x 8-in. cylinders, and the minimum durability factor of 80% was established after 300 cycles of freezing and thawing per ASTM C666 procedure A.

The SHRP study considered dense crushed limestone aggregate from West Fork, Arkansas, with Arkansas native sand, to be the closet aggregate type for Kansas. The coarse aggregate, supplied by McClinton-Anchor, met ASTM C33 #67 specifications and was composed of 97% limestone and 3% clay minerals. The fine aggregate, which was gathered from the Arkansas River, consisted of 85-62% quartz, 4-16% chert, 11% microcline, and 1%–5% rock fragments. The cement with the Arkansas aggregate was Type III Portland cement with low alkali content, and it was supplied by Blue Circle Cement, Inc. from Tulsa, Oklahoma. The chemical admixtures used in the SHRP research program included an accelerator called Darex Corrosion Inhibitor (DCI) (calcium nitrite-based) and an air-entraining admixture (vinsol resin-based), both supplied by W. R. Grace, as well as two HRWRs (melamine- and naphthalene-based) and a retarder (lignin-based). Cormix supplied the HRWRs and retarder (Zia et al., 1993).

Only 7 of the 35 Arkansas mix design trials that contained Type III Portland cement achieved at least 1,800 psi compressive strength, and three of those seven trial mixes demonstrated poor workability with w/c ratios of 0.34. All trial mixes had a cement content of 870 lb/yd$^3$ and no additional retarder; instead, mixtures contained an HRWR, an accelerator, and an air-entraining agent. All trial mixes had an air content ranging from 5% to 6.8%, and insulation was used to trap hydration heat. No further data on shrinkage or freeze-thaw tests for these concrete mixes were available.

The SHRP study concluded that compressive strengths of 2,000 psi can be obtained in 6 hours using Type III Portland cement. However, care should be taken since the rate of hydration is expedited due to a higher cement content and a lower w/c ratio. For durability, frost resistant, high early strength concrete should contain at least 5% entrained air even with a very low w/c ratio, thereby allowing concrete to meet the stringent durability factor requirement of 80% after 300 cycles of freezing and thawing according to ASTM C666 procedure A. However, no data collected during the Arkansas Type III Portland cement trial mixes affirmed this assertion (Zia et al., 1993).

## 2.7 Factors Affecting Concrete Durability

Durable concrete should maintain its strength and serviceability throughout its service life and withstand the deterioration process due to exposure (Neville, 1996). Factors affecting concrete durability include drying shrinkage, rapid freezing and thawing, excessive scaling, and permeability. Because concrete deterioration is rarely due to a single factor, the broader definition of durability is the overall quality of the concrete (Neville, 1996).

### *2.7.1 Drying Shrinkage*

Excess water left over from the hydration of concrete evaporates with time, leading to inevitable volume reductions. Excessive drying shrinkage causes severe cracking that hinders concrete durability due to water and chloride penetration (Almudaihem & Hansen, 1987). External and internal factors affect drying shrinkage. External factors include ambient conditions and size or shape of the concrete sample. High ambient temperature, low relative humidity, and increased air movement around the concrete cause extensive drying shrinkage. Geometry of the member also significantly impacts drying shrinkage. For example, a large, thick concrete sample will have less shrinkage than a small, thin sample for the same drying period.

Internal factors that affect drying shrinkage include concrete mix design and placing/curing of concrete. Aggregates in a concrete mix can effectively restrain or reduce the effects of shrinkage. Good quality, well graded, low absorptive aggregates minimize drying shrinkage. Depending on their formulation, admixtures in a concrete mix can also affect the drying shrinkage of concrete (Cement Concrete & Aggregates Australia, 2002). Many previous studies have established that the use of calcium chloride increases the drying shrinkage of concrete (Torrans, Ivey, & Hirsch, 1964) (Ramachandran, 1978) (Cement Concrete & Aggregates Australia, 2002). Water content in a mixture also affects drying shrinkage because shrinkage increases with increased water content due to the evaporation of additional water intended for workability. The concrete mixture must be properly consolidated during construction to produce a dense concrete with discontinuous capillaries that reduce moisture loss. Application of appropriate curing procedures is essential before the concrete surface is dry. Drying shrinkage cracks are problematic when the concrete is unable to move, as in a concrete patch repair. Therefore, adequate mix designs that control excessive dry shrinkage are vital to produce durable concrete pavement (Cement Concrete & Aggregates Australia, 2002).

### *2.7.2 Excessive Permeability*

The degree of concrete permeability, or penetration of various ions, liquids, and gases, can directly affect the rate of concrete deterioration (Basheer et al., 2001). Studies have generally indicated that concrete with low permeability tends to be more durable because it more effectively resists chemical attacks and freeze-thaw deterioration (Ramezanianpour et al., 2011). Pore size distribution and continuity in the cement paste not only directly affect the permeability of concrete, but they also dictate the extent of damage caused by freeze-thaw cycling. Capillary voids larger than 50 macropores can also negatively affect the rate of deterioration in concrete pavements because water existing in those pores expands and contracts due to freezing and thawing, resulting in potential internal cracking (Basheer et al., 2001).

KDOT standard specifications include guidelines to meet permeability requirements. One way to decrease permeability is to decrease the paste of a concrete mixture and increase the coarse aggregate portion during the mix design process. Mixtures with more than 60% coarse aggregate retained on the U.S. No. 8 sieve and with a fineness modulus above 4.75 have been shown to be less permeable because water and chlorides can easily permeate the mortar and paste but cannot readily permeate through coarse aggregates.

KDOT recommends a w/c ratio less than 0.43 to meet permeability specifications; concrete mixtures with less than 25% paste generally meet those requirements. High early strength concrete mixes with Type III cement can meet permeability requirements because of their low w/c ratios and the use of specific guidelines (KDOT, 2007).

### *2.7.3 Excessive Scaling*

Concrete scaling occurs when repeated freeze and thaw cycles cause the loss of surface layers of cement and expose coarse aggregates over time (Afrani & Rogers, 1994). Although scaling is a surface phenomenon, it is the most apparent form of deterioration in concrete (Pigeon et al., 1986). Salt scaling, occasionally described as superficial damage, is common on concrete pavements that use de-icing salts in cold climates (Valenza & Scherer, 2006). This damage is progressive and capable of reducing the mechanical integrity, thereby increasing the need for expensive repair or replacement of concrete pavements. Salt scaling accelerates the ingression of aggressive chemicals, such as chlorides, and promotes a high degree of saturation in concrete pavements, which causes strength loss due to internal frost action. Maximum damage can occur with moderate amounts of salt; in fact, a 3% salt solution will achieve maximum damage regardless of the solute used (Valenza & Scherer, 2006).

### *2.7.4 Rapid Freezing and Thawing*

Internal cracking of concrete is the main form of deterioration in most frost-susceptible concrete. This type of deterioration can be replicated by exposing concrete specimens to freeze-thaw cycles and monitoring length change and RDME (Pigeon et al., 1986). ASTM C666 requires RDME but classifies length change measurements as optional indications of durability (ASTM, 2015a). RDME can be measured via a nondestructive test method that uses an accelerometer and a modally-tuned impact hammer. RDME is related to the resonant frequency, also known as natural frequency or fundamental transverse frequency. As the resonant frequency decreases so does the RDME. Resonant frequency is measured as the frequency at which the greatest amplitude is reached when a mechanical wave is induced through a hammer impact. Internal micro-cracking deterioration can occur if the concrete is not adequately air entrained during freeze and thaw cycling. An increase in internal micro cracks leads to increased damping in the mechanically-induced wave, thus decreasing the resonant frequency. The resonant frequency is also reduced in concrete that contains cracks because waves require more time to travel through a cracked medium (FHWA, 2006).

## 2.8 Durability Testing Methods

Common testing mechanisms to monitor the factors affecting concrete durability mentioned in Section 2.7 include ASTM C157 and ASTM C666 or KTMR-22, ASTM C672, and KT-79.

### *2.8.1 ASTM C157: Length Change of Hardened Hydraulic Cement Mortar and Concrete*

ASTM C157 standard *Length Change of Hardened Hydraulic-Cement Mortar and Concrete* measures concrete strain due to drying shrinkage. This test, commonly referred to as the free shrinkage test, quantifies potential volumetric changes, such as expansion or contraction of concrete, without the effects of load or temperature change.

The procedure for this standard consists of casting at least three rectangular prism specimens for each test condition. The concrete prisms must be cast in layers, and they are consolidated by either rodding or external vibration. The specimens are cured in a moist room for 24 hours and then removed from the molds and placed in lime-saturated water maintained at 73 ± 1°F for a minimum of 30 minutes. The lime-saturated water allows an accurate length reading without significant variation due to temperature. After immersion in lime-saturated water, the specimens are wiped with a damp cloth, and immediately the initial comparator reading is taken. Length change is then calculated by dividing the difference between the specimen and a reference bar by the gage length. ASTM C157 requires lime-saturated curing for 28 days after the first comparator reading, followed by another comparator reading. Specimens are stored in a drying room. Comparator readings for the specimens are taken 4, 7, 14, and 28 days after final curing and after 8, 16, 32, and 64 weeks. The drying room should be kept at a temperature of 73 ± 3°F with a relative humidity of 50 ± 4% (ASTM, 2014a).

In 2012, a research group at KDOT studied the length change of hardened concrete using ASTM C157. They found that the free shrinkage test could be used in design or acceptance criteria for construction projects in which minimal cracking would be detrimental. They also concluded that, due to equipment and curing regimes needed for this test, private laboratories should conduct the test rather than contractors. Thus, ASTM C157 may be precluded from the acceptance requirements (Jenkins, 2016).

### *2.8.2 KT-79: Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration*

Concrete resistivity is a geometry-independent material property that can be correlated with degree of permeability. Resistivity is a function of moisture content and concrete composition. Studies have shown that zones with high permeability and consequently high chloride penetration have comparatively low resistivity. A nondestructive four-point Wenner array probe is commonly used to measure concrete resistivity. A current is applied to two outer probes, and the potential difference is measured between the two inner probes. The instantaneous measurements and ease of use make this test a promising alternative test for measuring chloride penetration potential (Ramezanianpour et al., 2011).

In January 2014, KT-79: *Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration* was added to KDOT standard specifications. This test method measures concrete permeability. Surface resistivity is usually recorded at 28 days because KDOT research indicated a strong correlation between this measurement and 56-day rapid chloride permeability test results. The procedure for KT-79 consists of the following (KDOT, 2012):

(a) Three 4 x 8-in. cylinders are cast at the time of mixing and cured in 100% humidity in the laboratory for 28 days;

(b) At the end of the curing period, each specimen is marked four times on the top of the circular face at 0-, 90-, 180-, and 270-degree points. Each mark on the circular face extends into the middle of the longitudinal sides of the specimens;

(c) Each cylinder is placed on its side, preferably in a moist placeholder. A Resipod Proceq meter is placed at the center of the long side of the specimen with the longitudinal maker centered between the two inner probes;

(d) Surface resistivity values are recorded at every 0-, 90-, 180-, and 270-degree points twice for a total of eight readings for each of the three specimens; and

(e) Average resistivity and the percent relative standard deviation for each specimen are calculated. If the percent relative standard deviation is greater than 7.5%, the specimen is fully immersed in water for 2 hours before retesting. If the new surface resistivity values produce a relative standard deviation less than 7.5%, then those values are used to calculate the average surface resistivity. Otherwise, all 16 values are used to calculate the average surface resistivity of the specimens.

Jenkins (2015) compared the American Association of State Highway and Transportation Officials (AASHTO) rapid chloride permeability changes to equivalent KDOT preliminary surface resistivity design values. Comparison results indicated high chloride ion permeability with surface resistivity values less than 7 kΩ-cm, moderate chloride ion permeability of 7–13 kΩ-cm, low chloride ion permeability of 13–24.3 kΩ-cm, very low chloride ion permeability of 24.3–191 kΩ-cm, and negligible chloride ion permeability for values greater than 191 kΩ-cm (Jenkins, 2015). KDOT standard specifications now stipulate a minimum surface resistivity value for air-entrained concrete pavements in order to maintain low permeability in concrete and thus ensure long-term durability.

### *2.8.3 ASTM C672: Scaling Resistance of Concrete Surfaces Exposed to De-icing Chemicals*

ASTM C672 *Scaling Resistance of Concrete Surfaces Exposed to De-icing Chemicals,* developed in 1971, is a standard durability test conducted on concrete (Saric-Coric & Aitcin, 2002). This test utilizes visual examination to evaluate the surface resistance of concrete exposed to freeze-thaw cycles. The two specimens in this test must have a minimum surface area of 72 sq. in. ($in^2$) and be at least 3 in. deep. Specimen fabrication includes filling the mold in one layer, rodding one time for every 2 $in^2$ of surface, and then surface leveling with a flat trowel. After the concrete has stopped bleeding, the surface is prepared with three sawing-motion passes of a wood strike-off board. The surface is then brushed with a brush of medium stiffness. The specimens must have a dike approximately 0.75 in. high for storing water with calcium chloride (ASTM, 2012a).

Specimens for this test are cured for 14 days in a moist room, followed by 14 days in an air-drying environment at $73.5 \pm 3.5^{o}$F and $50 \pm 5\%$ relative humidity. After curing, the specimens are covered with approximately 0.25 in. of calcium chloride water with a concentration of 4 grams of anhydrous calcium chloride per 100 milliliters of water. The specimens are then ready to undergo freezing and thawing cycles in which they are placed in a freezing environment for 16–18 hours and then air-dried for 6–8 hours in a room with a temperature of $73.5 \pm 3.5^{o}$F and 50 ±

5% relative humidity. After every fifth cycle, the surface is flushed off, a visual examination is made, and the specimen surface is rated. ASTM C672 provides a rating condition table, as shown in Table 2-1 (ASTM, 2012a). Although similar European standards use comparable numbers of cycles, they also quantify the weight of the specimen after various number of cycles to determine the mass of debris that has scaled off (Valenza & Scherer, 2006). In Quebec, Canada, the maximum mass loss allowed after 56 cycles of freezing and thawing is 500 grams per meter squared (g/m$^2$) of exposed concrete surface area; in Sweden, maximum permitted mass loss is 1,000 g/m$^2$ after 56 cycles (Saric-Coric & Aitcin, 2002).

**Table 2-1: ASTM C672 Scaling Rating**

| Rating | Condition of Surface |
|---|---|
| 0 | No scaling |
| 1 | Very slight scaling, no coarse aggregate visible |
| 2 | Slight to moderate scaling |
| 3 | Moderate scaling, some coarse aggregate visible |
| 4 | Moderate to severe scaling |
| 5 | Severe scaling, coarse aggregate visible over entire surface |

### *2.8.4 ASTM C666: Resistance of Concrete to Rapid Freezing and Thawing and KTMR-22*

ASTM C666 *Resistance of Concrete to Rapid Freezing and Thawing* is a standardized internal cracking durability test consisting of two procedures. In procedure A, specimens are continually surrounded by water, while procedure B freezes the specimens in air and thaws them in water. Procedure A is more conservative in determining concrete durability because the specimens are kept at a constant saturation; however, procedure B more realistically simulates field saturation patterns. Each freezing and thawing cycle alternately decreases the temperature at the center of the specimen from 40°F to 0°F and then increases it in 2–5 hours. The curing regime for ASTM C666 consists of 14 days in saturated limewater prior to beginning the freeze-thaw cycles. Mass, resonant frequency of vibration, and length change are recorded for each concrete specimen at a given interval. The collected data are then translated to an RDME value and percent expansion. The specimens undergo freeze-thaw cycles until 300 cycles have been completed, the RDME reaches 60% of the initial modulus, or the specimens reach a 0.10% expansion, whichever comes first (ASTM, 2015a).

KDOT has adopted their own version of ASTM C666 that more accurately represents concrete freeze-thaw cycles in Kansas weather. Test method KTMR-22, which originated in the 1980s, modifies the ASTM C666 curing regime for procedure B. The test method utilizes three 3 x 4 x 16-in. rectangular concrete prisms that are taken through a 90-day curing period consisting of 67 days in 100% humidity curing environment, 21 days in a drying room at 73$^{o}$F and 50% relative humidity, 1 day in a tampering tank at 70$^{o}$F, and 1 day in ice water at 40$^{o}$F. At the end of this curing period, the specimens undergo the freeze-thaw cycles at a rate of approximately 8 cycles per day. Mass, resonant frequency of vibration, and length change are recorded at intervals not exceeding 36 cycles up to 660 cycles, when the RDME reaches 95%, or when the specimens reach a 0.025% expansion, whichever occurs first. The increase from 300 cycles to 660 cycles was based on a 20-year material design life in which the average Kansas annual number of freeze-thaw cycles of 33 was multiplied by 20 years (Distlehorst, 2015).

## 2.9 Summary

High early strength concrete is an essential material for repair of transportation facilities serving high-traffic volumes. This literature review evaluated methods to obtain high early strength concrete and current practices utilized by various state highway agencies for high early strength concrete mixtures. This section also reviewed requirements throughout the United States, as well as the origins of current opening-to-traffic criteria for high early strength concrete. The national SHRP high performance concrete study, published in 1993, was critically examined, and durability factors known to affect high early strength concrete and common test methods for testing durability were presented.

# Chapter 3 - Materials

This chapter reviews materials used in this study. The materials include aggregates, Type III Portland cement, and various admixtures.

## 3.1 Aggregates

Coarse (CPA-4) and fine (FA-A) paving aggregates from Cornejo & Sons in Wichita, Kansas, were collected for this research. CPA-4 aggregate for KDOT may include siliceous gravel, chat, or crushed stone; Cornejo aggregates were crushed stone. Fine aggregates used for concrete include naturally occurring sand resulting from the disintegration of siliceous/calcareous rock or manufactured sand produced by crushing siliceous materials (KDOT, 2015). Figure 3-1 and Figure 3-2 depict coarse and fine aggregates, respectively. Aggregate properties such as saturated surface dry (SSD) bulk specific gravity, absorption, and gradations are essential for concrete mix design proportioning. These values were obtained in accordance with Kansas Test Method KT-6 *Specific Gravity and Absorption of Aggregates* and KT-2 *Sieve Analysis of Aggregates*.


**Figure 3-1: District V Coarse Aggregate**

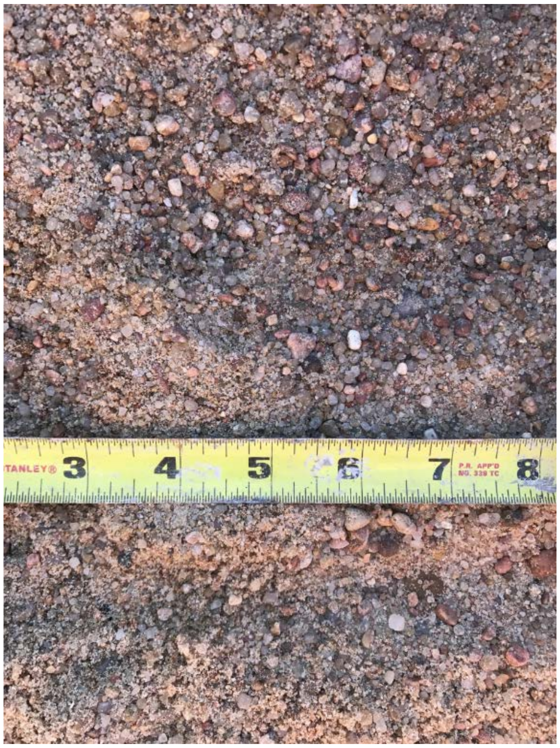


**Figure 3-2: District V Fine Aggregate**

### *3.1.1 Coarse Aggregate Properties*

A sample of coarse aggregate in accordance with KT-6 was washed thoroughly over a U.S. No. 4 sieve to remove any dust that adhered to the aggregates. The sample was then oven-dried at a temperature of 230 ± 9$^{o}$F, immersed in water, and soaked for 24 hours. The sample was brought to SSD condition by rolling it in a damp towel. The aggregates reached SSD condition when they appeared to be moist but not shiny. The weight of the sample in SSD condition was recorded, as well as the aggregate's mass in water and the oven-dried mass of the sample after an additional 24 hours in the oven. The SSD bulk specific gravity was then determined by dividing the SSD mass in air by the difference between the SSD mass in air and the SSD mass in water. Absorption of the coarse aggregate was determined by dividing the difference between the SSD mass in air and the oven-dried mass by the oven-dried mass (KDOT, 2007). The coarse aggregate properties for District V are shown in Table 3-1.

**Table 3-1: District V Coarse Aggregate Properties**

| Producer | SSD Bulk Specific Gravity | Absorption (%) |
|---|---|---|
| Cornejo & Sons | 2.61 | 2.44 |

For gradation analysis of coarse aggregates, a sample size per KT-2 test method was oven-dried to a constant mass at a temperature of 230 ± 9°F, and the original dry mass was recorded. Then the sample was passed through a series of sieves in decreasing sizes, and the mass retained on each sieve was measured and recorded. The percent passing was then calculated based on the cumulative percent retained (KDOT, 2007). Table 3-2 shows the coarse aggregate size distribution, and Table 3-3 compares KDOT standard gradations for CPA-4 according to the 2015 standard specifications to District V coarse aggregate. The gradation curve for District V coarse aggregate is also shown in Figure 3-3.

**Table 3-2: Coarse Aggregate Sieve Distribution**

| Coarse Aggregate: CPA-4 | | | | | |
|---|---|---|---|---|---|
| Sieve Size | | Remaining in sieve (g) | % Retained | Cumulative % Retained | % Passing |
| in. | mm | | | | |
| # 1 | 25.4 | 0.0 | 0.0 | 0.00 | 100.0 |
| # 3/4 | 19 | 403.8 | 8.1 | 8.1 | 92 |
| # 1/2 | 12.7 | 1409.8 | 28.2 | 36.3 | 64 |
| # 3/8 | 9.5 | 830.1 | 16.6 | 52.9 | 47 |
| # 4 | 4.8 | 1759.1 | 35.2 | 88.1 | 12 |
| # 8 | 2.4 | 310.9 | 6.2 | 94.3 | 6 |
| Pan: | | 282.8 | 5.7 | 100.0 | 0 |
| Sum: | | 4996.5 | 100.0 | - | - |

**Table 3-3: Comparison of KDOT CPA-4 Standard Gradation and District V CPA-4 Gradation**

| Coarse Aggregate: CPA-4 | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Type | Percent Retained-Square Mesh Sieves | | | | | | | |
| | 1 1/2" | 1" | 3/4" | 1/2" | 3/8" | No. 4 | No. 8 | Pan |
| KDOT: CPA-4 | - | - | 0-20 | - | - | - | 95-100 | - |
| District V: CPA-4 | - | - | 8 | 36 | 53 | 88 | 94 | 100 |
| | | | Pass | | | | Slightly Less | |

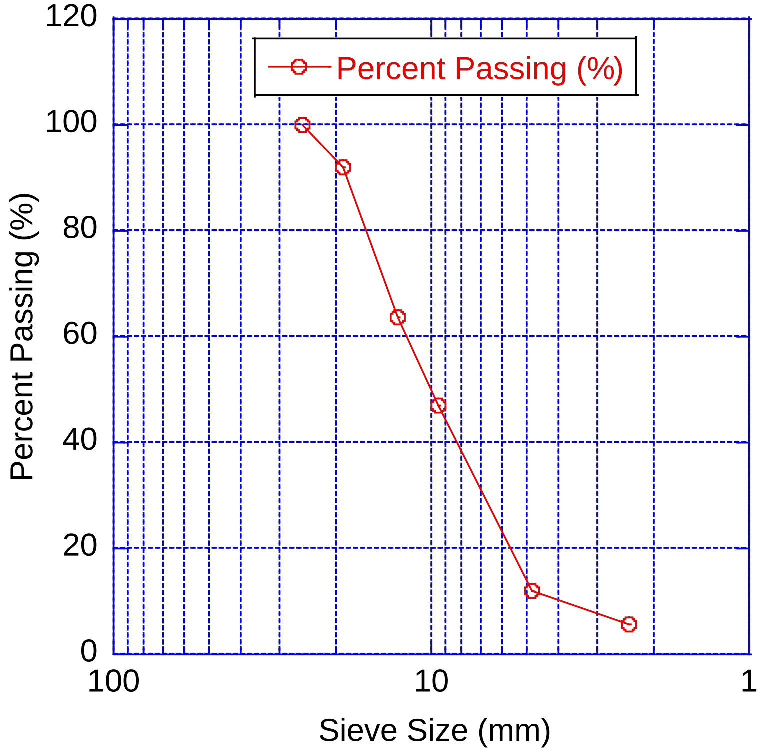


**Figure 3-3: District V Coarse Aggregate (CPA-4) Gradation**

### *3.1.2 Fine Aggregate Properties*

Approximately 1,000 grams of fine aggregate were washed over the U.S. No. 100 sieve to remove dust particles and then oven-dried to a constant mass. The sample was then soaked for 24 hours in water and brought to SSD condition using two slightly rusted drying pans. The sample was continuously transferred from one pan to another until all free moisture was gone, as indicated by the coloration of the rust at the bottom of the pan. The SSD bulk specific gravity was determined using a pycnometer, water, and various mass measurements of sand in SSD condition. Absorption was measured in a similar fashion to coarse aggregate absorption (KDOT, 2007). Fine aggregate properties from District V are shown in Table 3-4.

**Table 3-4: District V Fine Aggregate Properties**

| Producer | SSD Bulk Specific Gravity | Absorption (%) |
|---|---|---|
| Cornejo & Sons | 2.62 | 0.42 |

With the exception of different sieve sizes and use of a mechanical shaker, the same procedure used for coarse aggregates was followed to determine the gradation of fine aggregates. Fine aggregate size distribution is outlined in Table 3-5, and Table 3-6 compares KDOT FA-A gradation per the 2015 standard and District V FA-A. Gradation for District V fine aggregate is shown in graphical form in Figure 3-4.

**Table 3-5: Fine Aggregate Sieve Distribution**

| Fine Aggregate: FA-A | | | | | |
|---|---|---|---|---|---|
| Sieve Size | | Remaining in sieve (g) | % Retained | Cumulative % Retained | % Passing |
| in. | mm | | | | |
| # 4 | 4.8 | 39.2 | 7.9 | 7.9 | 92 |
| # 8 | 2.4 | 93.6 | 18.8 | 26.7 | 73 |
| # 16 | 1.2 | 109.2 | 21.9 | 48.6 | 51 |
| # 30 | 0.6 | 98.9 | 19.9 | 68.5 | 32 |
| # 50 | 0.3 | 104.0 | 20.9 | 89.4 | 11 |
| # 100 | 0.1 | 45.5 | 9.2 | 98.6 | 1 |
| Pan: | | 6.7 | 1.4 | 100.0 | 0 |
| Sum: | | 497.1 | 100.0 | - | - |

**Table 3-6: Comparison of KDOT FA-A Standard Gradation and District V FA-A Gradation**

| Fine Aggregate: FA-A | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Type | Percent Retained-Square Mesh Sieves | | | | | | | |
| | 3/8" | No. 4 | No. 8 | No. 16 | No.30 | No. 50 | No. 100 | Pan |
| KDOT: FA-A | 0 | 0-10 | 0-27 | 15-55 | 40-77 | 70-93 | 90-100 | 98-100 |
| District V: FA-A | - | 8 | 27 | 49 | 69 | 89 | 99 | 100 |
| | | Pass | Pass | Pass | Pass | Pass | Pass | Pass |

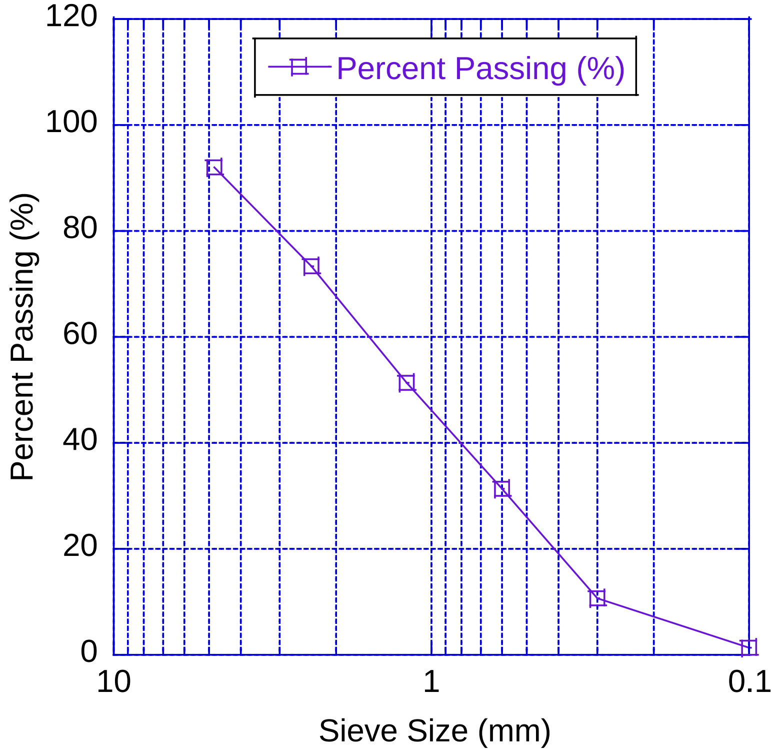


**Figure 3-4: District V Fine Aggregate (FA-A) Gradation**

## 3.2 Cement

Type III Portland cement, manufactured by Monarch, Inc. of Humboldt, Kansas, was used in all concrete mixes in this study. Physical and chemical compositions of the cement are shown in Table 3-7.

**Table 3-7: Mill Test Results for Monarch Type III Portland Cement**

| Property | | Reported Value | Spec Limit |
|---|---|---|---|
| **Physical Properties** | | | |
| 325 Sieve, % Passing | | 99.7 | - |
| Blaine fineness, specific surface – Air Permeability ($cm^2/g$) | | 5,860 | - |
| Time of Setting, Gilmore Test: | Initial (hrs:min) | 1:40 | 60 min |
| | Final (hrs:min) | 2:35 | 600 max |
| Air Content of Mortar (volume %) | | 7.0 | 12.0 max |
| Autoclave Expansion (%) | | -0.014 | 0.80 max |
| Compressive Strength (psi) | 1 Day | 3,709 | 1,740 min |
| | 3 Days | 5,019 | 3,480 min |
| | 7 Days | 5,564 | - |
| **Chemical Properties** | | | |
| $SiO_2$ – Silicon Dioxide (%) | | 21.20 | - |
| $Fe_2O_3$ – Ferric Oxide (%) | | 2.89 | 6.0 max |
| $Al_2O_3$ – Aluminum Oxide (%) | | 4.06 | 6.0 max |
| CaO – Calcium Oxide (%) | | 64.15 | - |
| MgO – Magnesium Oxide (%) | | 1.53 | 6.0 max |
| $SO_3$ – Sulphur Trioxide (%) | | 2.79 | 3.5 max |
| Loss on Ignition (%) | | 0.85 | 3.0 max |
| Insoluble Residue (%) | | 0.22 | 1.50 max |
| Free Lime (%) | | 1.29 | - |
| $Na_2O$ – Sodium Oxide (%) | | 0.17 | - |
| $K_2O$ – Potassium Oxide (%) | | 0.53 | - |
| Equivalent Alkalies (%) | | 0.51 | 0.60 max |
| **Potential Calculated Compounds** | | | |
| $C_3S$ – Tricalcium Silicate (%) | | 60.7 | - |
| $C_2S$ – Dicalcium Silicate (%) | | 15.0 | - |
| $C_3A$ – Tricalcium aluminate (%) | | 5.9 | 8 max |
| $C_4AF$ – Tetracalcium aluminoferrite (%) | | 8.8 | - |

## 3.3 Air Entrainment

Daravair® 1400 from GCP Applied Technologies was the air-entraining agent used for all District V concrete mixes in this study. Although the product data sheet indicates no standard addition rate for this air entrainment, typical addition rates range from 0.5 to 3 fl oz/cwt of cement (GRACE, 2007).

## 3.4 Water Reducer

ADVA® 140M from GCP Applied Technologies was the water reducer used for all District V concrete mixes in this study. The typical range is 2–20 oz/cwt of cement. This water reducer is a mid- and high-range water reducer; typical addition rates for mid-range are 5–9 oz/cwt of cement, while typical addition rates for high-range are 9–16 oz/cwt of cement (GRACE, 2007).

## 3.5 Accelerator

Liquid calcium chloride with 32% concentration (Scotwood Industries, Inc.) was used as an accelerator for all concrete mixes used in this study.

# Chapter 4 - Methods

This chapter presents the experimental methods used in this study, including concrete mix proportioning, trial mix designs, and full PCCP mix designs.

## 4.1 Concrete Mix Proportioning

The concrete mix designs in this project were developed to determine the effect of cement and calcium chloride quantities on high early strength concrete durability. The absolute volume method for proportioning concrete was used to create the concrete mix designs. This method consists of designing 1 cu. yd. of concrete based on the volume occupied by each constituent, including an assumed volume of entrapped air. The method requires a cement content and w/c ratio value to be set and then uses the specific gravity of each material to convert the masses into volumes. These volumes are subtracted from 1 cu. yd. and the remainder volume is distributed to the coarse and fine aggregates depending on the percent blend in the mix design and corresponding aggregate specific gravity. An Excel worksheet was created according to American Concrete Institute (ACI) absolute volume methodology and KDOT form number 694 to facilitate mix design calculations. A sample concrete mix design is shown in Figure 4-1.

Table 4-1 shows theoretical mix proportions per cubic yard based on the ACI absolute volume method with aggregates in SSD condition. The PCCP ID number and the amount of liquid calcium chloride used were based on Table 1-1. The w/c ratio was fixed at 0.37 for all PCCP mixtures, and the liquid calcium chloride was considered part of the mixing water. Aggregate weights were obtained by the absolute volume method, but dosages for the air-entraining and water-reducing admixtures were obtained through trial mixes until adequate fresh concrete properties and desired strength were achieved.

**ABSOLUTE VOLUME METHOD FOR ONE CUBIC YARD**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Cement Volume | $\frac{\text{Design CF X \% Cement}}{\text{S.G X 62.4}}$ | = | 564 / 3.15 | X / X | 100% / 62.243 | = | 2.8766 | cu. ft. |
| Water Volume | $\frac{\text{Design CF X Design w/c}}{62.4}$ | = | 564 X 0.37 / 62.243 | | | = | 3.3527 | cu. ft. |
| Air Volume (Entrained and Entrapped) | | | 6.50% | X | 27 | = | 1.7550 | cu. ft. |
| Absolute Volume of Cementitious, Water, Air, and Admixtures | | | | | | = | 7.9843 | cu. ft. |
| Absolute Volume of Aggregate Required | | | 27 | - | 7.98 | = | 19.0157 | cu. ft. |

Determine the Absolute Volume of 100 lb of Combined Aggregates

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Rock CPA-4 | $\frac{\text{\% Aggregate No. 1 X 100 lb}}{\text{S.G. No. 1 X 62.4}}$ | = | 50% / 2.61 | X / X | 100 / 62.243 | = | 0.3082 | cu. ft. |
| Sand FA-A | $\frac{\text{\% Aggregate No. 2 X 100 lb}}{\text{S.G. No. 2 X 62.4}}$ | = | 50% / 2.62 | X / X | 100 / 62.243 | = | 0.3067 | cu. ft. |
| | | | | | cu. Ft. | = | 0.6149 | cu. ft. |
| Total Aggregate Required for One Cubic Yard | | = | 100 X 19.02 / 0.6149 | | | = | 3092.64 | lbs. |

Batch Weights

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Cement | 564 | X | 100% | = | 564 | lbs. |
| | Water | 564 | X | 0.37 | = | 209 | lbs. |
| Rock | Aggregate No. 1 | 3092.64 | X | 50% | = | 1546 | lbs. |
| Sand | Aggregate No. 2 | 3092.64 | X | 50% | = | 1546 | lbs. |
| | Weight for One Cubic Yard | | | | = | 3865 | lbs. |
| | Concrete Unit Weight | | | | = | 143.16 | lbs./cu.ft. |
| | Air Free Unit Weight | | | | = | 153.11 | lbs./cu.ft. |

**Figure 4-1: Sample Mix Design Worksheet**

**Table 4-1: Concrete Mixture Proportions Based on Table 1.1 PCCP ID and $CaCl_2$ Dosage**

| PCCP ID | Mass (lbs/yd$^3$) | | | | Admixtures (oz/cwt of cement) | | Admixtures (oz/ yd$^3$) | | Accelerator Admixture (oz/yd$^3$) |
|---|---|---|---|---|---|---|---|---|---|
| | Cement | Water | Fine Aggregate (SSD) | Coarse Aggregate (SSD) | Daravair ® 1400 | ADVA ® 140M | Daravair ® 1400 | ADVA ® 140M | Liquid Calcium Chloride |
| 1 | 564 | 202.0 | 1546 | 1546 | 2.25 | 19.0 | 12.69 | 107.16 | 157.6 |
| 2 | 564 | 195.2 | 1546 | 1546 | 2.50 | 19.0 | 14.10 | 107.16 | 316.1 |
| 3 | 658 | 235.6 | 1462 | 1462 | 2.70 | 12.25 | 17.77 | 80.61 | 184.4 |
| 4 | 658 | 227.8 | 1462 | 1462 | 2.25 | 12.25 | 14.81 | 80.61 | 368.8 |
| 5 | 752 | 269.3 | 1377 | 1377 | 1.63 | 10.13 | 12.26 | 76.18 | 210.7 |
| 6 | 752 | 260.3 | 1377 | 1377 | 1.38 | 10.0 | 10.38 | 75.20 | 421.4 |

## 4.2 Trial Mix Designs

The main objective of this study was to design and create high early strength concrete mixtures that produce a minimum compressive strength of 1,800 psi in 6 hours and have acceptable slump and air content to resist freeze-thaw damage. High early strength concrete patching mixtures were studied, and several mix designs were created that varied in cement content and calcium chloride dosage (Table 1-1). Before conducting these full-scale mix designs, trial mixes were tested at small batches of 0.5 ft$^3$ for slump, air content, and compressive strength to obtain adequate air-entraining and water-reducing dosages that would produce the desired fresh concrete properties.

### *4.2.1 Trial Mix Desired Properties and Procedure*

Trial mixtures for each full-scale PCCP mix were mixed in a small Lancaster counter-current mixer, shown in Figure 4-2. The procedure for laboratory concrete mixing was in accordance with Section 2.2.4 Mixing Procedure for High Early Strength Concrete of this thesis. Desired fresh and hardened concrete properties in the trial mixes included slump (2.5–5 in.), air content (6.5 ± 1.5%), and minimum compressive strength at 6 hours (1,800 psi maximum).

**Figure 4-2: Lancaster Counter-Current Mixer**

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### *4.2.2 Slump*

The slump test was conducted on each trial mix according to ASTM C143 methodology. The procedure consisted of placing fresh concrete in a damp Abrams cone seated and clamped to the flat base in three layers of equal volume and rodding each layer 25 times while ensuring that the rod penetrated the layer below. Then the cone was lifted, and the vertical difference between the top of the cone and the center of the top surface of the displaced fresh concrete was recorded to the nearest 0.25 in. (ASTM, 2016a). A picture of the slump test equipment is shown in Figure 4-3.

**Figure 4-3: ASTM C143 Slump Test Equipment**

### *4.2.3 Air Content*

The air content test was conducted using the pressure method on each trial mix design in accordance with ASTM C231. The operational principle of this method is to equalize a known volume of air at a known pressure with an unknown volume of air in a concrete sample. The dial on the pressure gauge was calibrated to read the air percentage at which equalization occurs. The air content procedure consisted of placing fresh concrete in the damp air pressure meter bowl in three layers of equal volumes and rodding each layer 25 times, making sure that the rod penetrated the layer below. After the final, or third, layer, the excess concrete was removed, and the lip around the perimeter of the bowl was cleaned to ensure adequate unit weight and air content measurements. The top of the air pressure meter was clamped down once the petcocks were open and all the air was out. Using a syringe, water was pumped through one petcock until a steady stream of water came through the other petcock. Air was pumped into the chamber to the calibrated percentage of air, both petcocks were closed, and pressure was applied to the lever arm while the air pressure bowl was hit with a mallet. The pressure then was equalized, and the value that the needle on the dial gage normalized on was the air content percentage of the concrete (ASTM, 2017a). A picture of the air content test setup is shown in Figure 4-4.


**Figure 4-4: ASTM C231 Air Content Test**

### *4.2.4 Compressive Strength*

Compressive strength was determined for all trial mixes according to ASTM C39. Three 4 x 8-in. cylinders were used to determine the compressive strength at 4 and 6 hours after casting the specimens. The cylinders were filled with fresh concrete in two layers with each layer being vibrated with one insertion. After the vibration was complete, the sides were tapped with the palm of the hand to finish the consolidation process and avoid honeycombing (ASTM, 2015b). The specimens were tested using a 250-kip Forney testing machine. Specimens were secured to the compressive machine using neoprene pads and end caps. The loading rate applied to the specimens was 355–525 lbs/sec per ASTM C39 (ASTM, 2015b). A picture of a 4 x 8-in. cylinder being tested for compressive strength is shown in Figure 4-5.


**Figure 4-5: ASTM C39 Compressive Strength Test**

## 4.3 Full PCCP Mix Designs

Full concrete batches were mixed once the air-entraining and water-reducing dosages were established and the compressive strengths met the requirements. Each batch consisted of two 3 $ft^3$ sub-batches. The full PCCP batch was divided into two parts due to mixing constraints. In addition to conducting slump and air content tests as with the trial mixes, compressive and tensile strength tests (per ASTM C39 and ASTM C496, respectively) were conducted every 4, 6, 24, and 72 hours, as well as 7 and 28 days after casting specimens. ASTM C157, KT-79, ASTM C672 and KTMR-22 were utilized for durability testing. The unit weight and temperature of the concrete were also measured according to ASTM C138 and ASTM C1064, respectively. Table 4-2 shows the mix design for each PCCP ID reduced to 3 $ft^3$. Table 4-2 also shows the weight of the aggregates in the oven-dried condition and water corrections for the 32% liquid calcium chloride and aggregate absorption.

**Table 4-2: Mix Design Quantities**

| PCCP ID | Mix Design for 3 ft$^3$ of High Early Strength Concrete | | | | | | |
|---|---|---|---|---|---|---|---|
| | Cement (lbs) | Water (lbs) | Dry Rock (lbs) | Dry Sand (lbs) | Daravair® 1400 (oz) | ADVA® 140M (oz) | Calcium Chloride (oz) |
| 1 | 62.7 | 27.3 | 167.7 | 171.1 | 1.4 | 11.9 | 17.6 |
| 2 | 62.7 | 26.5 | 167.7 | 171.1 | 1.6 | 11.9 | 35.1 |
| 3 | 73.1 | 30.7 | 158.6 | 161.8 | 2.0 | 9.0 | 20.5 |
| 4 | 73.1 | 29.9 | 158.6 | 161.8 | 1.7 | 9.0 | 41.0 |
| 5 | 83.6 | 34.2 | 149.4 | 152.4 | 1.4 | 8.5 | 23.4 |
| 6 | 83.6 | 33.2 | 149.4 | 152.4 | 1.2 | 8.4 | 46.8 |

### *4.3.1 Compressive Strength*

Compressive strength tests were completed at 4, 6, 24, and 72 hours and at 7 and 28 days after casting specimens according to the same procedure used for the trial mixes.

### *4.3.2 Split Tensile Strength*

Split tensile strength was determined according to ASTM C496. Two 6 x 12-in. cylinders were used to determine tensile strength at 4, 6, 24, and 72 hours and at 7 and 28 days after casting the specimens. The cylinders were filled with fresh concrete in two layers with each layer vibrated with two insertions. After the vibration was completed, the sides were tapped with the palm of the hand to finish the consolidation process and avoid honeycombing. The specimens were tested using a hydraulic compression machine. The cylindrical specimens were secured to the compression machine, and force was applied along each specimen’s length at a rate of 188–375 lb/sec (ASTM, 2011a). A 6 x 12-in. cylinder being tested for tensile strength is shown in Figure 4-6.

**Figure 4-6: ASTM C496 Split Tensile Strength Test**

### *4.3.3 Durability Testing: Preparation of Concrete Beams*

For ASTM C157 and KTMR-22, six concrete beams with dimensions of 3 x 4 x 16 in. were made per ASTM C192 for every full PCCP batch mix (ASTM, 2016b). The beams were placed in an external vibrating table, filled with one layer of fresh concrete, and vibrated for 30 seconds. This method of consolidation was chosen because of the fast rate of hydration experienced by high early strength mix designs. All the faces of the molds were struck with a mallet several times as the final consolidation technique, and then a trowel was used to smooth the exposed surface. Figure 4-7 shows the setup for casting the concrete beams.

**Figure 4-7: Setup for Casting Concrete Beams**

### *4.3.4 Dry Shrinkage Testing ASTM C157*

Dry shrinkage testing was conducted in accordance with ASTM C157 for each full PCCP mix. To monitor the length change of the concrete beams, stainless steel gauges, manufactured by Humboldt, were installed at both ends of the beams prior to mixing. The specimens were removed from their molds after 6 hours and placed in a 100% moist room for 24 hours. After 24 hours, the beams were placed in lime-saturated water maintained at 73 ± 1$^{o}$F for a minimum of 30 minutes to minimize length variations due to variations in temperature. The beams were then removed from the lime-saturated water bath and wiped with a damp cloth, and then the initial comparator reading and the initial beam length were immediately taken. The comparator reading was obtained by placing an invar bar in the length comparator equipped with a digital deflection indicator and then setting a zero-reference point. The digital deflection indicator displays the length difference with a precision of 0.00001 in. Next, the invar bar was removed and replaced by a beam. The beam was secured to the comparator using gauge studs. The beam was then rotated around the comparator, and the smallest length displayed was taken. This process was repeated for two additional beams, and the initial indicator reading was determined. The three beams were then placed in the lime-saturated water for 14 days; another comparator reading was taken for the three beams at the end of the 14 days. The beams were transferred to a drying room that maintained a 73 ± 3°F temperature and a relative humidity of 50 ± 4%. Comparator readings for all three beams were taken after 4,7,14, and 28 days in the drying room and after 8 and 16 weeks. The percentage of length change of the specimens was calculated using Equation 4-1. Figure 4-8 shows the length change comparator used in this study.

$$\textbf{Length Change}\ (\%) = \frac{(\boldsymbol{l_x} - \boldsymbol{l_o})}{(\boldsymbol{l_1})} * \mathbf{100} \qquad \textbf{Equation 4-1}$$

Where: $l_x$ = Indicator reading (in.)

$l_o$ = Initial indicator reading (in.)

$l_i$ = Initial prism length (in.)

**Figure 4-8: Length Change Comparator**

### *4.3.5 Surface Resistivity (KT-79)*

The surface resistivity test was conducted prior to compressive strength testing on the 4 x 8-in. concrete cylinders using a Resipod Proceq meter, as shown in Figure 4-9. The surface resistivity measurement was obtained by marking the top of the circular face at 0-, 90-, 180-, and 270-degree points and then transferring those marks to the center of the longitudinal side of the specimen. The concrete cylinder was placed on its length side in a moist placeholder, and the Resipod meter was placed on the longitudinal side with the two inner probes centered in the mid mark. Surface resistivity was then recorded at 0-, 90-, 180-, and 270-degree points twice for a total of eight readings for each specimen.


**Figure 4-9: Resipod Proceq Surface Resistivity Meter**

### *4.3.6 Scaling Resistance Testing ASTM C672*

The ASTM C672 test was conducted using plastic rings made of standard five-gallon plastic buckets. The plastic rings, which were 3 in. deep, were used for the specimen side forms and the forthcoming dike walls. The minimum surface area requirement of 72 $in^2$ was met by the plastic rings with minimum diameters of 10 in. (ASTM, 2012a). Each plastic ring was cut on the side and duct taped together for easy removal after casting.

Prior to mixing, the rings were secured with silicone to a plastic sheet, and the plastic sheet was secured to the ground to avoid leakage during concrete curing. One specimen was made for each full batch of PCCP mix due to refrigeration constraints. Before concrete placement, a form release agent was applied to the inside of the mold, and the concrete was poured into the mold in a single lift. The specimen was then rodded 36 times with a 0.625-in. diameter steel rod one time for each 2 $in^2$ of surface area (ASTM, 2012a). The surface was then leveled with a trowel. After the concrete stopped bleeding, the surface was finished with sawing motion of a strike-off board and brushed with a medium-stiffness brush for the final finishing. Figure 4-10 shows the mold setup for the scaling resistance test. A plastic sheet was then placed over the specimen (but did not touch the specimen) for 24 hours. After the 24 hours, the specimen was demolded, placed in a 100% relative humidity room for 14 days, and then transferred to a drying room for another 14 days. The plastic ring previously used for the form was then placed on the hardened concrete with a 0.5-in. dike. A solution of calcium chloride approximately 0.25 in. deep then covered the surface of the specimen. Freeze-thaw cycles were conducted, and evaluation of the specimen and photographic documentation occurred at 5-cycle intervals.

**Figure 4-10: Scaling Resistance Mold Setup**

### *4.3.7 Freeze-Thaw Testing KTMR-22*

This research study followed the KTMR-22 freeze-thaw test method. After 6 hours in the steel molds, the beams were de-molded and placed in a 100% moist room for 67 days. They were then transferred to a drying room till the 88th day since casting. The drying room was maintained at a 73 ± 3°F temperature and a relative humidity of 50 ± 4%. On the 88th day, the specimens were submerged in tap water maintained between 60°F and 80°F for 24 hours. and then they were submerged in ice water for another 24 hours. At the end of the 90-day curing period, the initial mass, resonant frequency, and length change readings were recorded. Continuous readings were taken at intervals not exceeding 36 cycles.

A freeze-thaw machine developed by Scientemp Corporation was used to automatically cycle the concrete beams through the temperatures stipulated in ASTM C666. The freeze-thaw chamber had a capacity of 20 concrete beams, with two slots containing concrete beams with thermocouple wires in order to monitor internal concrete temperatures. The beams were subjected to freezing in air for 110 minutes, reaching a temperature of 0 ± 3°F, and then thawed in water with an end temperature of 40 ± 3°F. The freeze-thaw machine was programed to complete 8 cycles per day.

At cycles not exceeding 36, the mass, resonant frequency, and length change readings were measured and recorded. The mass was recorded using a scale that met the ASTM 666 requirement of capacity more than 50% of the specimens' mass (ASTM, 2015a). Each beam was patted dry with a moist towel to maintain consistent moisture conditions before weighing. The percent change in mass was calculated using Equation 4-2. The resonant frequency was obtained using a James E-Meter™ MK II equipped with an impact hammer and an accelerometer. The dimensions of the beam were input into the James E-Meter™ MK II, and then the accelerometer was placed approximately 1 in. from the specimen end. The hammer impacted the beam 1 in. from the other specimen end.. The accelerometer received the induced wave, and the meter computed the frequency spectrum (NDT James Instruments Inc., 2017). Equation 4-3 shows how the resonant transverse frequency obtained from the meter was used to calculate the RDME of each beam. Length change was calculated using Equation 4-1.

$$\textbf{Mass Change}\ (\%) = \frac{(\mathbf{m_x} - \mathbf{m_o})}{(\boldsymbol{m_o})} * \mathbf{100} \qquad \textbf{Equation 4-2}$$

Where: $m_x$ = Mass reading at $x$ freeze-thaw cycle (kg)

$m_o$ = Initial mass reading (kg)

$$\textbf{RDME}\ (\%) = \frac{(\boldsymbol{n_x}^{2})}{(\boldsymbol{n_o}^{2})} * \mathbf{100} \qquad \textbf{Equation 4-3}$$

Where: $n_x$ = Transverse frequency reading at $x$ freeze-thaw cycle (Hz)

$n_o$ = Initial transvers frequency reading (Hz)

# Chapter 5 - Results

Durable concrete should be designed to withstand the process of deterioration while maintaining serviceability throughout a given service life and despite exposure conditions (Neville, 1996). Traditionally, no accepted standard has quantitatively defined durability, allowing concrete durability to be measured by a variety of factors. This research study conducted isothermal calorimetry to measure heat generated due to hydration as well as compressive and tensile tests to monitor strength gain. Dry shrinkage, surface resistivity, scaling resistance, and freeze-thaw measurements were taken to monitor the deterioration process.

## 5.1 High Early Strength Concrete Mix Designs and Fresh Properties

Material quantities for six high early strength concrete mixes are tabulated in Table 4-2. The fresh concrete properties for those mixes are shown in Table 5-1.

**Table 5-1: Fresh Concrete Properties**

| PCCP ID | Fresh Concrete Properties | | | |
|---|---|---|---|---|
| | Unit Weight (lbs./ft$^3$) | Slump (in.) | Air Content (%) | Concrete Temperature ($^o$F) |
| 1 | 142.7 | 3.5 | 7.2 | 74.3 |
| 2 | 138.6 | 6.25 | 9 | 77.4 |
| 3 | 143.4 | 2.75 | 6 | 78.7 |
| 4 | 142.0 | 3.25 | 6.6 | 80.1 |
| 5 | 141.0 | 2.75 | 6.5 | 79.6 |
| 6 | 143.6 | 3.75 | 4.5 | 79.8 |

## 5.2 Isothermal Calorimetry

Standard KDOT practices for isothermal calorimetry testing include constant thermal mass and a w/c ratio of 0.45. These parameters determine the base cement and water weights used in each isothermal calorimetry sample. However, these parameters differ from PCCP mixes designed to have varying cement contents and w/c ratios of 0.37. Moreover, materials that include cement, air-entraining agent, water reducer, and calcium chloride for isothermal calorimetry testing were identical to the materials used for all PCCP mixes. Exact dosages of the chemical admixtures for PCCP ID 1–6 of this study were used for isothermal calorimetry testing. KDOT performed these tests using the TAM Air Isothermal Calorimeter and named the tests KSU 1– KSU 6. Deionized water was also used in the isothermal calorimetry testing. To monitor the heat generated by various admixtures, each KSU experimental plan was composed of the following four subtests:

- Cement + Water
- Cement + Water + Air Entraining Agent
- Cement + Water + Air Entraining Agent + Water Reducer
- Cement + Water + Air Entraining Agent + Water Reducer + Calcium Chloride

The base amounts of cement and water were then adjusted to account for the cementitious material and admixtures used. The water volume was also adjusted to account for water content of the admixtures to maintain a w/c ratio of 0.45. The weight of the cement was also adjusted to meet a constant thermal mass, and the results (heat generated) were normalized to the mass of the cement. Table 5-2 shows the heat generated by the KSU samples that incorporated all chemical admixtures. Figure 5-1 shows the isothermal calorimetry data for the first subtest, as was representative of all KSU tests that included cement and water. Appendix A includes individual results for each KSU test performed and their corresponding graphs.

**Table 5-2: Heat Generated by KSU Samples**

| KSU ID | Calcium Chloride (% by dry weight of cement) | Air-Entraining Agent (oz/cwt) | Water Reducer (oz/cwt) | Peak Heat Generated (mW/g) | Time of Peak Heat Generation (Hours) | Total Heat Generated Within 2 Days (J/g) |
|---|---|---|---|---|---|---|
| KSU 1-4 | 1 | 2.25 | 19 | 8.13 | 7.08 | 310.30 |
| KSU 2-4 | 2 | 2.50 | 19 | 12.54 | 3.86 | 310.19 |
| KSU 3-4 | 1 | 2.70 | 12.25 | 8.69 | 6.41 | 314.87 |
| KSU 4-4 | 2 | 2.25 | 12.25 | 13.93 | 3.65 | 315.31 |
| KSU 5-4 | 1 | 1.63 | 10.13 | 10.85 | 4.50 | 319.51 |
| KSU 6-4 | 2 | 1.38 | 10 | 14.87 | 3.23 | 319.70 |

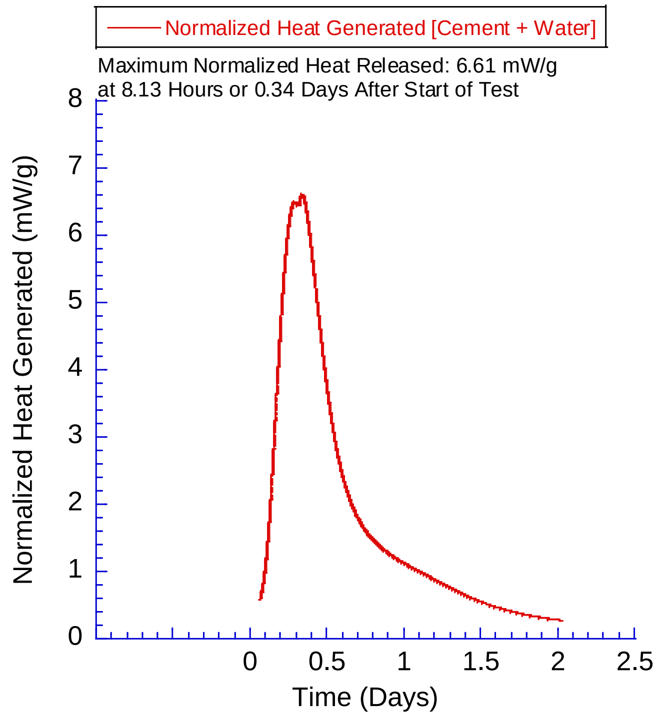


**Figure 5-1: Normalized Heat Generated for all KSU Cement + Water Isothermal Calorimetry**

## 5.3 Compressive Strength Testing

Compressive strength test results are shown in Table 5-3, and Figure 5-2 illustrates the results for all mixtures. ACI committee 209 reported a strength-development-with-time equation (Equation 5-1) in their report *Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete* (ACI Committee 209, 2008). Figure 5-2 also shows the ACI-predicted compressive strength with a best-fit curve for each mix design. Using KaleidaGraph software, the *s* value, as shown in Equation 5-1, was found for each PCCP mix through a best-fit curve line with its given coefficient of determination (Synergy Software, 2016). This statistical data is presented in Table 5-4. Figure 5-2 also shows a KDOT-indicated threshold, which stipulates a minimum of 1,800 psi compressive strength at 4 hours for mixtures with 2% calcium chloride or 1,800 psi compressive strength at 6 hours for mixtures containing 1% calcium chloride (KDOT, 2015). Additional graphical representations of compressive strengths based solely on cement content or calcium chloride and the best-fit curve line are provided in Appendix B.

$$\boldsymbol{f_{cmt} = \beta_e^2 f_{cm28}} \qquad \textbf{Equation 5-1}$$

Where:

$\beta_e$ = $exp\left[\frac{s}{2}\left(1-\sqrt{\frac{28}{t}}\right)\right]$

$s$ = 0.13 for Type III cement per ACI 209

$t$ = age of the concrete in days

$f_{cm28}$ = average 28-day compressive strength

**Table 5-3: Compressive Strength Test Results**

| PCCP ID | Average Compressive Strength (psi) | | | | | |
|---|---|---|---|---|---|---|
| | 4 hours | 6 hours | 24 hours | 72 hours | 7 days | 28 days |
| 1 | 796 | 2,125 | 5,076 | 6,599 | 7,409 | 8,411 |
| 2 | 1,654 | 2,482 | 5,178 | 6,354 | 6,700 | 7,528 |
| 3 | 920 | 2,301 | 4,843 | 6,747 | 7,476 | 8,469 |
| 4 | 2,139 | 3,358 | 5,192 | 6,738 | 7,022 | 8,228 |
| 5 | 1,149 | 2,394 | 4,704 | 6,348 | 6,697 | 7,979 |
| 6 | 2,765 | 4,308 | 6,591 | 8,081 | 8,814 | 10,091 |

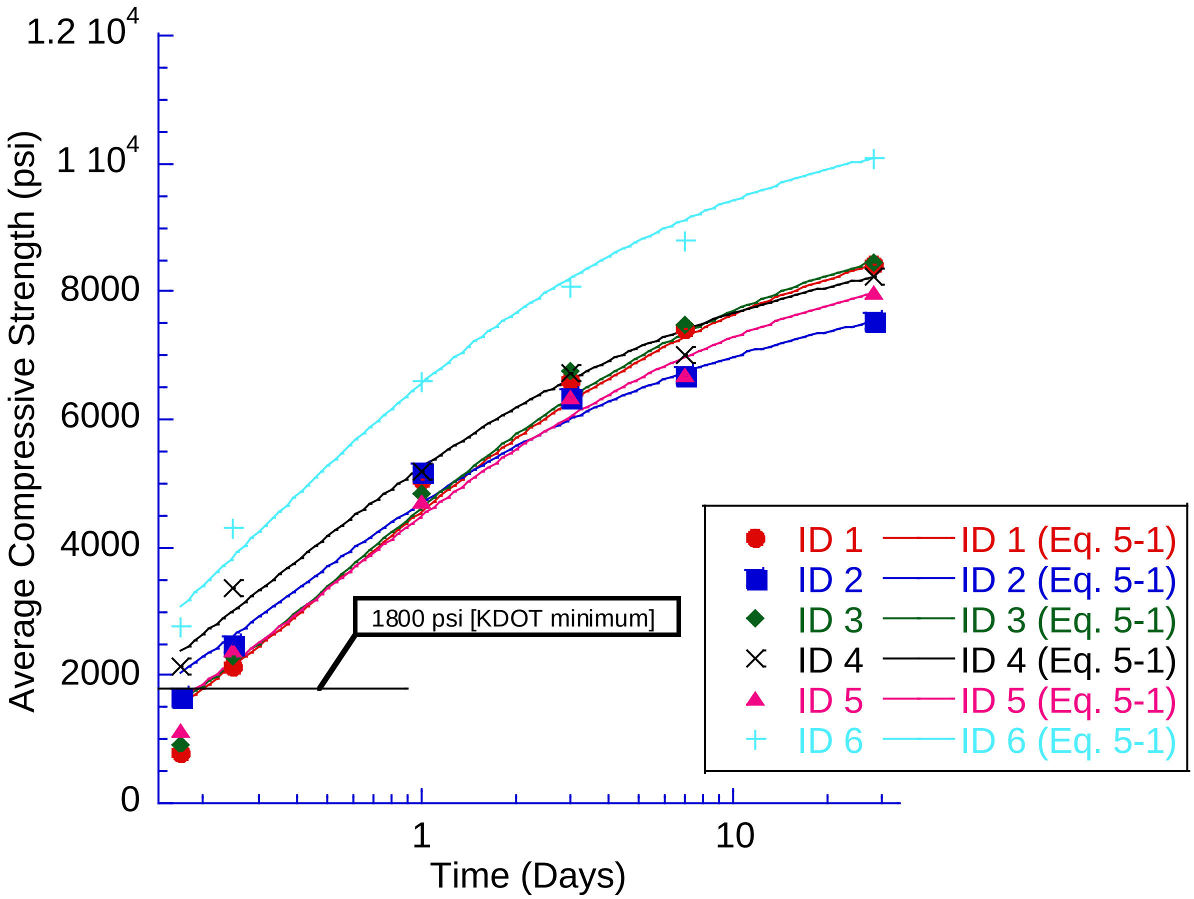


**Figure 5-2: Average Compressive Strength Development of all Mixes**

**Table 5-4: Strength Development with Time Statistical Values**

| PCCP ID | Best-fit *s* Value | Coefficient of Determination [$R^2$] |
|---|---|---|
| 1 | 0.14 | 0.98 |
| 2 | 0.11 | 0.98 |
| 3 | 0.14 | 0.98 |
| 4 | 0.10 | 0.99 |
| 5 | 0.13 | 0.99 |
| 6 | 0.10 | 0.99 |

## 5.4 Split Tensile Strength Testing

Split tensile strength test results are shown in Table 5-5, and a graphical representation of those results and a trend line are provided in Figure 5-3. The trend line is also demonstrated with Equation 5-2.

**Table 5-5: Split Tensile Strength Test Results**

| PCCP ID | Average Split Tensile Strength (psi) | | | | | |
|---|---|---|---|---|---|---|
| | 4 hours | 6 hours | 24 hours | 72 hours | 7 days | 28 days |
| 1 | 130 | 214 | 303 | 445 | 391 | 452 |
| 2 | 129 | 218 | 391 | 429 | 382 | 439 |
| 3 | 71 | 197 | 394 | 468 | 513 | 487 |
| 4 | 181 | 314 | 403 | 409 | 458 | 438 |
| 5 | 108 | 233 | 324 | 486 | 456 | 474 |
| 6 | 175 | 236 | 419 | 473 | 460 | 532 |

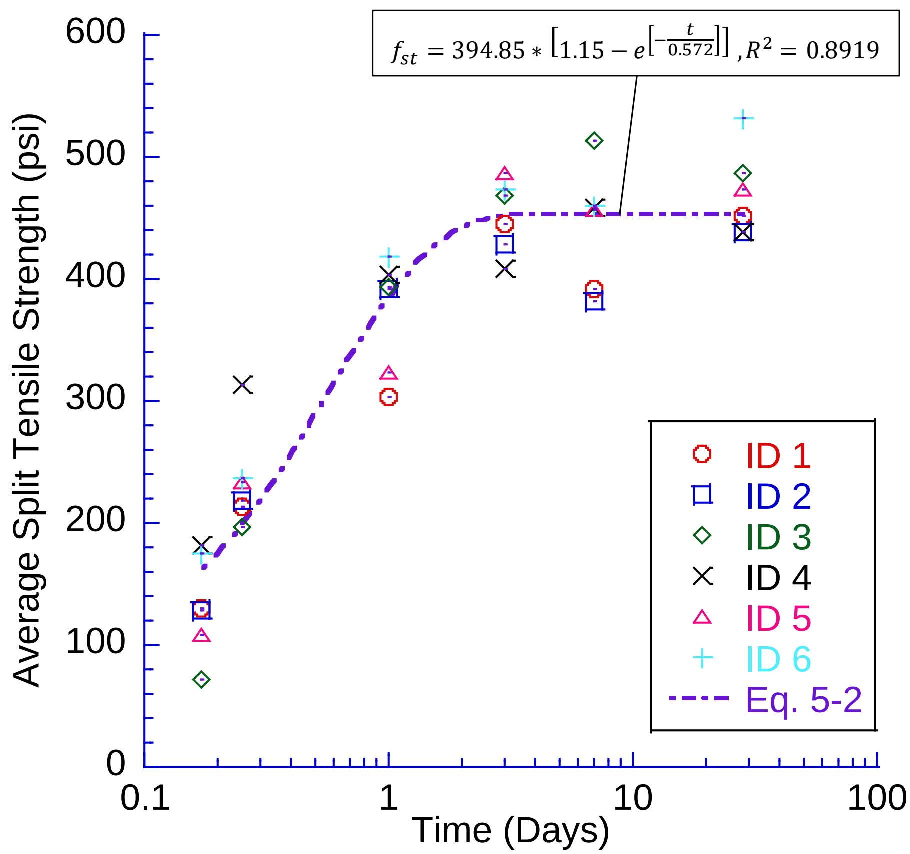


**Figure 5-3: Average Split Tensile Strength of all Mixes**

$$\boldsymbol{f_{st} = 394.85 * \left[1.15 - e^{\left[-\frac{t}{0.572}\right]}\right]} \qquad \textbf{Equation 5-2}$$

Where: $f_{st}$ = splitting tensile strength (psi)

$t$ = time after casting in (days)

## 5.5 Split Tensile Strength Versus Compressive Strength

Certain equations model the relationship between split tensile strength and compressive strength of new concrete. Equation 5-3 shows this relationship as described in Chapter 18 of the ACI building code. ACI committee 363 also established an upper bound relationship equation, as shown in Equation 5-4 (Oluokun et al., 1991). However, when these equations were plotted on the split tensile versus compressive strength graph, they overestimated the tensile strength of the high early strength mixes. Equation 5-5 more accurately predicts the split tensile strength given the compressive strength for PCCP mixes because of the increased air content compared to typical structural concrete. A general trend between all split tensile and compressive strengths of the high early strength concrete mixtures is illustrated in Figure 5-4. Figure 5-4 also incorporates Equation 5-3 and Equation 5-4 to graphically represent how these established equations overestimate split tensile strength. Figure 5-5 compares Equation 5-3 and Equation 5-5. Additional graphical representations based solely on cement content or calcium chloride are provided in Appendix C.

$$\mathbf{f_{st} = 6[f_c']^{0.5}}$$ **Equation 5-3**

$$\mathbf{f_{st} = 7.4[f_c']^{0.5}}$$ **Equation 5-4**

$$\mathbf{f_{st} = 2.63[f_c' - 432.26]^{0.58}}$$ **Equation 5-5**

Where: $f_c'$ = compressive strength (psi)

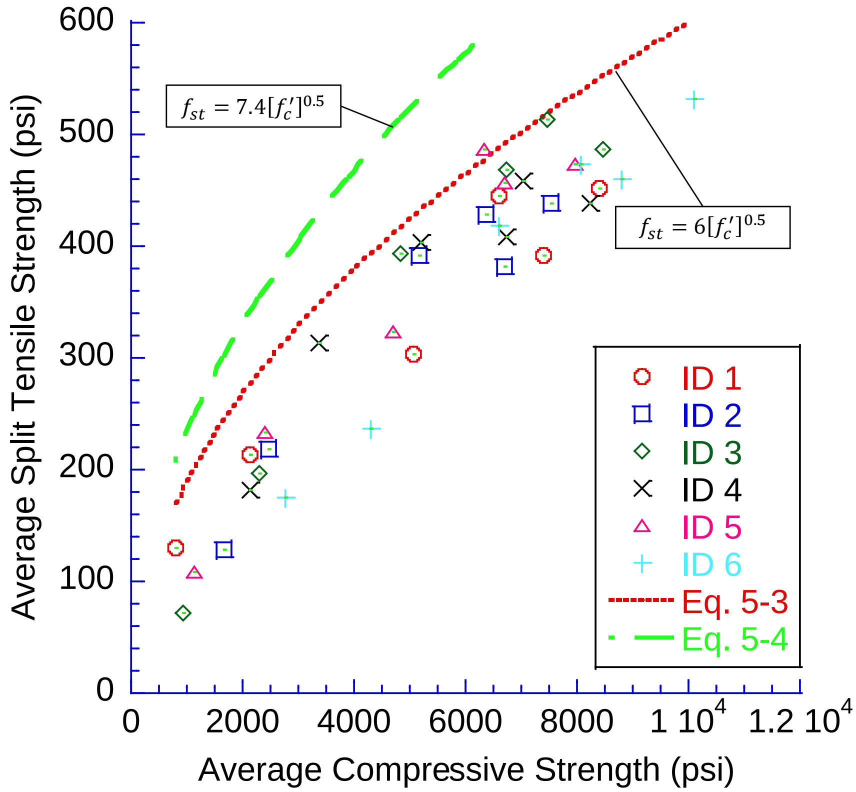


**Figure 5-4: Trend Between Split Tensile and Compressive Strength with Eq. 5-3 and Eq. 5-4**

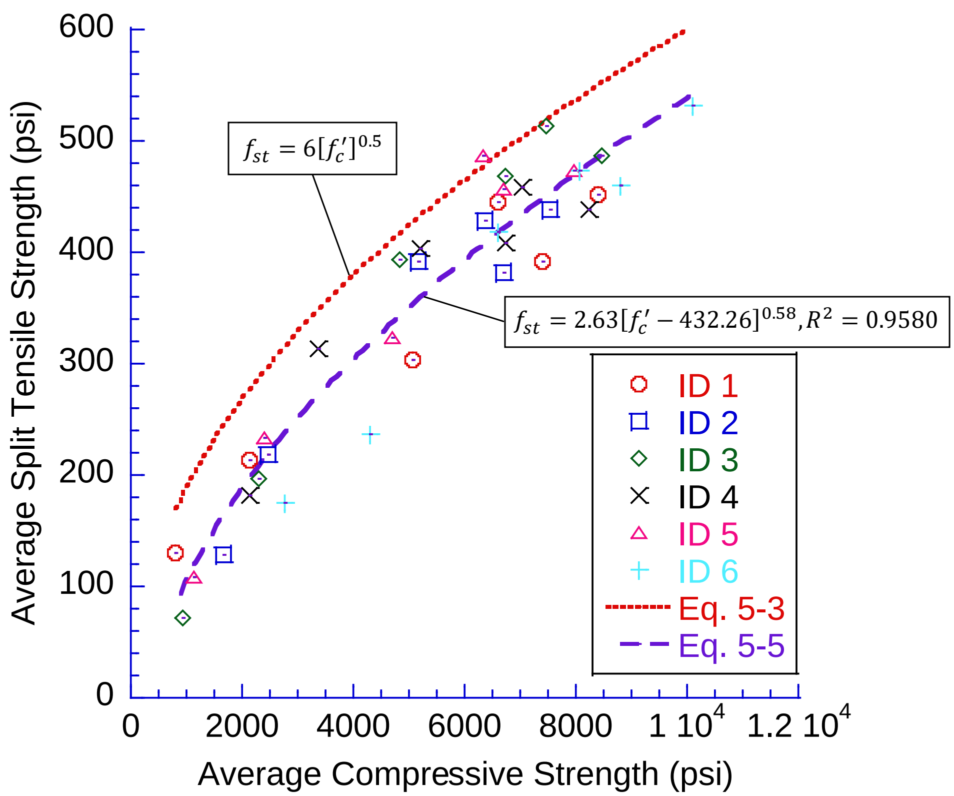


**Figure 5-5: Trend Between Split Tensile and Compressive Strength with Eq. 5-3 and Eq. 5-5**

## 5.6 Dry Shrinkage Testing

Table 5-6 shows the decrease in length due to dry shrinkage at various time intervals after 14 days of lime-saturated water curing. To further analyze the shrinkage strain of the high early strength mixes, dry shrinkage strain was compared to the shrinkage strain equation provided by ACI committee 209 (Equation 5-6) (ACI Committee 209, 2008). Figure 5-6 shows the shrinkage strain according to ACI and Equation 5-6 for each PCCP mix. Using KaleidaGraph, the *k* value in Equation 5-6 was found for each PCCP mix with its given coefficient of determination (Synergy Software, 2016). The statistical data are presented in Table 5-7. Additional graphical representations of shrinkage strain based solely on cement content or calcium chloride are included in Appendix D.

$$\boldsymbol{\varepsilon_{sh}(t, t_c) = \varepsilon_{shu}\beta(h)\beta(t - t_c)}$$ **Equation 5-6**

Where:

$\varepsilon_{shu}$ = $900k\left(\frac{4350}{f_{cm28}}\right)^{1/2} x10^{-6}\ in\ in.-lb$ [ultimate shrinkage strain]

$k$ = 1.15 for Type III cement per ACI 209

$f_{cm28}$ = average 28-day compressive strength

$\beta(h)$ = $(1 - 1.18h^4)$ [humidity correction term]

$h$ = relative humidity in decimal form

$\beta(t - t_c)$ = $\left[\frac{(t-t_c)}{(t-t_c)+77(V/S)^2}\right]^{1/2} in\ in.-lb$ [time of drying correction term]

$t$ = age of concrete (days)

$t_c$ = age drying starts or end of moist curing (days)

*V/S* = volume-surface ratio (in.)

**Table 5-6: Drying Shrinkage Results**

| PCCP ID | Average Dry Shrinkage (x $10^{-4}$) with 14 days of Lime-Saturated Water Cure (% change) | | | | | |
|---|---|---|---|---|---|---|
| | 4 days | 7 days | 14 days | 28 days | 56 days (8 weeks) | 112 days (16 weeks) |
| 1 | 110 | 168 | 245 | 308 | 327 | 324 |
| 2 | 102 | 200 | 236 | 271 | 276 | 268 |
| 3 | 77 | 154 | 240 | 293 | 308 | 320 |
| 4 | 132 | 221 | 289 | 318 | 322 | 313 |
| 5 | 110 | 195 | 276 | 422 | 445 | 449 |
| 6 | 78 | 159 | 235 | 270 | 293 | 268 |

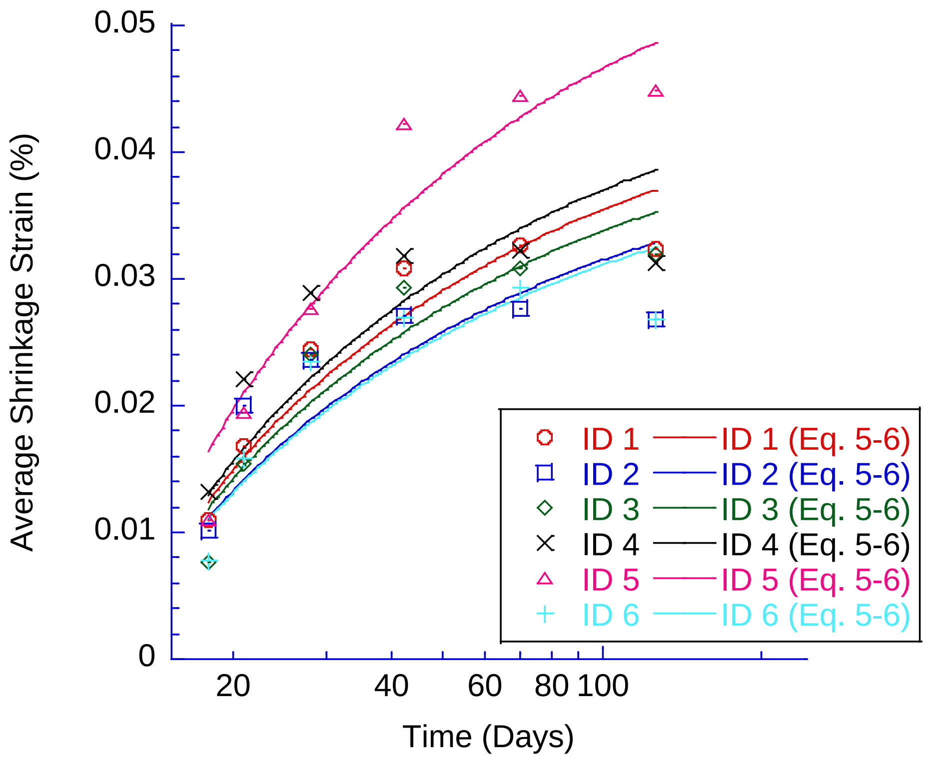


**Figure 5-6: Average Shrinkage Strain Development with Eq. 5-6**

**Table 5-7: Shrinkage Strain Statistical Values**

| PCCP ID | Best-fit *k* Value | Coefficient of Determination [$R^2$] |
|---|---|---|
| 1 | 0.76 | 0.88 |
| 2 | 0.63 | 0.53 |
| 3 | 0.72 | 0.88 |
| 4 | 0.78 | 0.49 |
| 5 | 0.97 | 0.91 |
| 6 | 0.73 | 0.76 |

## 5.7 Surface Resistivity Testing

Results of surface resistivity testing conducted on all mixes are shown in Table 5-8, and a graphical representation of the surface resistivity of all mixtures is shown in Figure 5-7. Figure 5-7 also shows the KDOT-specified minimum surface resistivity threshold value. Additional graphical representations of surface resistivity based solely on cement content or calcium chloride are presented in Appendix E.

**Table 5-8: Surface Resistivity Results**

| PCCP ID | Average Surface Resistivity (kΩ-cm) | | | | | |
|---|---|---|---|---|---|---|
| | 4 hours | 6 hours | 24 hours | 72 hours | 7 days | 28 days |
| 1 | 1.4 | 2.2 | 5.6 | 8.2 | 9.4 | 14.2 |
| 2 | 1.5 | 2.2 | 4.6 | 6.9 | 8.3 | 13.0 |
| 3 | 1.2 | 1.9 | 5.1 | 8.0 | 9.8 | 13.6 |
| 4 | 1.5 | 2.2 | 4.2 | 5.8 | 7.6 | 11.7 |
| 5 | 1.2 | 2.0 | 4.6 | 6.7 | 8.2 | 11.9 |
| 6 | 1.3 | 2.0 | 4.0 | 5.4 | 6.8 | 10.8 |

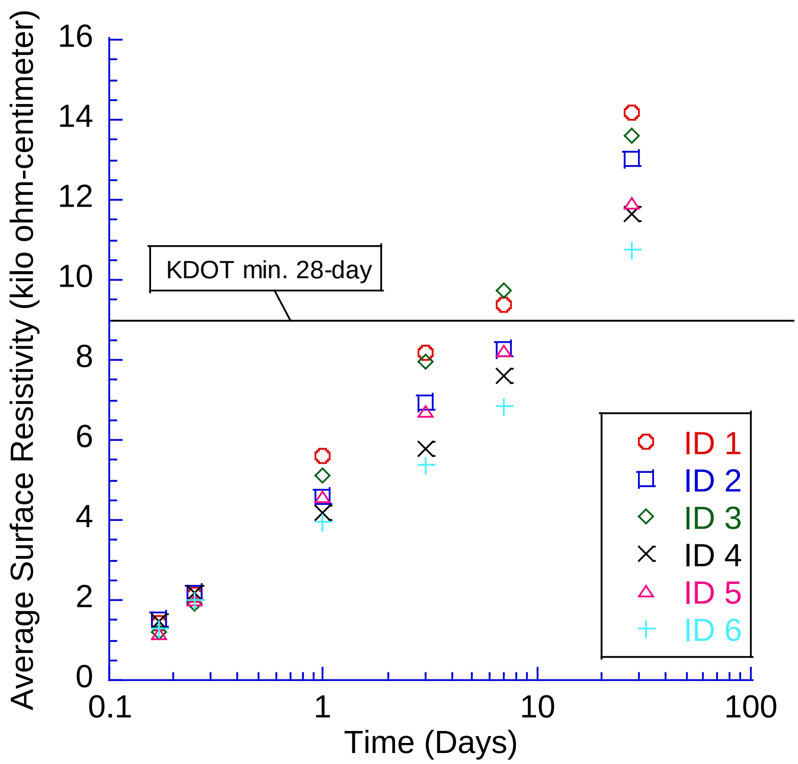


**Figure 5-7: Average Surface Resistivity for all Mixes**

## 5.8 Scaling Resistance Testing

The scaling resistance rating after every five cycles was taken for each specimen in accordance with ASTM C672. Appendix F shows images for each PCCP mix before and after conducting ATSM C672. The type of caulking used on the outer edges of the specimen could not be removed without chipping the concrete. Therefore, the final end mass of each specimen was not recorded, and the mass lost after the freeze-thaw cycles was not obtained. The sealant used was a siliconized acrylic adhesive. Figure 5-8 and Figure 5-9 show the specimens from PCCP 6 before and after conducting ASTM C672, respectively.

**Figure 5-8: Before Conducting ASTM C672 PCCP 6 Rating = 0**

**Figure 5-9: Before Conducting ASTM C672 PCCP 6 Rating = 2**

## 5.9 Freeze-Thaw Testing

Changes in mass, expansion, and RDME were measured as part of the freeze-thaw testing of specimens for all PCCP mixtures. The means of these properties were then graphed for each cement content group, calcium chloride group, and individual PCCP mix. These graphical representations are presented in Appendix G through Appendix I. Results for mass change, expansion, and RDME for all six PCCP mixtures are discussed in the following subsections.

### *5.9.1 Change in Mass*

The change in mass was recorded throughout the freezing and thawing period, with intervals not exceeding 36 cycles. Table 5-9 shows results of the average change in mass after 300 cycles and the recorded change in mass at the end of the test following KTMR-22. Figure 5-10 shows a graphical representation of the data.

**Table 5-9: Average Change in Mass After 300 Cycles and After Final Cycle for Each PCCP Mix**

| PCCP ID | Change in Mass After 300 Cycles (%) | Final Change in Mass (%) | Final No. of Cycles |
|---|---|---|---|
| 1 | 0.21 | 0.32 | 657 |
| 2 | 0.33 | 0.26 | 659 |
| 3 | 0.25 | 0.23 | 622 |
| 4 | 0.30 | 0.29 | 655 |
| 5 | 0.24 | 0.26 | 656 |
| 6 | 0.34 | 0.97 | 650 |

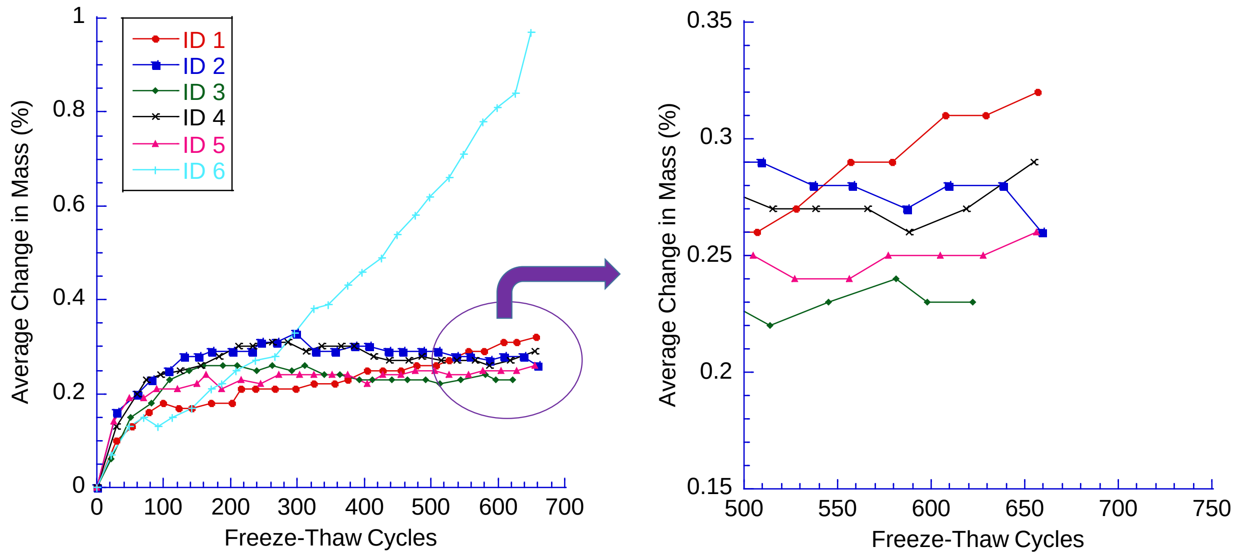


**Figure 5-10: Average Change in Mass for all PCCP Mixes**

### *5.9.2 Expansion*

Expansion was recorded throughout the freezing and thawing period, with intervals not exceeding 36 cycles. Table 5-10 shows the average expansion after 300 cycles and the recorded expansion at the end of the test. Figure 5-11 shows a graphical representation of the data, and Figure 5-12 compares ASTM C666 and KTMR-22 requirements for specimens that were cured by the KTMR-22 procedure.

**Table 5-10: Average Expansion After 300 Cycles and After Final Cycle for Each PCCP Mix**

| PCCP ID | Expansion After 300 Cycles (%) | Final Expansion (%) | Final No. of Cycles |
|---|---|---|---|
| 1 | 0.030 | 0.048 | 657 |
| 2 | 0.015 | 0.020 | 659 |
| 3 | 0.025 | 0.031 | 622 |
| 4 | 0.025 | 0.026 | 655 |
| 5 | -0.042 | -0.045 | 656 |
| 6 | 0.096 | 0.564 | 650 |

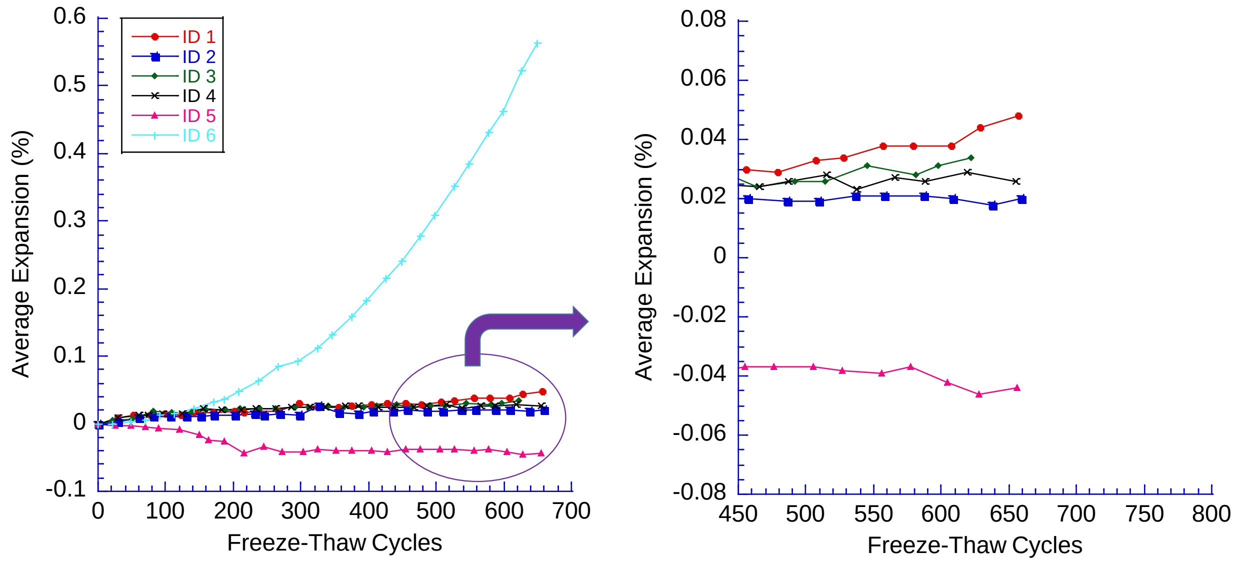


**Figure 5-11: Average Expansion for all PCCP Mixes**

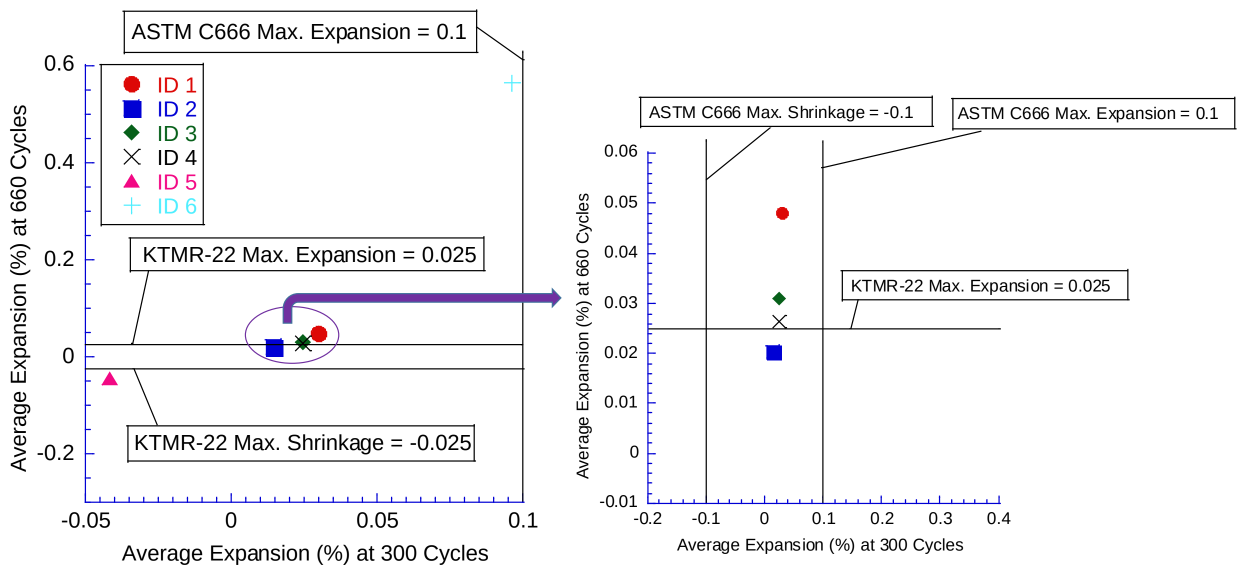


**Figure 5-12: Comparison of Expansion Results to ASTM C666 and KTMR-22 Requirements**

### *5.9.3 Relative Dynamic Modulus of Elasticity*

RDME was recorded throughout the freezing and thawing period, with intervals not exceeding 36 cycles. Table 5-11 shows the results of the average RDME after 300 cycles and the recorded RDME at the end of the test. Figure 5-13 shows a graphical representation of the data, and Figure 5-14 compares ASTM C666 and KTMR-22 requirements for specimens that were cured by the KTMR-22 procedure.

**Table 5-11: Average RDME After 300 Cycles and After Final Cycle for Each PCCP Mix**

| PCCP ID | RDME After 300 Cycles (%) | Final RDME (%) | Final No. of Cycles |
|---|---|---|---|
| 1 | 94 | 92 | 657 |
| 2 | 97 | 96 | 659 |
| 3 | 97 | 97 | 622 |
| 4 | 95 | 95 | 655 |
| 5 | 96 | 94 | 656 |
| 6 | 71 | 0 | 650 |

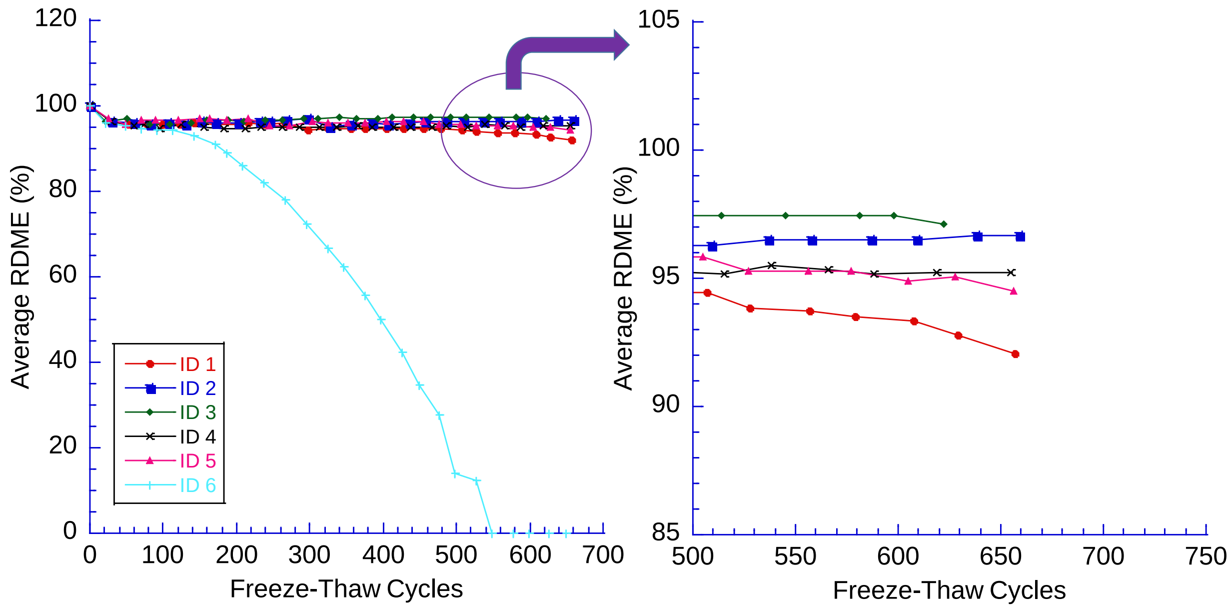


**Figure 5-13: Average RDME for all PCCP Mixes**

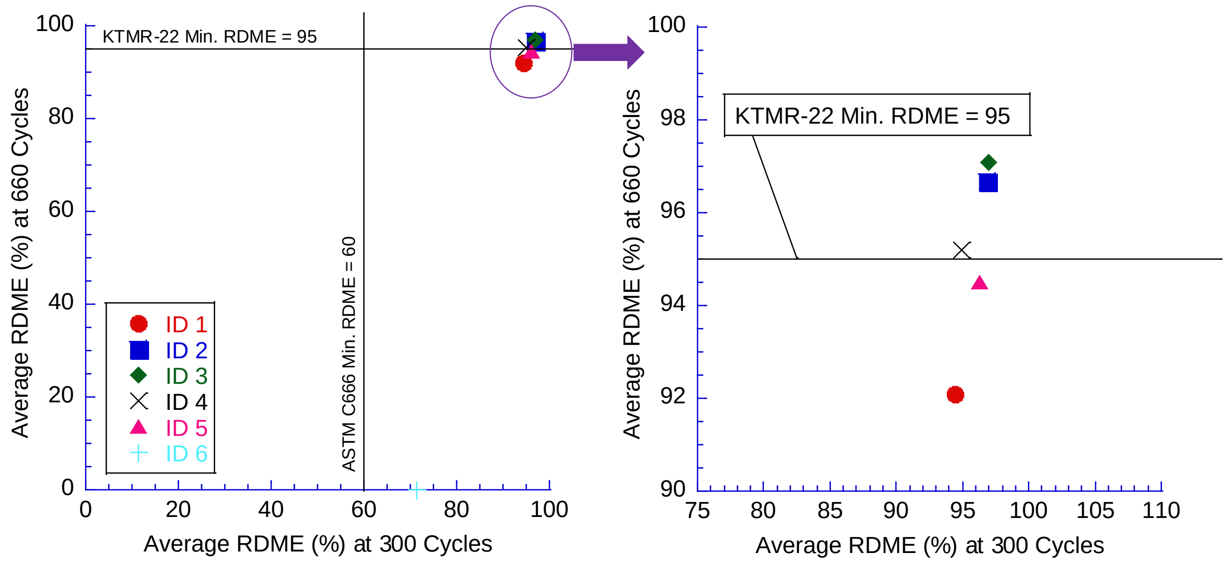


**Figure 5-14: Comparison of RDME Results to ASTM C666 and KTMR-22 Requirements**

# Chapter 6 - Analysis and Discussion

## 6.1 Isothermal Calorimetry

Isothermal calorimetry performed on cement paste samples generally indicated that lower dosages of air-entraining agent and water reducer produce a greater peak heat in less time, as shown by comparing KSU 4-4 results to KSU 6-4 isothermal calorimetry results. Both tests had the same w/c ratio and 2% calcium chloride. KSU 4-4 had an air-entraining dosage of 2.25 oz/cwt and a water reducer dosage of 12.25 oz/cwt; its maximum peak heat generated was 13.93 mW/g at 3.65 hours after the start of calorimetry testing. KSU 6-4 had an air-entraining dosage of 1.38 oz/cwt and a water reducer dosage of 10 oz/cwt; its maximum peak heat generated was 14.87 mW/g at 3.23 hours after the start of calorimetry testing. A similar trend was seen for the 1% calcium chloride cement paste samples when comparing KSU 3-4 to KSU 5-4 isothermal calorimetry results. Mixtures with 2% calcium chloride show a greater maximum heat generation in a shorter amount of time than those with 1% calcium chloride. Out of all the isothermal calorimetry tests performed, KSU 6-4 had the largest peak heat generated in the shortest amount of time. The lowest peak heat generated was 8.13 mW/g from KSU 1-4, which had 2.25 oz/cwt of air-entraining agent, 19 oz/cwt of water reducer, and 1% calcium chloride. The maximum peak heat generated for KSU 1-4 occurred at 7.08 hours after the start of the test.

The cumulative heat produced by these samples was also analyzed using the trapezoidal rule for calculating area under a curve for each heat generated versus time isothermal calorimetry graph. Results for total heat generated after two days for specimens containing all chemical admixtures indicated a maximum 3% increase in total heat generated between KSU 2-4 and KSU 6-4 although both contained 2% calcium chloride. Similarly, a 3% increase in total heat generated occurred between KSU 1-4 and KSU 5-4, both of which contained 1% calcium chloride.

## 6.2 Strength Behavior

All PCCP mixes met the required strength of 1,800 psi within their corresponding time frames except PCCP ID 2, which had a strength of 1,654 psi at 4 hours. However, this mixture had the highest measured air content (9%) and slump (6.25 in.). A strength of 1,800 psi at 4 hours could occur if the dosages of air-entraining agent and water reducer of PCCP ID 2 were adjusted. All mixes significantly exceeded the compressive strength of 1,800 psi at 6 hours. PCCP 1 demonstrated the lowest compressive strength 2,125 psi at 6 hours, but that strength still vastly exceeded the minimum required strength. The compressive strength more than doubled at 4 hours with the addition of 2% calcium chloride compared to 1% calcium chloride regardless of cement content. In addition, 1% calcium chloride mixes had a closer approximation to the *s* value stipulated by ACI committee 209 for Portland Cement Type III mixes in the strength-development-with-time equation than the *s* value for 2% calcium chloride mixes. The 1% and 2% calcium chloride mixes had an average *s* value of 0.14 and 0.11, respectively. ACI committee 209 recommends an *s* value of 0.13 for Portland Cement Type III mixes when using their strength-development-with-time equation, shown in Equation 5-1. The current *s* value stipulated by ACI committee 209 could potentially be modified for high early strength PCCP mixtures with calcium chloride accelerator.

The split tensile strength data obtained from this study were graphed with an approximate equation for the split tensile strength development with time, as shown in Equation 5-2, and with a coefficient of determination of 0.89. The current ACI-established relationship between concrete-splitting tensile strength and compressive strength does not accurately represent high early strength mixes because it overpredicts the splitting tensile strength when a compressive strength value is known. Therefore, Equation 5-5, developed in this study, more accurately represents the relationship between splitting tensile and compressive strength of high early strength concrete mixes.

## 6.3 Drying Shrinkage

In general, drying shrinkage value of mixtures with 1% calcium chloride was higher than mixtures with 2% calcium chloride. PCCP 5, which had 752 lb/yd$^3$ cement content and 1% calcium chloride, experienced the most dry shrinkage, with a maximum 0.0449% decrease compared to significantly less drying shrinkage for other mixtures. None of the mixtures surpassed the limit of 0.05% decrease in drying shrinkage previously established by the Illinois Tollway special provisions for 28 days of air drying, even after 16 weeks of air drying. Additionally, when fitting the shrinkage strain Equation 5-6 in this thesis to the average strain of the PCCP mixtures, the equation reveled a higher coefficient of determination value for mixtures containing 1% calcium chloride (by dry weight of cement). Autogenous shrinkage was not measured in this study; therefore, the *k* value was lower for each PCCP mix than the value determined by ACI committee 209.

## 6.4 Durability

Mixtures with 1% calcium chloride and lower cement content generally demonstrated higher surface resistivity, which is an indication of low permeability. The highest 28-day average surface resistivity reading was 14.2 kΩ-cm for PCCP ID 1, followed by PCCP ID 3 with a 28-day average surface resistivity reading of 13.6 kΩ-cm. Furthermore, scaling resistance testing showed no significant scaling damage for any mixtures after the 55 cycles of freeze-thaw per ASTM C672, potentially due to the establishment of initial high air content. The lowest air content was 4.5% from PCCP ID 6. All samples had an end rating of 2, indicating slight-to-moderate scaling. Analysis of the freeze-thaw testing data showed that PCCP ID 6 had the highest percentage increase (0.97%) in mass, while the rest of the mixtures had relatively similar but smaller percentage increase in mass. Ultimately, PCCP ID 3 had the lowest mass increase (0.23%) at the end of the testing period, followed by PCCP ID 2 and PCCP ID 5 with 0.26% each. PCCP ID 6 experienced the greatest expansion, while PCCP ID 5 was the only mixture to shrink during the freezing and thawing cycles. The high cement content factor or calcium chloride dosage could potentially be influencing these increases. Although the relationship between calcium chloride and freeze-thaw performance is not clear from these results, PCCP ID 2 was the only mixture that passed KTMR-22 maximum expansion requirements. Analysis of the RDME data revealed that only PCCP ID 2, PCCP ID 3, and PCCP ID 4 passed KTMR-22 minimum requirements for RDME. PCCP ID 3 had the greatest RDME (97.09%), followed by PCCP ID 2 (96.68%), and PCCP ID 4 (95.20%).

# Chapter 7 - Conclusions and Recommendations

This research study utilized various concrete testing methods to monitor the deterioration process of high early strength concrete repair mixes. Heat generation of cement paste samples and strength gain of these repair mixes at various time intervals were also monitored. PCCP ID 2 generally showed better durability characteristics compared to the other mixtures. Isothermal calorimetry data for KSU 2-4, which had the same chemical dosages as PCCP ID 2, indicated the lowest total heat generated within the 2-day testing period. The minimum compressive strength of 1,800 psi was reached at 6 hours but failed to reach the threshold of 1,800 psi at 4 hours for mixtures with 2% calcium chloride (by dry weight of cement). Slight reductions of the air-entraining agent dosage could allow PCCP ID 2 to reach minimum compressive strength at 4 hours. However, PCCP ID 2 was among the top three mixtures with the highest surface resistivity after 28 days and the least drying shrinkage at the end of 16 weeks. PCCP ID 2 was ultimately determined to be a promising durable high early strength mixture because it passed both the maximum expansion and minimum RDME per KTMR-22.

## 7.1 Isothermal Calorimetry

Research observations proved that the total heat generated is primarily influenced by the dosages of air-entraining agent and water-reducing chemical admixtures rather than the calcium chloride accelerator. Furthermore, study results indicated that the calcium chloride accelerator contributes to the peak heat generated and the time at which this peak occurs.

## 7.2 Concrete Strength

The minimum compressive strength of 1,800 psi can most likely be reached using a cement content of 564 lb/yd$^3$ and a 1% calcium chloride (by dry weight of cement) accelerator after 6 hours from mixing. The ACI strength-development-with-time equation (Equation 5-1) can also be applied to high early strength concrete pavement with slight modification to the *s* value parameter. Moreover, split tensile strength tests indicated the possibility of PCCP mixtures with 1% calcium chloride (by dry weight of cement) to have a higher split tensile strength at later ages. Based on data that compared split tensile and compressive strength in this research study, the ratio of split tensile to compressive strength decreases as age and strength increase, as is consistent with normal strength structural concrete according to ACI. However, the relationship between split tensile and compressive strength of high early strength mixes differs from structural concrete, potentially due to chemical admixtures used in concrete pavements. The equation developed in this study for high early strength concrete more accurately predicts spilt tensile strength based on compressive strength for these mixtures.

## 7.3 Dry Shrinkage

A finer pore structure is created in concrete mixtures with 1% calcium chloride (by dry weight of cement) as demonstrated by higher measured shrinkage and reconfirmed by higher surface resistivity values for these mixtures. Further verification of the pore structure for these mixes is needed, however, and autogenous shrinkage should also be considered.

## 7.4 Durability

Surface resistivity measurements generally indicated lower permeability for high early strength mixtures with low cement factors. Since the dosage of calcium chloride was based on the dry cement weight, mixtures with a high cement content also contained a relatively high amount of calcium chloride, allowing the availability of more free chloride ions that could improve conductivity. The Resipod Proceq used in this study measures the electrical resistivity of concrete by taking the potential difference measured between the two inner probes and dividing it by the current that is carried by the ions in the pore liquid (Proceq, 2018). This allows the surface resistivity value of high cement mixtures with 2% calcium chloride (by dry weight of cement) to be low and therefore indicating a greater chance of permeability. Consequently, the surface resistivity test may provide confusing results for these mixtures; therefore, surface resistivity testing is not recommended as a basis for determining permeability with concrete mixtures that contain calcium chloride as an accelerator.

No visible scaling differences between PCCP mixtures were observed after conducting the scaling test. If ASTM C672 is conducted again in the future, a new type of waterproof sealant that can be removed from concrete should be considered to allow mass measurements before and after treatment and quantitative measurements of the mass lost due to scaling for each specimen. Freeze-thaw testing revealed that only PCCP ID 2 passed both the maximum expansion and minimum RDME recommended parameters established by KTMR-22. The mixture showed a 0.0203% expansion and 96.68% RDME, and also demonstrated the second lowest percent change in mass (0.26%) at the end of the testing period.

## 7.5 Recommendations

Recommendations relevant to the results and observations obtained from this work are noted in this section. The PCCP ID 2 mixture resulted in the most optimized performance in the strength and durability testing conducted in this study. Although it had an air content of 9% and a slump of 6.25 in., PCCP ID 2 would most likely be utilized in the field because it is above and below the minimum and maximum air content required by KDOT, respectively. The mixture itself also contains a water reducer classified as both type A and F, indicating normal and high-range water-reducing capabilities and potentially increased slump.

## 7.6 Future Work

This work provides a foundation for future work in the following areas:

- Further optimization of PCCP ID 2 by incorporating lightweight aggregates to be subjected to internal curing and observe effects on durability
- Expansion of the matrix in Table 1-1 by incorporating similar mixes with 0% calcium chloride, allowing increased understanding of the degree of influence of the accelerator to overall durability characteristics of high early strength concrete mixtures

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# Appendix A - Isothermal Calorimetry

KSU 1 Monarch Type III Cement

$\frac{w}{c} = 0.45$

$Daravair\ 1400\ Air\ Entrainer = 2.25\frac{0z}{cwt}$

$ADVA\ 140M\ Water\ Reducer = 19\frac{oz}{cwt}$

Bath Temperature = 23$^{o}$C

1% Calcium Chloride

1) Cement + Water
2) Cement + Water + Air Entrainer
3) Cement + Water + Air Entrainer + Water Reducer
4) Cement + Water + Air Entrainer + Water Reducer + Calcium Chloride

**Table A-1: Isothermal Calorimetry Data for KSU 1**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Cement (grams) | 10.00 | 10.00 | 10.00 | 10.09 |
| Water (grams) | 4.50 | 4.49 | 4.49 | 4.19 |
| Air Entrainer (mg) | - | 15 | 15 | 15 |
| Water Reducer (mg) | - | - | 124 | 125 |
| Calcium Chloride (mg) | - | - | - | 320 |
| Maximum Normalized Heat Generated (mW/g) | 6.61 | 6.41 | 5.86 | 8.13 |
| Time After Start of Test (Hours) | 8.13 | 8.28 | 13.75 | 7.08 |
| Total Heat Within 2 Days (J/g) | 306.96 | 304.05 | 299.28 | 310.30 |

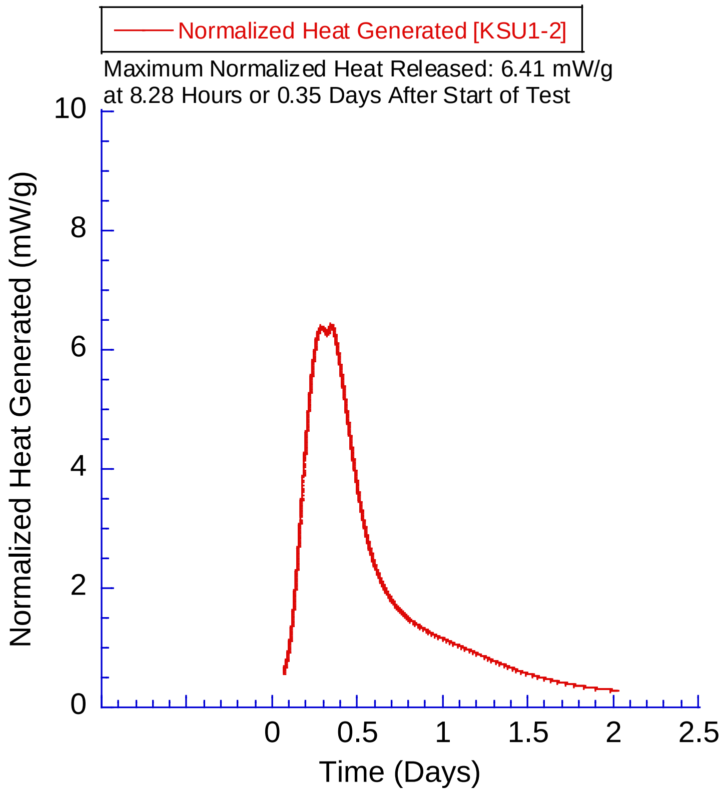



**Figure A-1: Normalized Heat Generated for KSU 1-2**

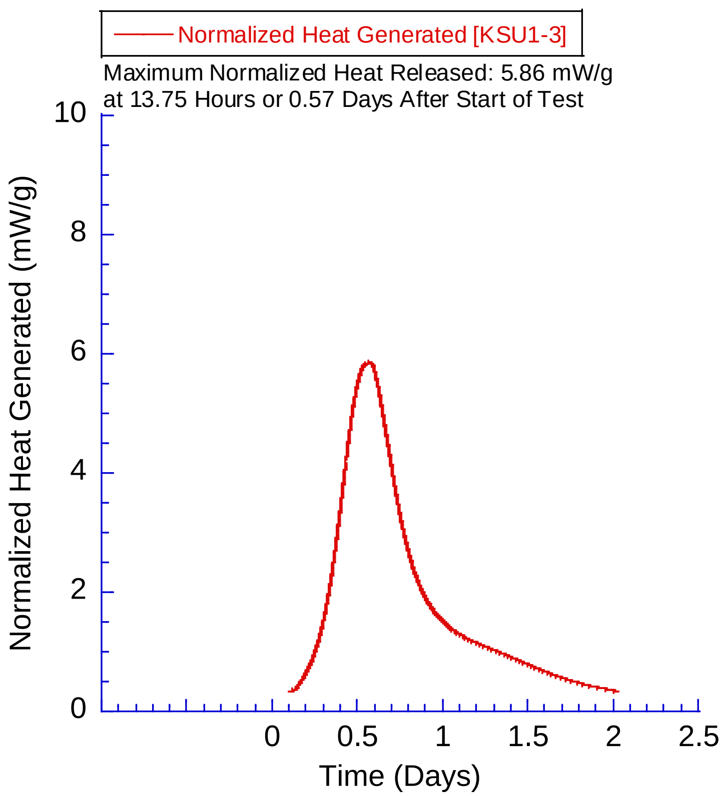



**Figure A-2: Normalized Heat Generated for KSU 1-3**

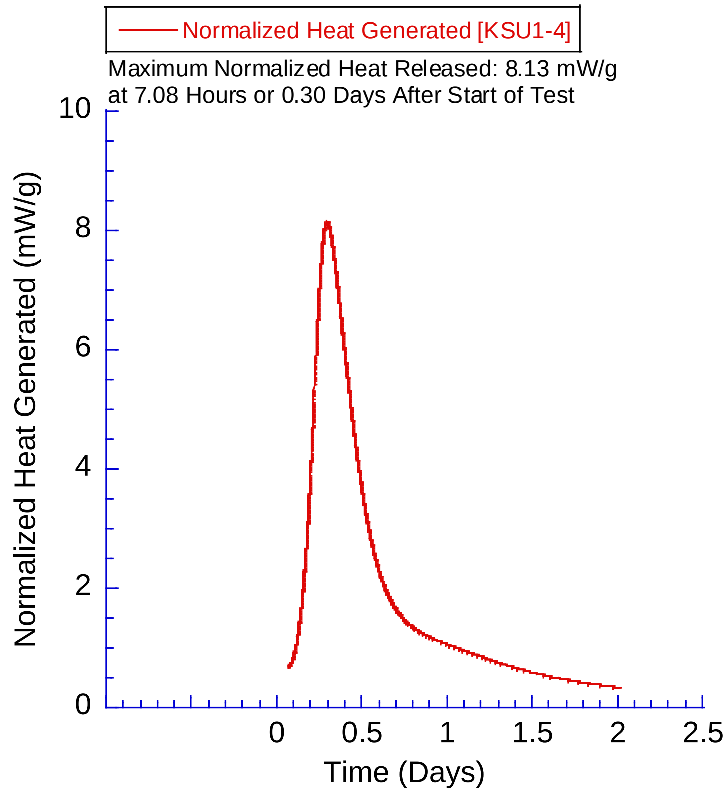



**Figure A-3: Normalized Heat Generated for KSU 1-4**

KSU 2 Monarch Type III Cement

$\frac{w}{c} = 0.45$

$Daravair\ 1400\ Air\ Entrainer = 2.50\,\frac{0z}{cwt}$

$ADVA\ 140M\ Water\ Reducer = 19\,\frac{oz}{cwt}$

Bath Temperature = 23ºC

2% Calcium Chloride

1) Cement + Water

2) Cement + Water + Air Entrainer

3) Cement + Water + Air Entrainer + Water Reducer

4) Cement + Water + Air Entrainer + Water Reducer + Calcium Chloride

**Table A-2: Isothermal Calorimetry Data for KSU 2**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Cement (grams) | 10.00 | 10.00 | 10.00 | 10.18 |
| Water (grams) | 4.50 | 4.48 | 4.36 | 4.00 |
| Air Entrainer (mg) | - | 16 | 16 | 17 |
| Water Reducer (mg) | - | - | 124 | 126 |
| Calcium Chloride (mg) | - | - | - | 640 |
| Maximum Normalized Heat Generated (mW/g) | 6.61 | 6.60 | 5.70 | 12.54 |
| Time After Start of Test (Hours) | 8.13 | 8.11 | 13.29 | 3.86 |
| Total Heat Within 2 Days (J/g) | 306.96 | 306.28 | 297.80 | 310.19 |

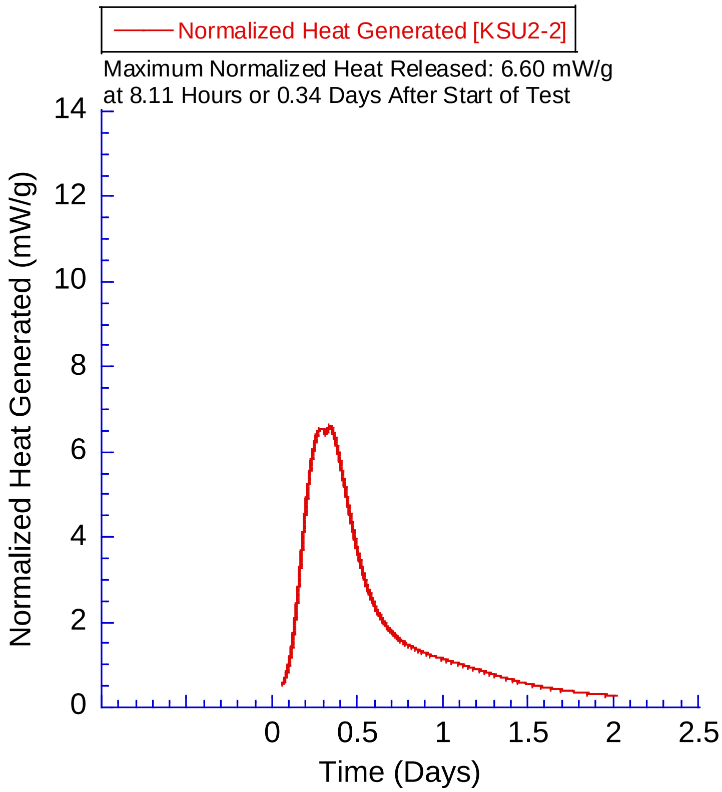



**Figure A-4: Normalized Heat Generated for KSU 2-2**

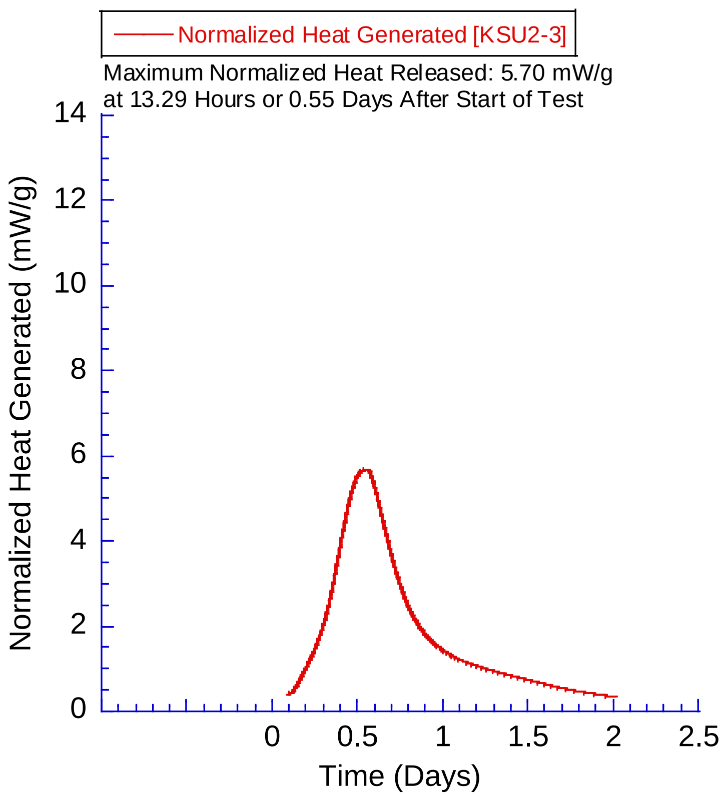



**Figure A-5: Normalized Heat Generated for KSU 2-3**

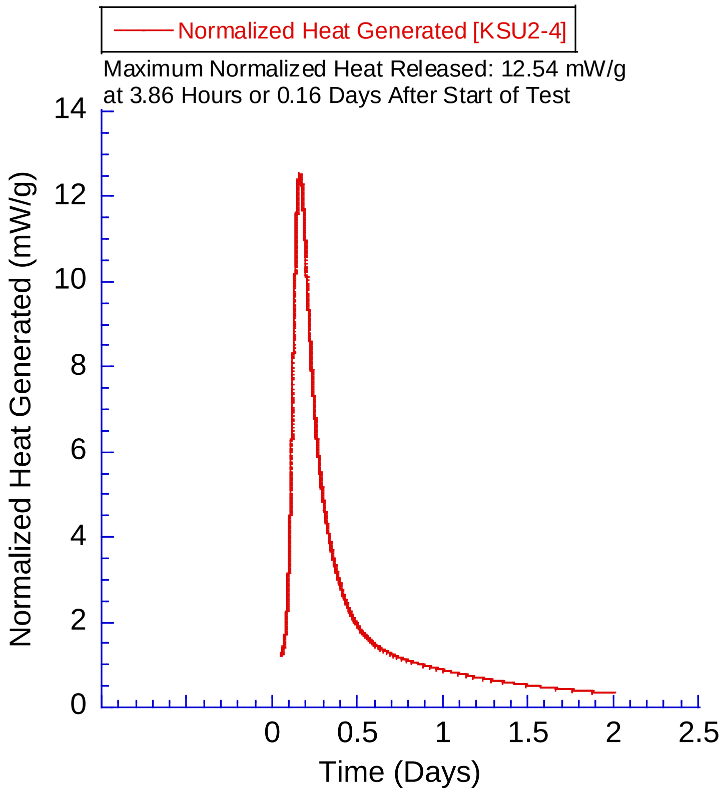



**Figure A-6: Normalized Heat Generated for KSU 2-4**

KSU 3 Monarch Type III Cement

$\frac{w}{c} = 0.45$

$Daravair\ 1400\ Air\ Entrainer = 2.70\,\frac{0z}{cwt}$

$ADVA\ 140M\ Water\ Reducer = 12.25\,\frac{oz}{cwt}$

Bath Temperature = 23ºC

1% Calcium Chloride

1) Cement + Water

2) Cement + Water + Air Entrainer

3) Cement + Water + Air Entrainer + Water Reducer

4) Cement + Water + Air Entrainer + Water Reducer + Calcium Chloride

**Table A-3: Isothermal Calorimetry Data for KSU 3**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Cement (grams) | 10.00 | 10.00 | 10.00 | 10.09 |
| Water (grams) | 4.50 | 4.48 | 4.40 | 4.23 |
| Air Entrainer (mg) | - | 18 | 18 | 18 |
| Water Reducer (mg) | - | - | 80 | 81 |
| Calcium Chloride (mg) | - | - | - | 315 |
| Maximum Normalized Heat Generated (mW/g) | 6.61 | 7.29 | 6.03 | 8.69 |
| Time After Start of Test (Hours) | 8.13 | 7.30 | 10.54 | 6.41 |
| Total Heat Within 2 Days (J/g) | 306.96 | 306.72 | 302.37 | 314.87 |

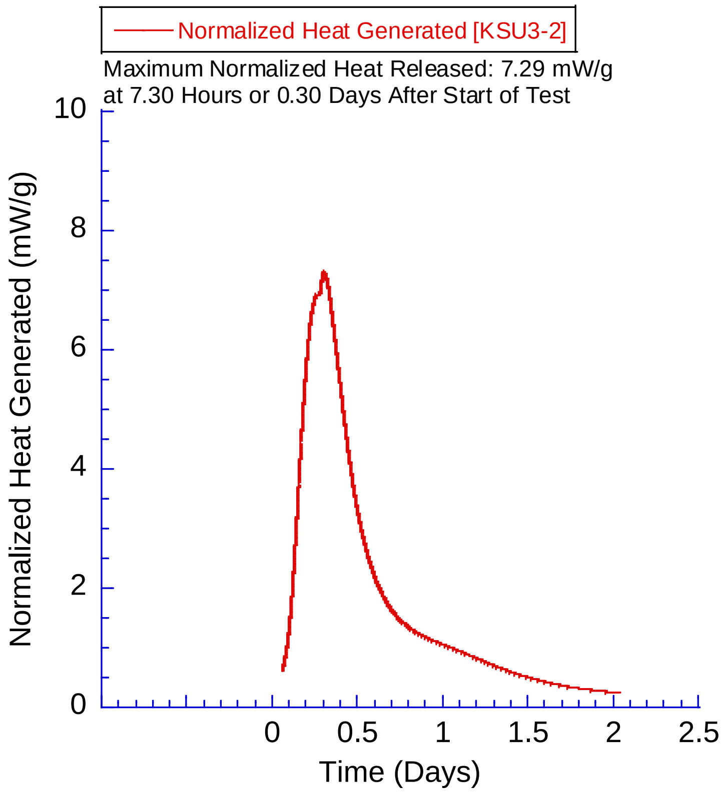



**Figure A-7: Normalized Heat Generated for KSU 3-2**

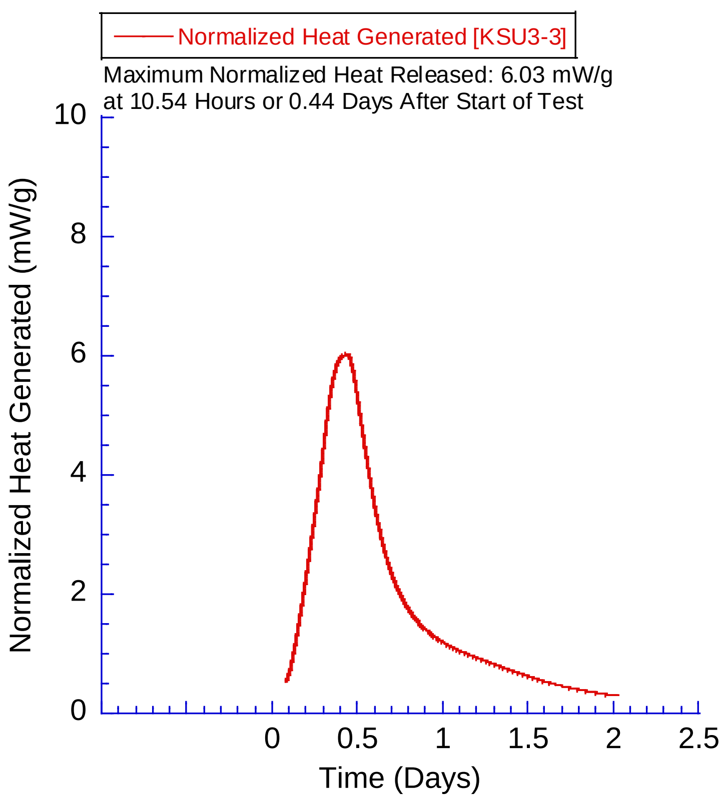



**Figure A-8: Normalized Heat Generated for KSU 3-3**

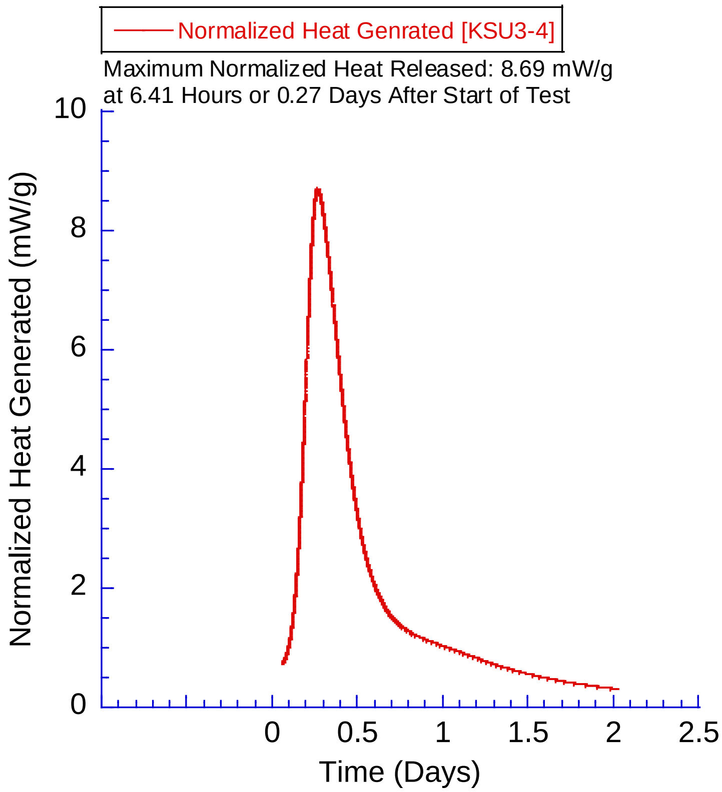



**Figure A-9: Normalized Heat Generated for KSU 3-4**

KSU 4 Monarch Type III Cement

$\frac{w}{c} = 0.45$

$Daravair\ 1400\ Air\ Entrainer = 2.25\frac{0z}{cwt}$

$ADVA\ 140M\ Water\ Reducer = 12.25\frac{oz}{cwt}$

Bath Temperature = 23ºC

2% Calcium Chloride

1) Cement + Water

2) Cement + Water + Air Entrainer

3) Cement + Water + Air Entrainer + Water Reducer

4) Cement + Water + Air Entrainer + Water Reducer + Calcium Chloride

**Table A-4: Isothermal Calorimetry Data for KSU 4**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Cement (grams) | 10.00 | 10.00 | 10.00 | 10.18 |
| Water (grams) | 4.50 | 4.49 | 4.41 | 4.05 |
| Air Entrainer (mg) | - | 15 | 15 | 15 |
| Water Reducer (mg) | - | - | 80 | 81 |
| Calcium Chloride (mg) | - | - | - | 636 |
| Maximum Normalized Heat Generated (mW/g) | 6.61 | 7.40 | 5.23 | 13.93 |
| Time After Start of Test (Hours) | 8.13 | 7.23 | 11.48 | 3.65 |
| Total Heat Within 2 Days (J/g) | 306.96 | 304.39 | 297.36 | 315.31 |

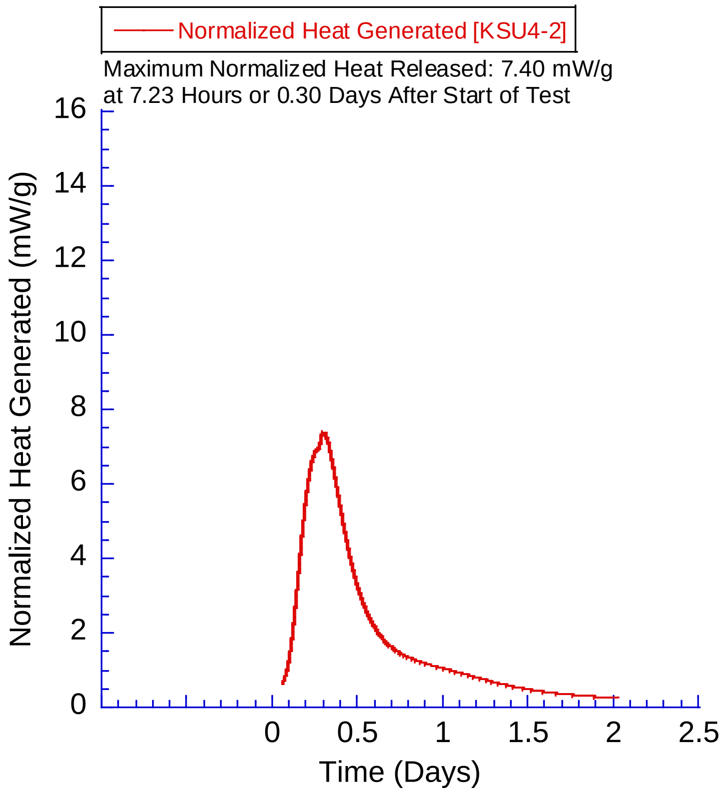



**Figure A-10: Normalized Heat Generated for KSU 4-2**

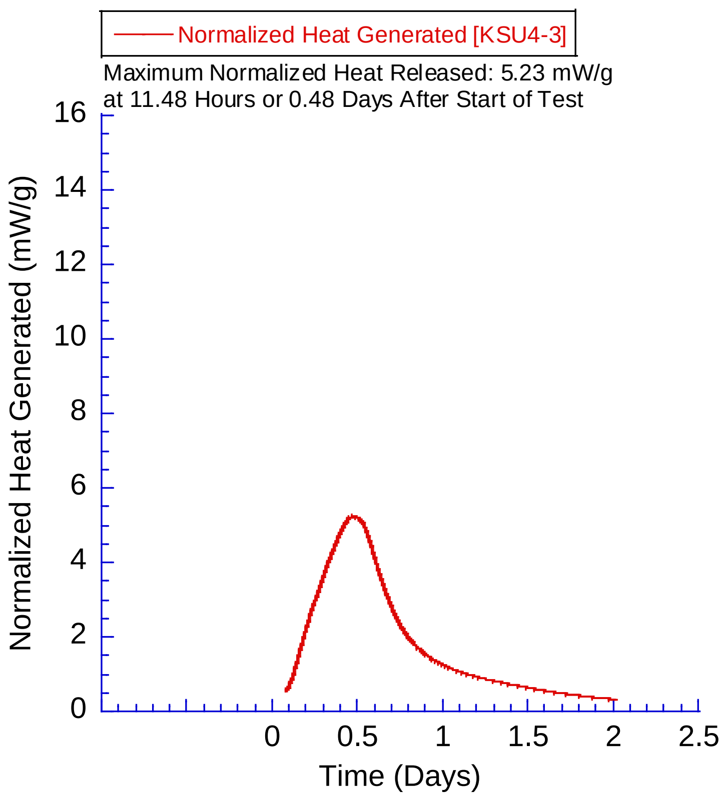



**Figure A-11: Normalized Heat Generated for KSU 4-3**

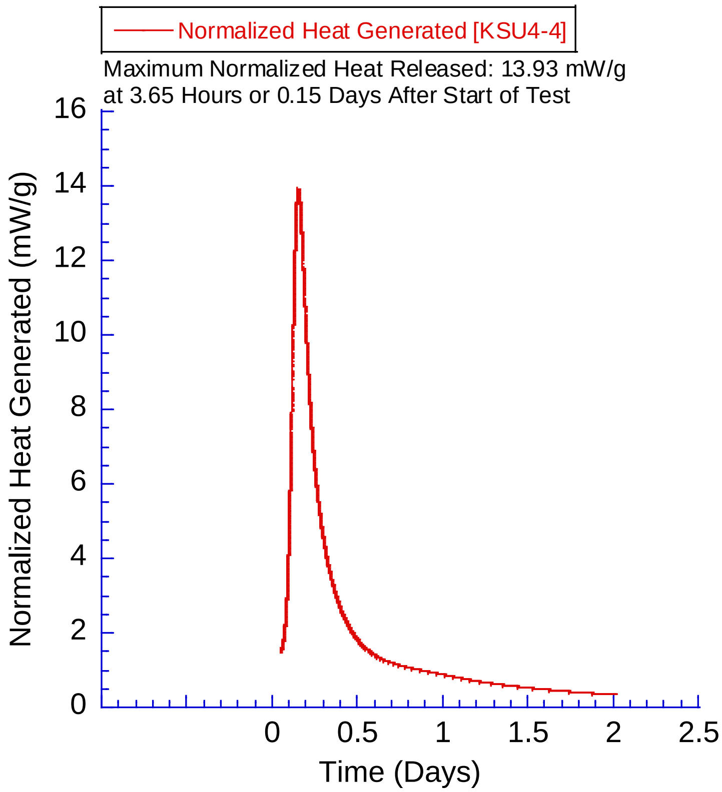



**Figure A-12: Normalized Heat Generated for KSU 4-4**

KSU 5 Monarch Type III Cement

$\frac{w}{c} = 0.45$

$Daravair\ 1400\ Air\ Entrainer = 1.625\,\frac{0z}{cwt}$

$ADVA\ 140M\ Water\ Reducer = 10.125\,\frac{oz}{cwt}$

Bath Temperature = 23$^{o}$C

1% Calcium Chloride

1) Cement + Water

2) Cement + Water + Air Entrainer

3) Cement + Water + Air Entrainer + Water Reducer

4) Cement + Water + Air Entrainer + Water Reducer + Calcium Chloride

**Table A-5: Isothermal Calorimetry Data for KSU 5**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Cement (grams) | 10.00 | 10.00 | 10.00 | 10.09 |
| Water (grams) | 4.50 | 4.49 | 4.42 | 4.25 |
| Air Entrainer (mg) | - | 11 | 11 | 11 |
| Water Reducer (mg) | - | - | 66 | 67 |
| Calcium Chloride (mg) | - | - | - | 320 |
| Maximum Normalized Heat Generated (mW/g) | 6.61 | 7.73 | 7.22 | 10.85 |
| Time After Start of Test (Hours) | 8.13 | 7.02 | 8.24 | 4.50 |
| Total Heat Within 2 Days (J/g) | 306.96 | 305.73 | 301.83 | 319.51 |

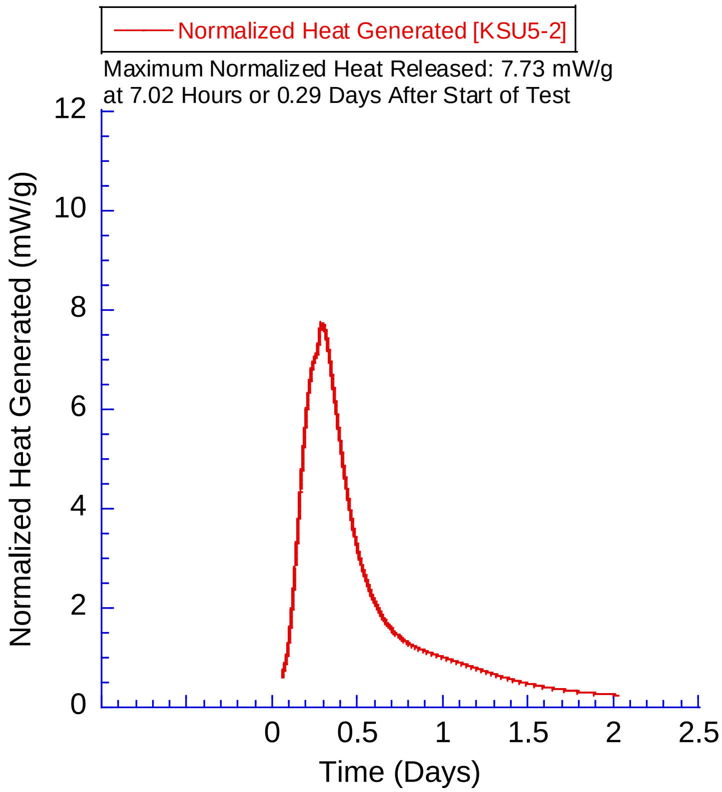



**Figure A-13: Normalized Heat Generated for KSU 5-2**

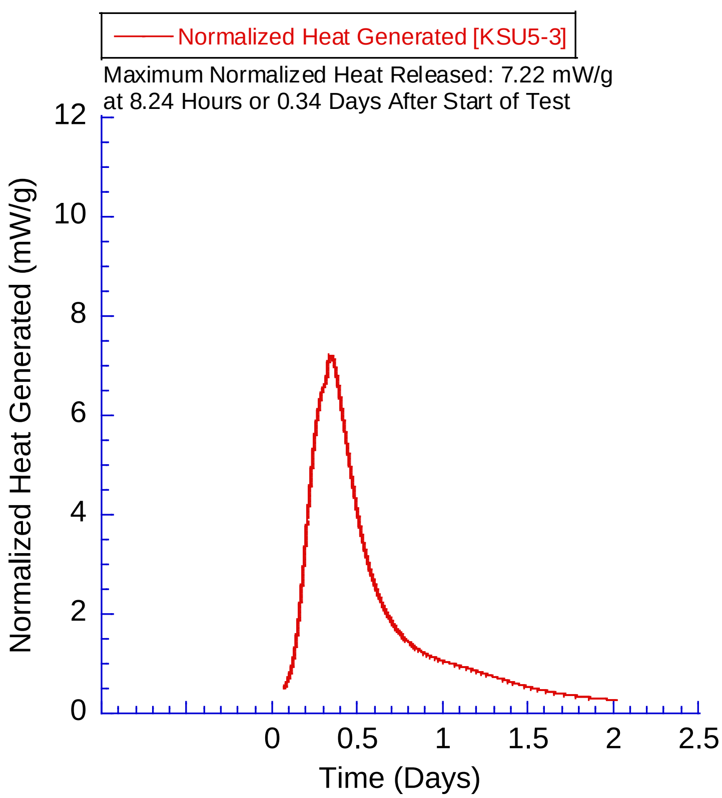



**Figure A-14: Normalized Heat Generated for KSU 5-3**

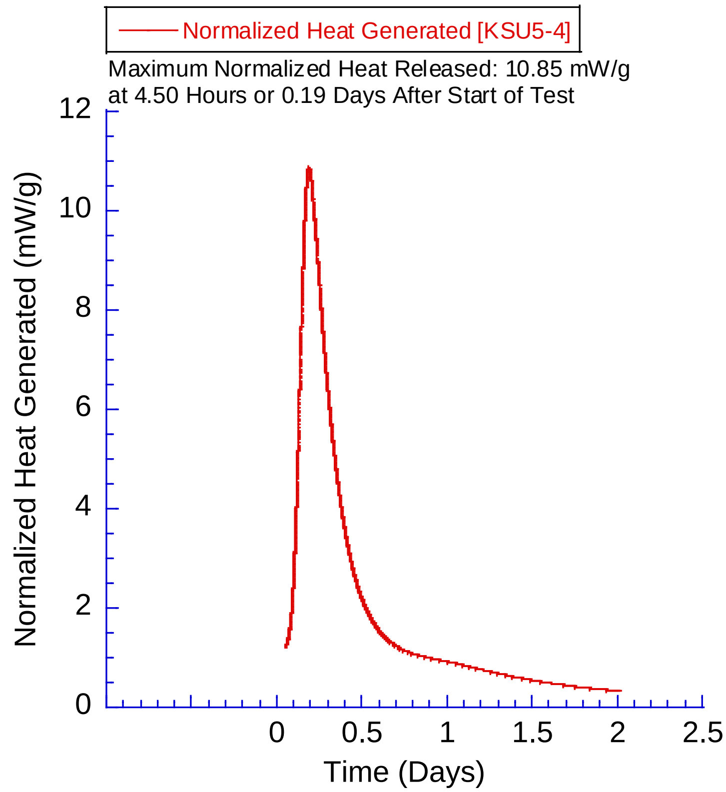



**Figure A-15: Normalized Heat Generated for KSU 5-4**

KSU 6 Monarch Type III Cement

$\frac{w}{c} = 0.45$

$Daravair\ 1400\ Air\ Entrainer = 1.375\,\frac{0z}{cwt}$

$ADVA\ 140M\ Water\ Reducer = 10\,\frac{oz}{cwt}$

Bath Temperature = 23°C

2% Calcium Chloride

1) Cement + Water

2) Cement + Water + Air Entrainer

3) Cement + Water + Air Entrainer + Water Reducer

4) Cement + Water + Air Entrainer + Water Reducer + Calcium Chloride

**Table A-6: Isothermal Calorimetry Data for KSU 6**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Cement (grams) | 10.00 | 10.00 | 10.00 | 10.18 |
| Water (grams) | 4.50 | 4.49 | 4.43 | 4.07 |
| Air Entrainer (mg) | - | 9 | 9 | 9 |
| Water Reducer (mg) | - | - | 65 | 66 |
| Calcium Chloride (mg) | - | - | - | 640 |
| Maximum Normalized Heat Generated (mW/g) | 6.61 | 7.70 | 7.34 | 14.87 |
| Time After Start of Test (Hours) | 8.13 | 6.81 | 9.23 | 3.23 |
| Total Heat Within 2 Days (J/g) | 306.96 | 300.38 | 300.91 | 319.70 |

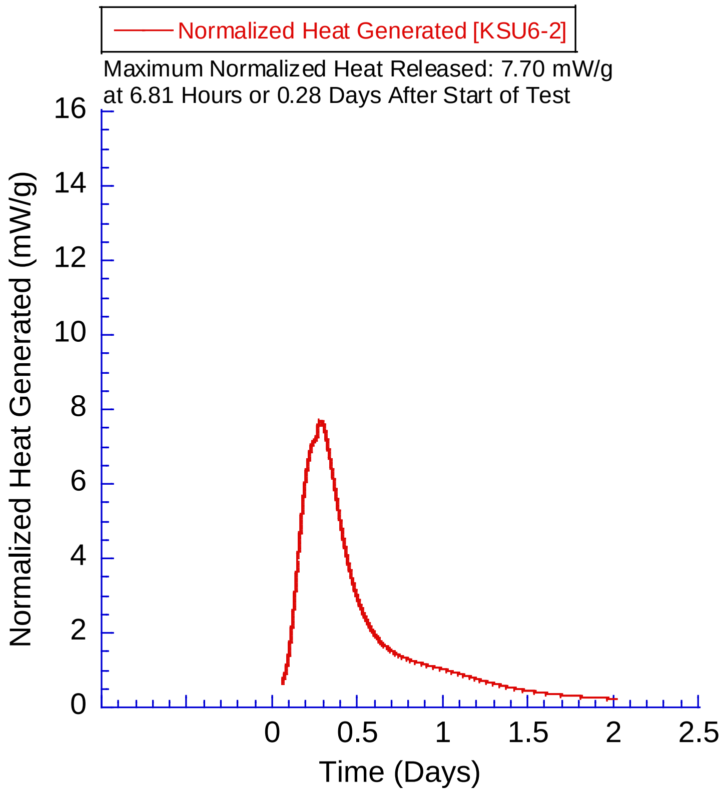



**Figure A-16: Normalized Heat Generated for KSU 6-2**

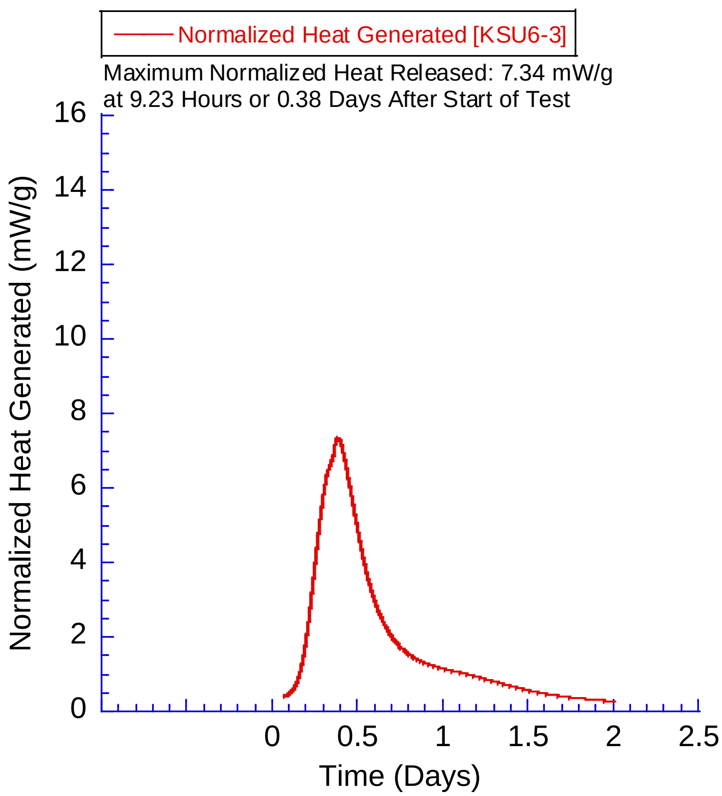



**Figure A-17: Normalized Heat Generated for KSU 6-3**

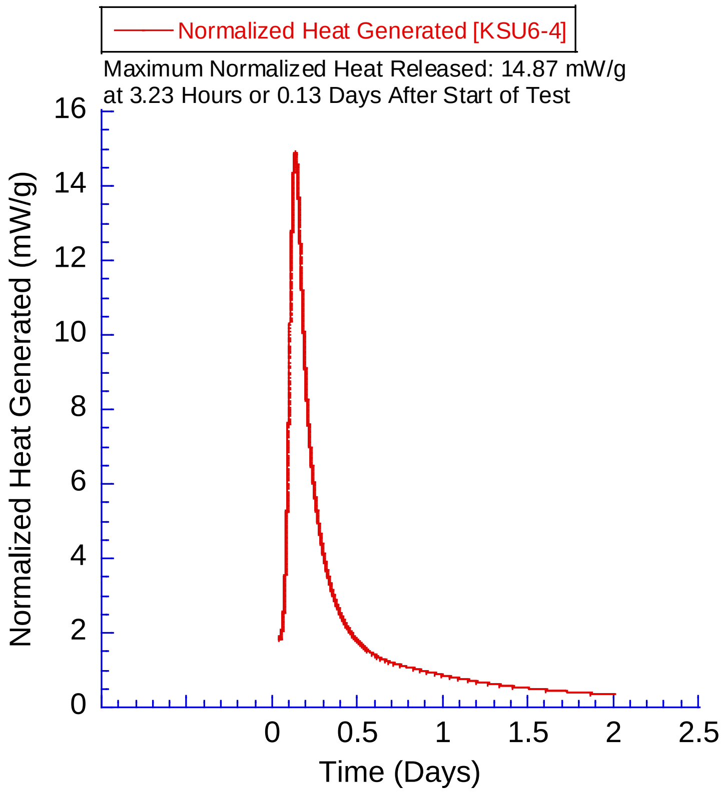



**Figure A-18: Normalized Heat Generated for KSU 6-4**

# Appendix B - Average Compressive Strength Versus Time

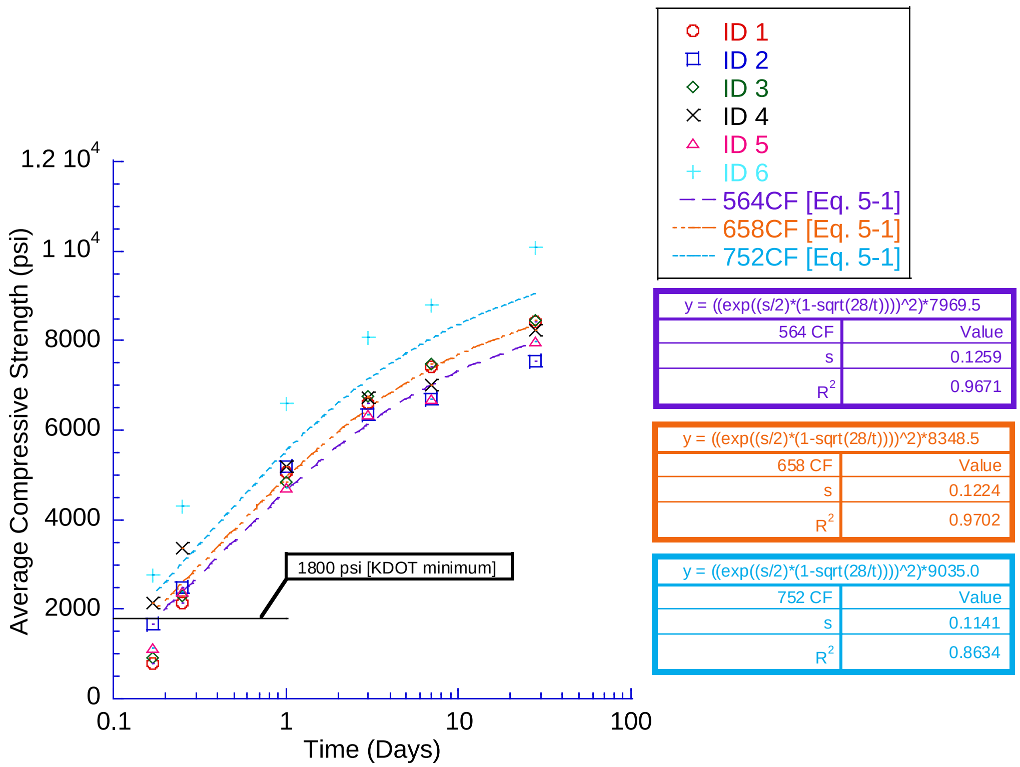



**Figure B-1: Compressive Strength Development with Time Based on Cement Factor**

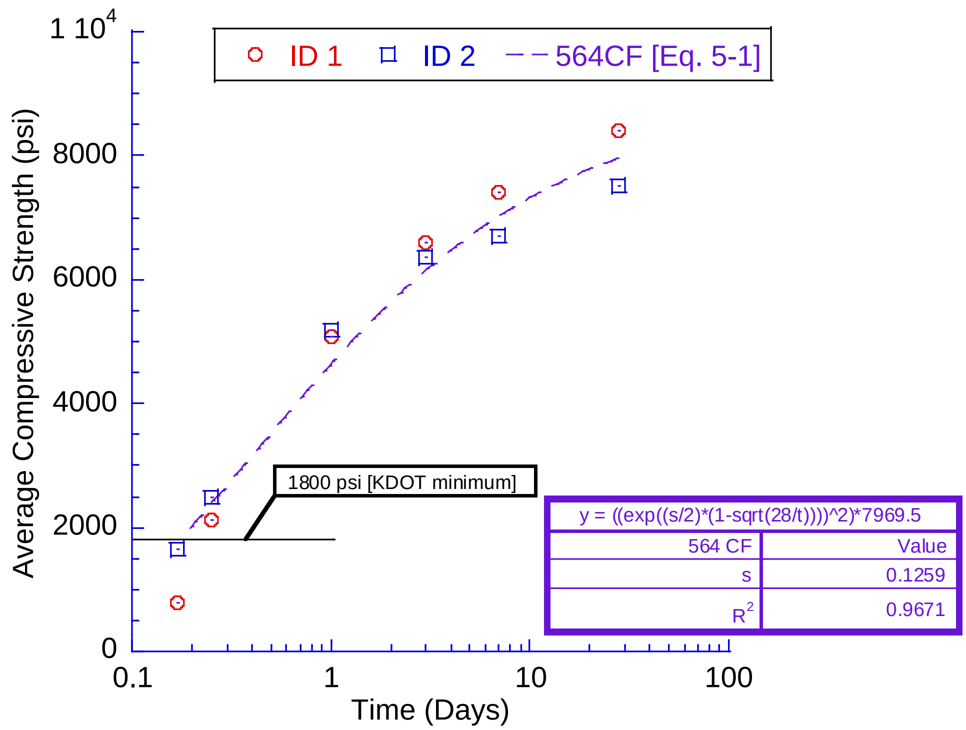



**Figure B-2: Compressive Strength Development with Time Based on 564 lb/yd$^3$ Cement Factor**

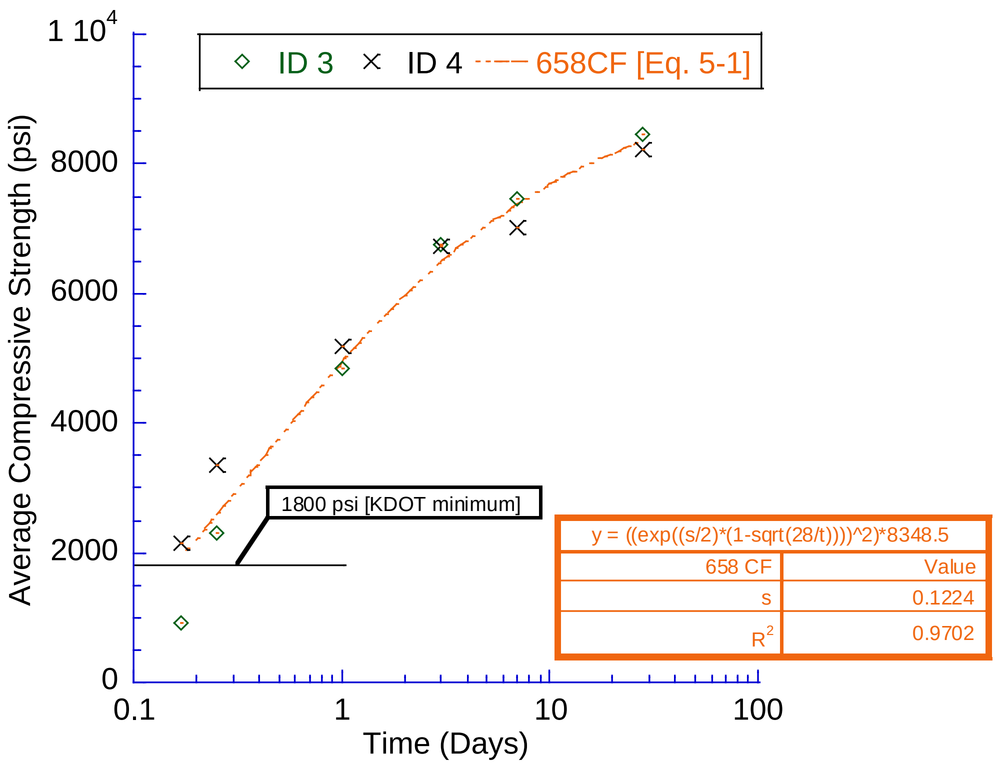



**Figure B-3: Compressive Strength Development with Time Based on 658 lb/yd³ Cement Factor**

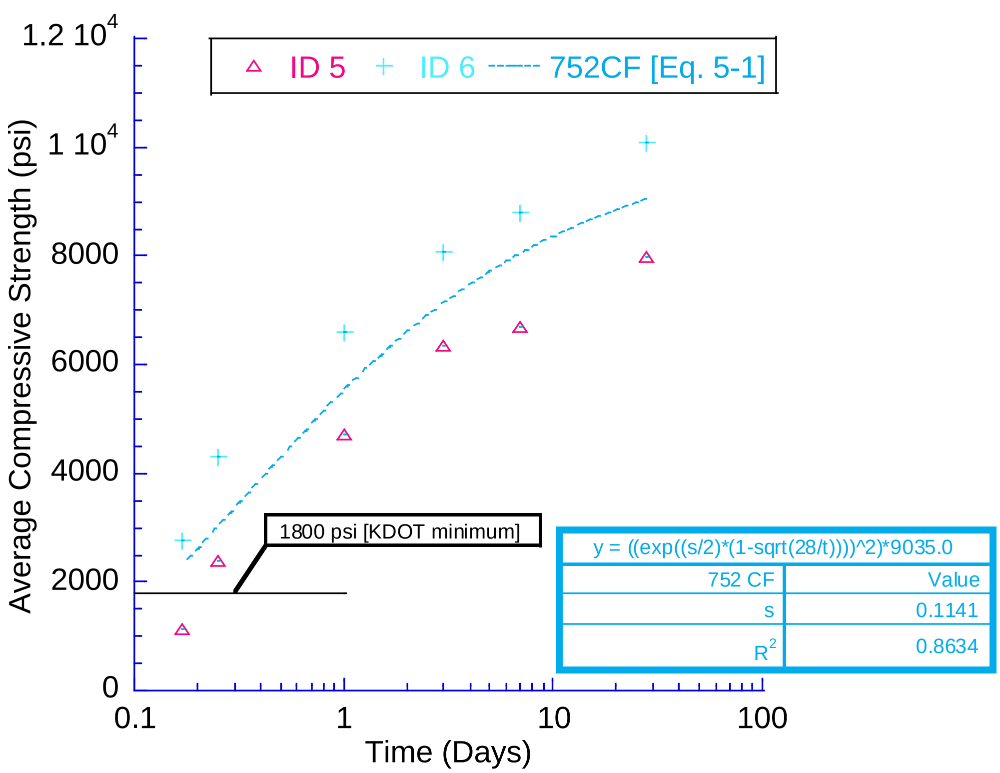



**Figure B-4: Compressive Strength Development with Time Based on 752 lb/yd³ Cement Factor**

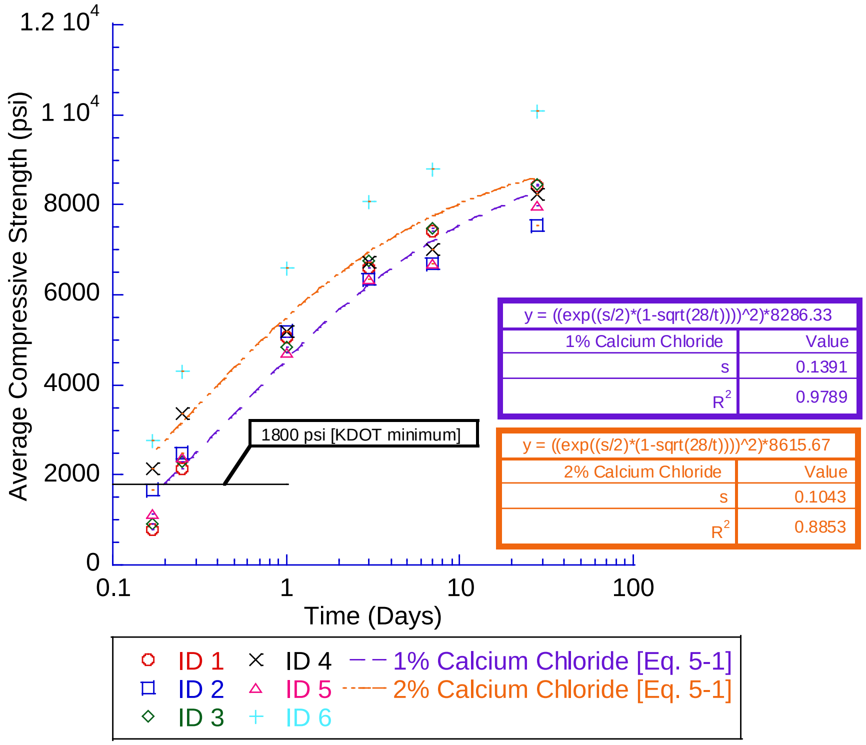



**Figure B-5: Compressive Strength Development with Time Based on Calcium Chloride Content**

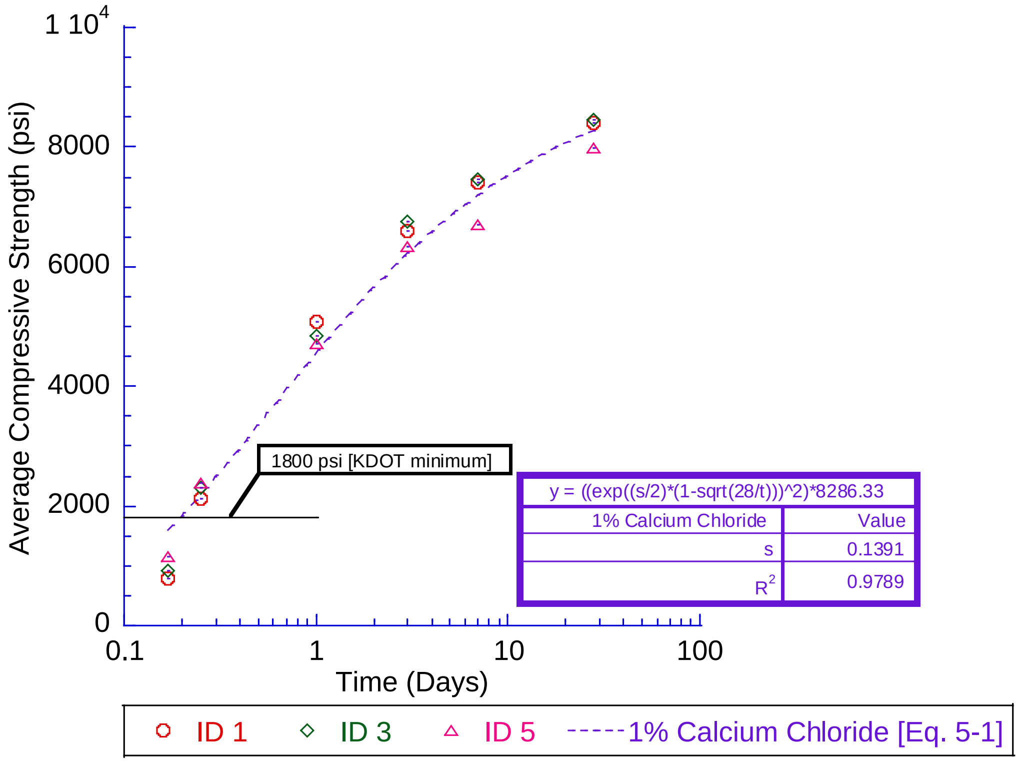



**Figure B-6: Compressive Strength Development with Time Based on 1% Calcium Chloride**

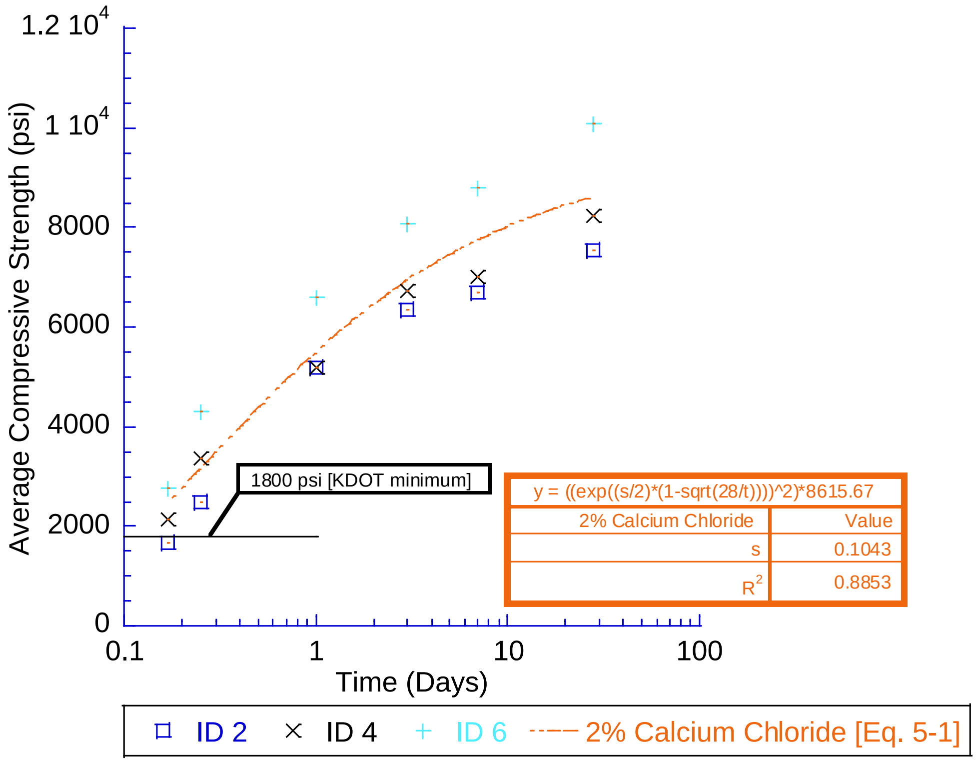



**Figure B-7: Compressive Strength Development with Time Based on 2% Calcium Chloride**

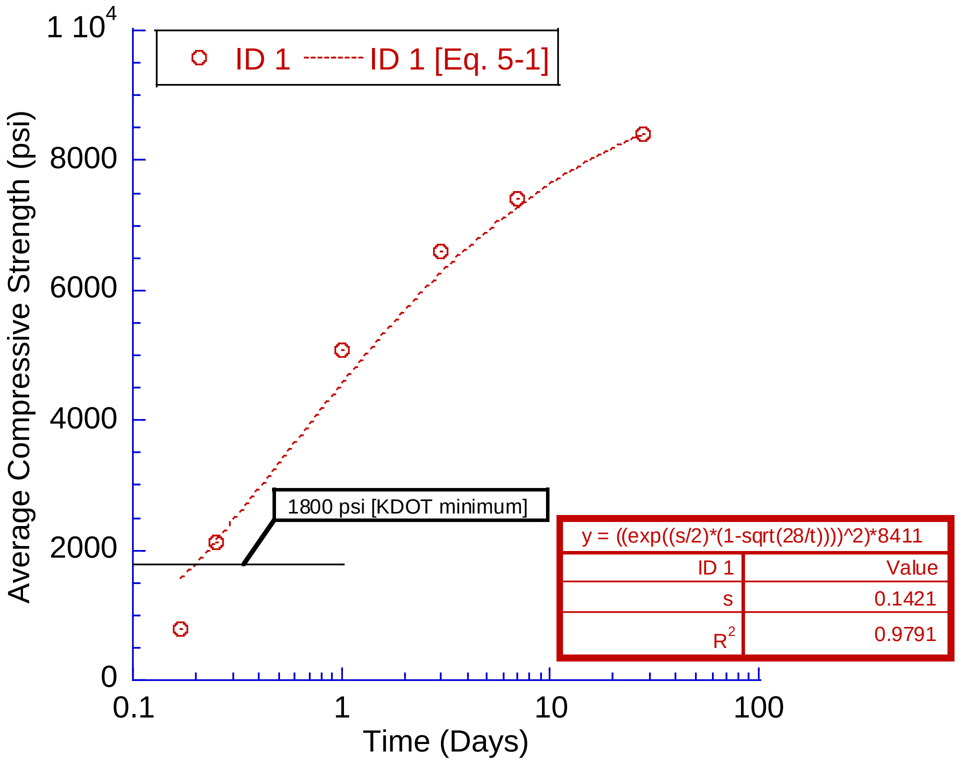



**Figure B-8: Compressive Strength Development with Time for PCCP ID 1**

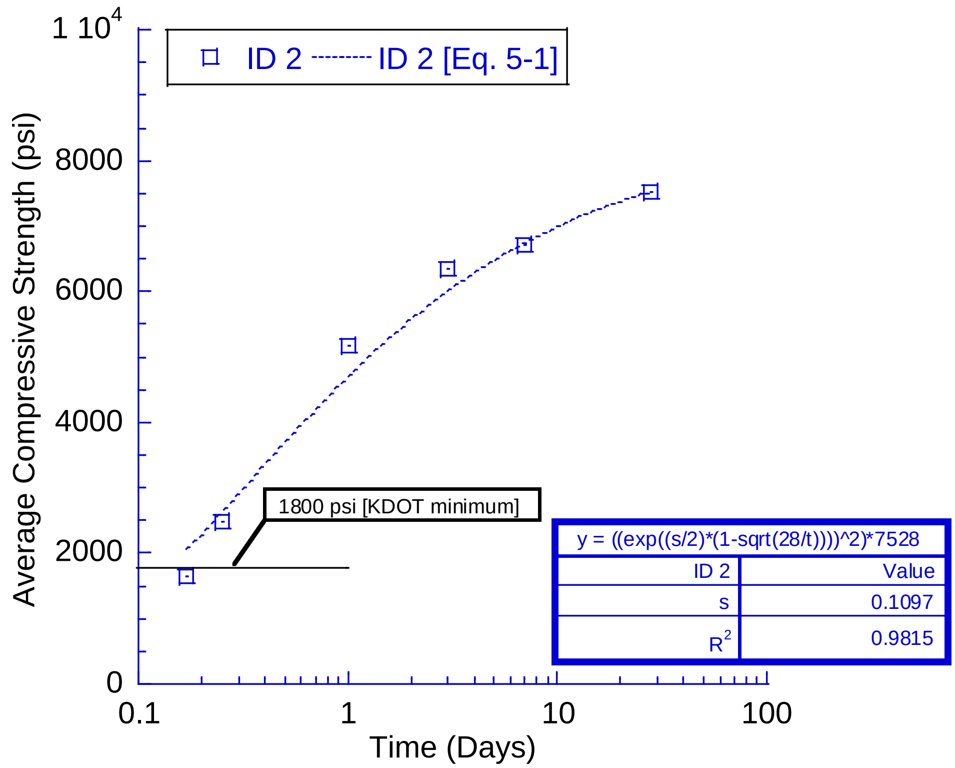



**Figure B-9: Compressive Strength Development with Time for PCCP ID 2**

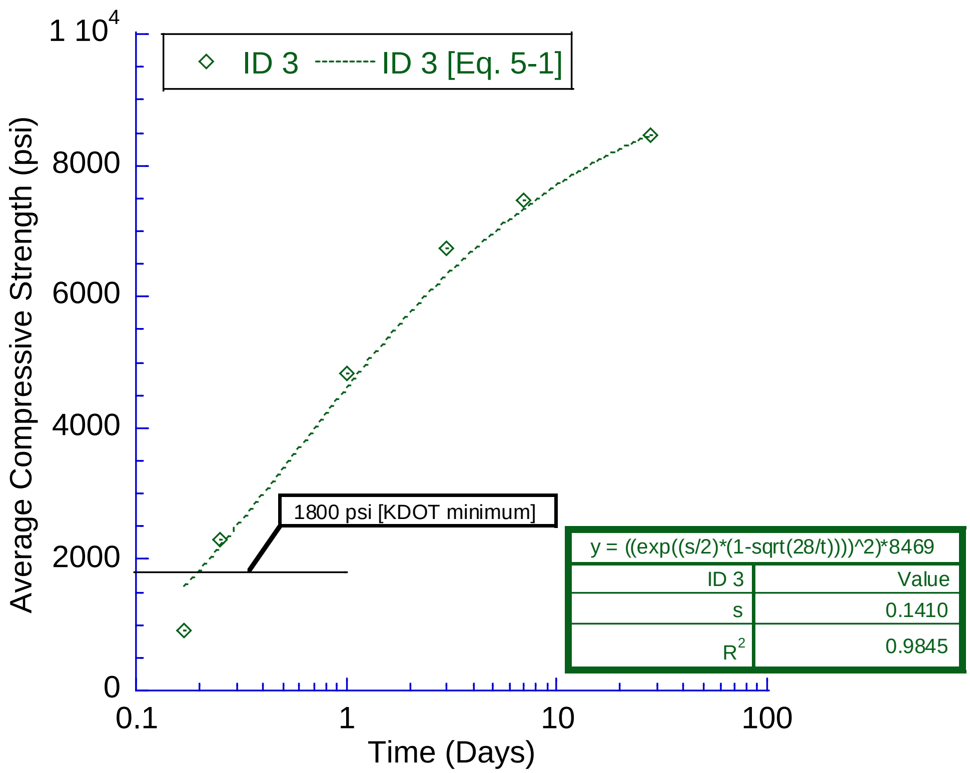



**Figure B-10: Compressive Strength Development with Time for PCCP ID 3**

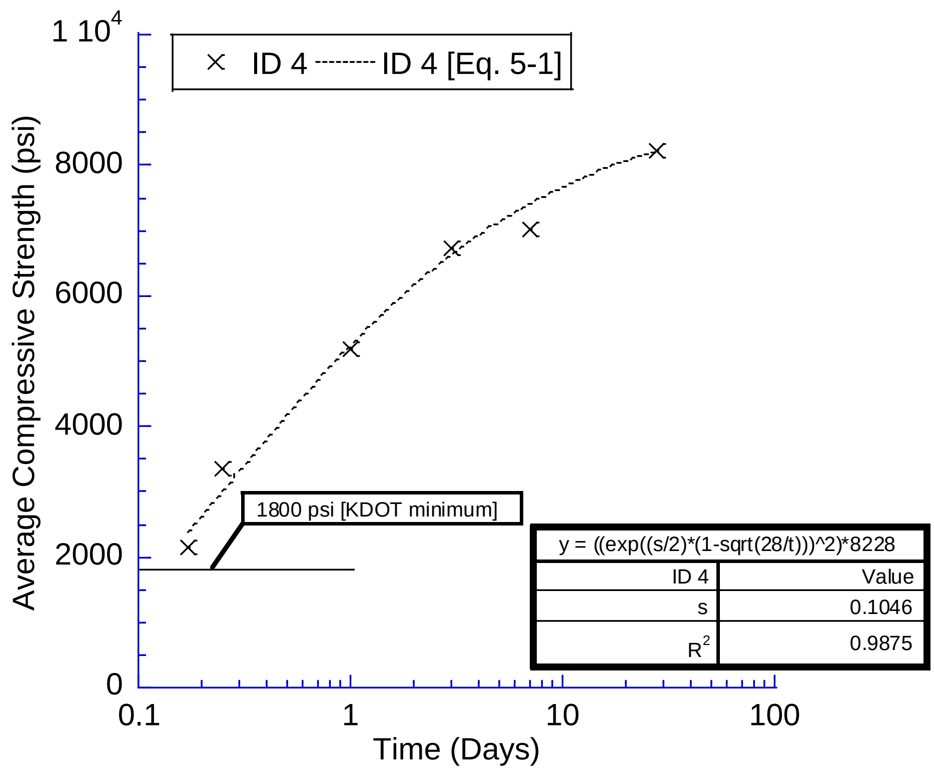



**Figure B-11: Compressive Strength Development with Time for PCCP ID 4**

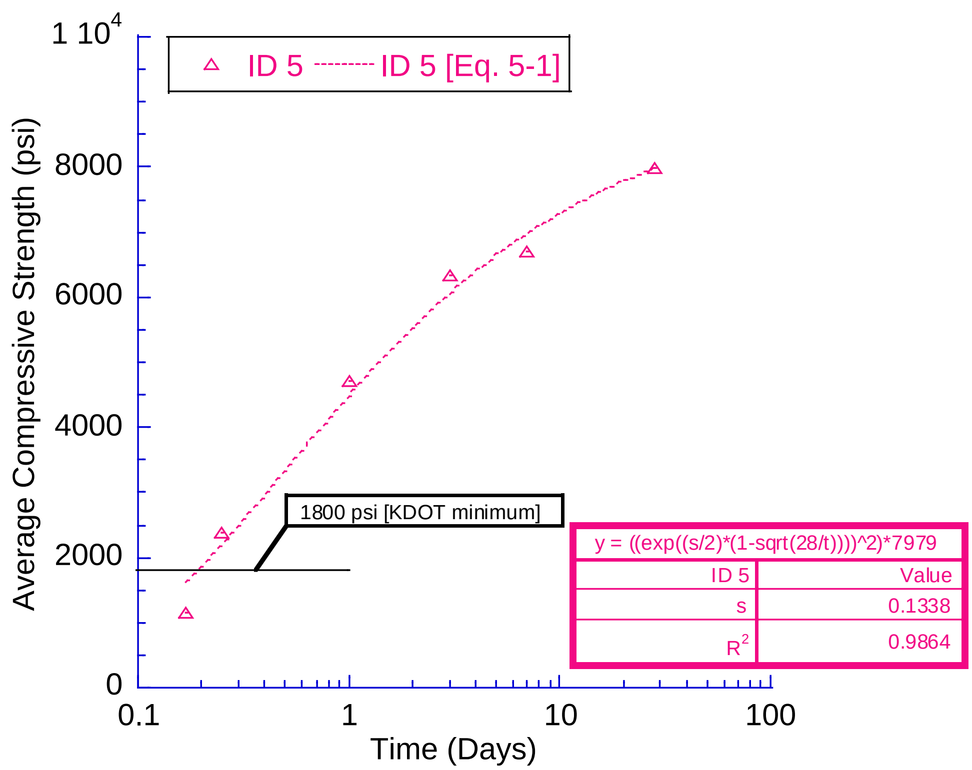



**Figure B-12: Compressive Strength Development with Time for PCCP ID 5**

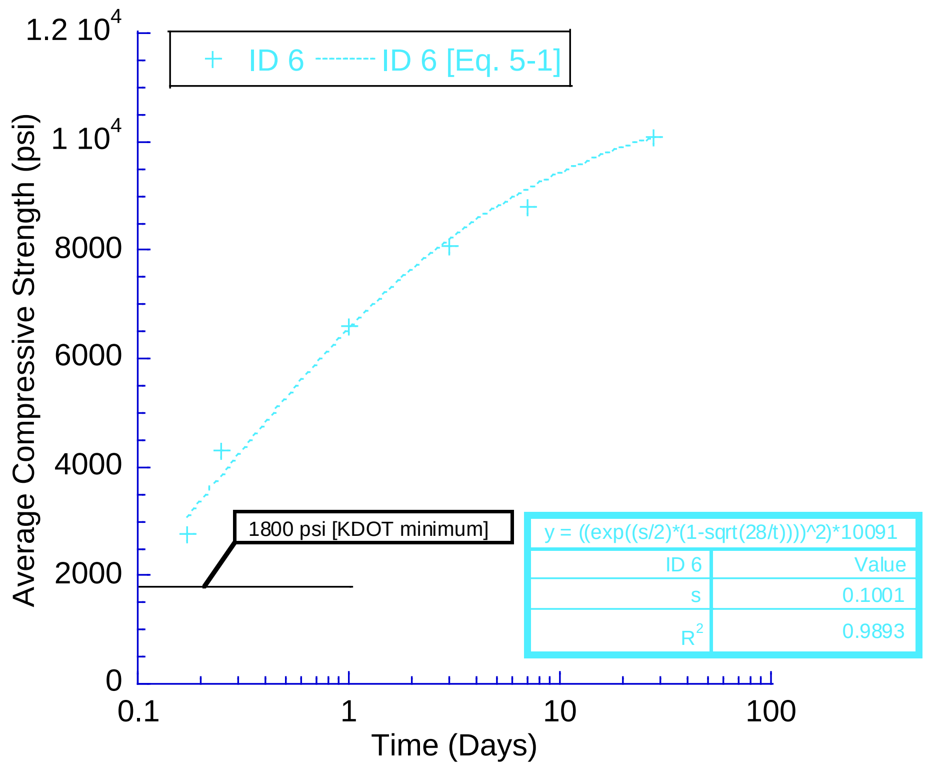



**Figure B-13: Compressive Strength Development with Time for PCCP ID 6**

# Appendix C - Splitting Tensile Versus Compressive Strength

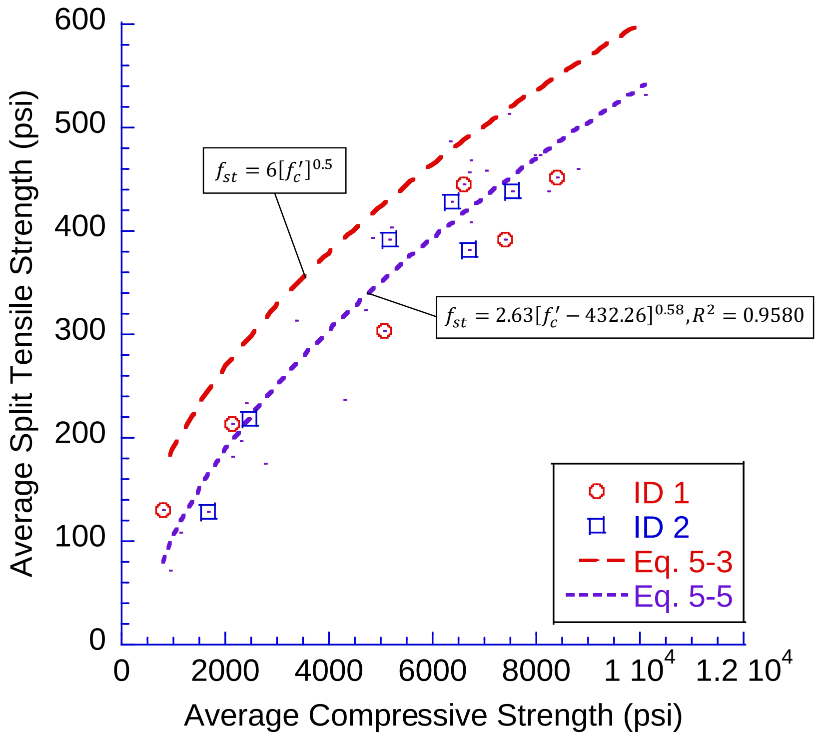



**Figure C-1: Split Tensile Versus Compressive Strength Based on 564 lb/yd³ Cement Factor**

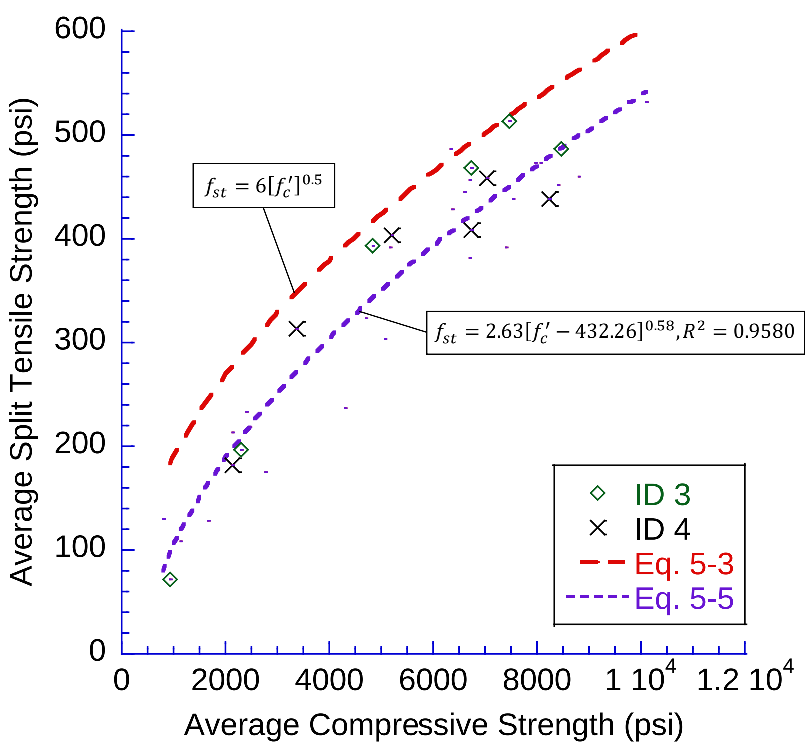



**Figure C-2: Split Tensile Versus Compressive Strength Based on 658 lb/yd³ Cement Factor**

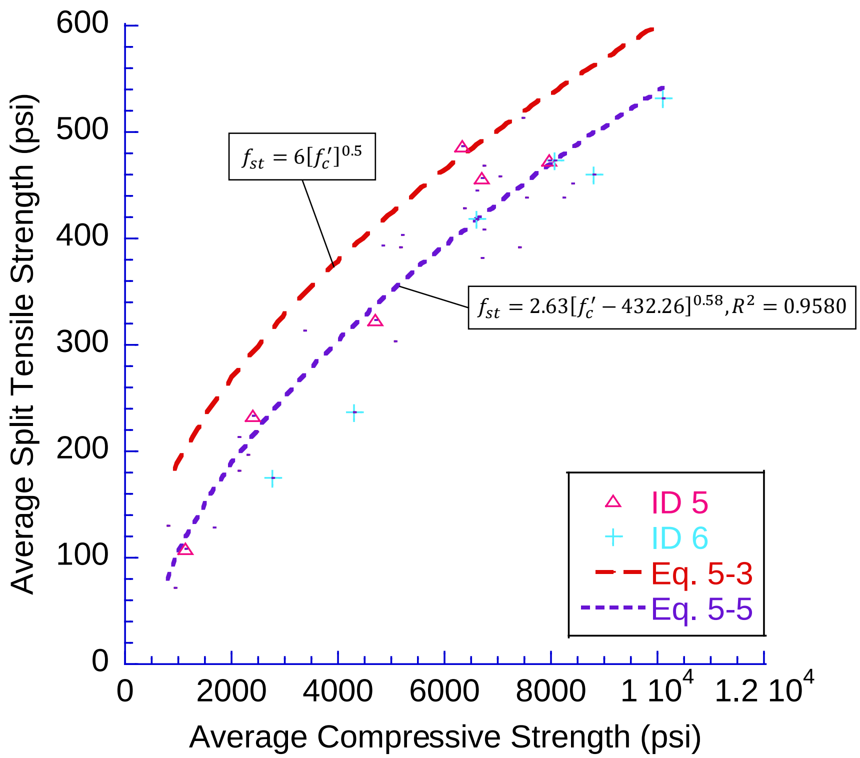



**Figure C-3: Split Tensile Versus Compressive Strength Based on 752 lb/yd³ Cement Factor**

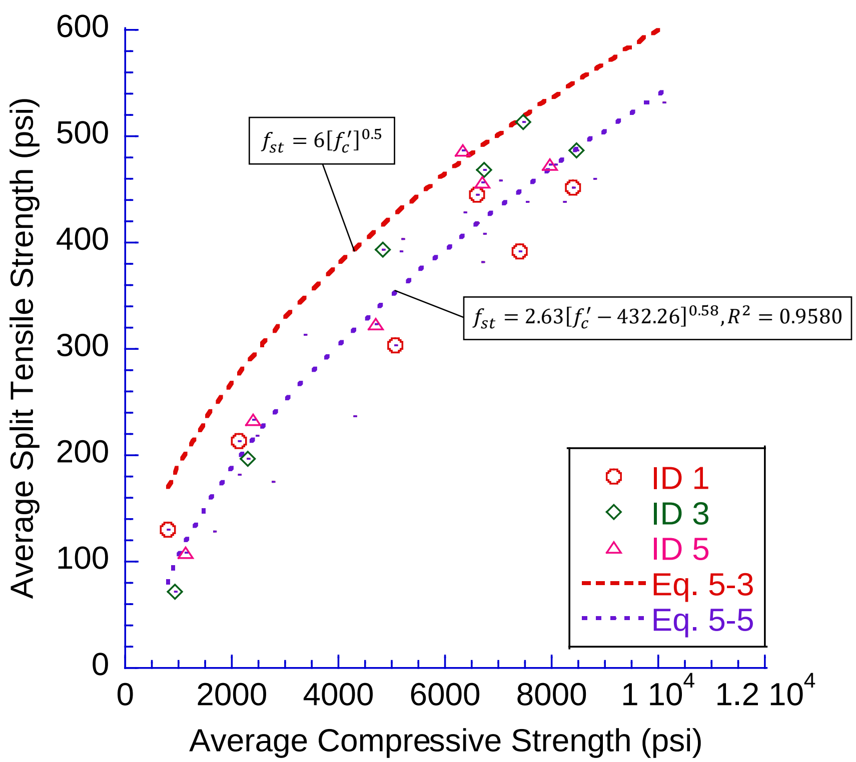



**Figure C-4: Split Tensile Versus Compressive Strength Based on 1% Calcium Chloride**

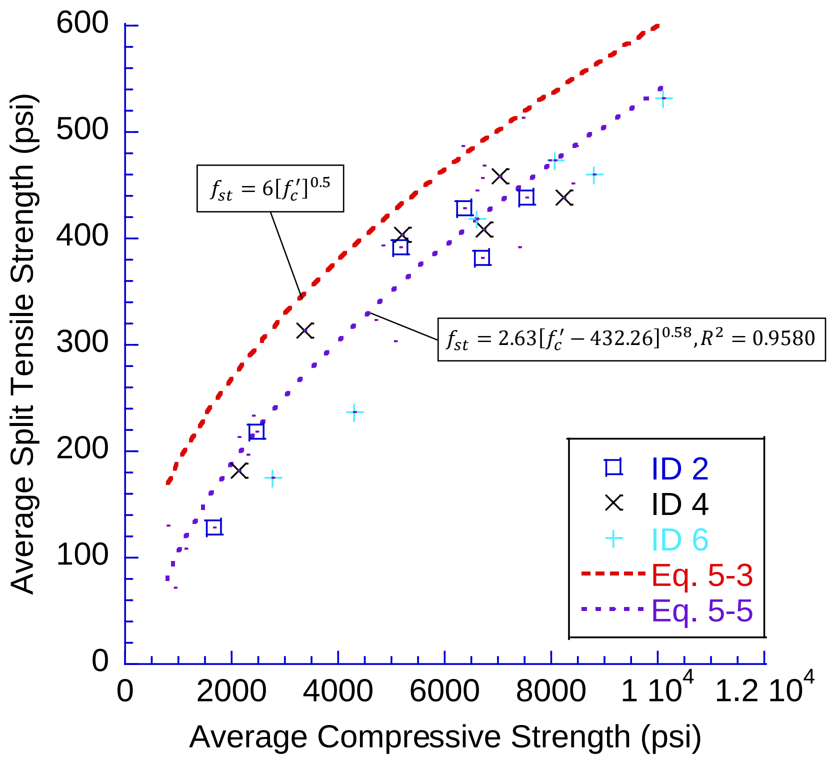



**Figure C-5: Split Tensile Versus Compressive Strength Based on 2% Calcium Chloride**

# Appendix D - Drying Shrinkage

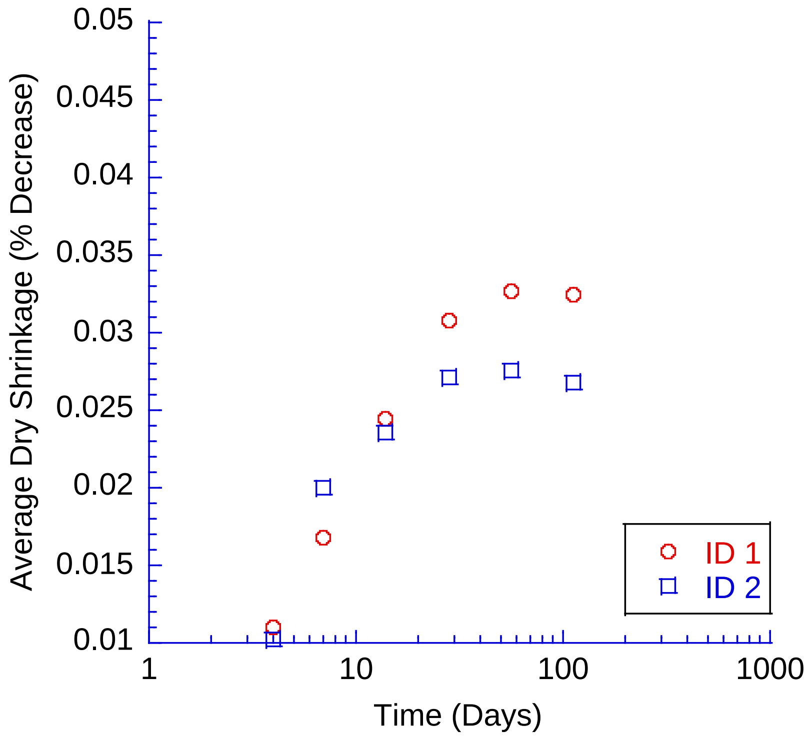



**Figure D-1: Average Dry Shrinkage Based on 564 lb/yd$^3$ Cement Factor**

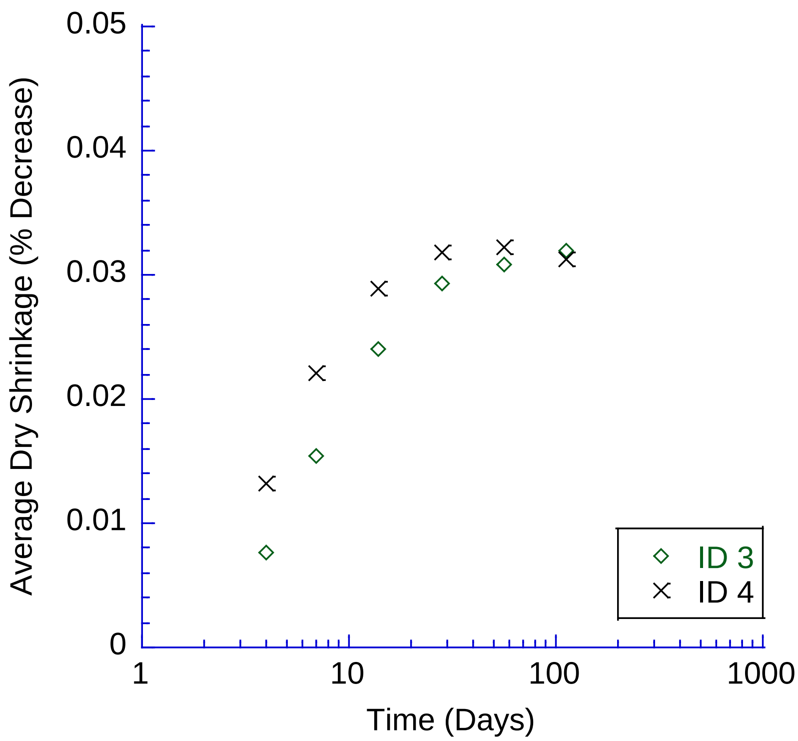



**Figure D-2:Average Dry Shrinkage Based on 658 lb/yd$^3$ Cement Factor**

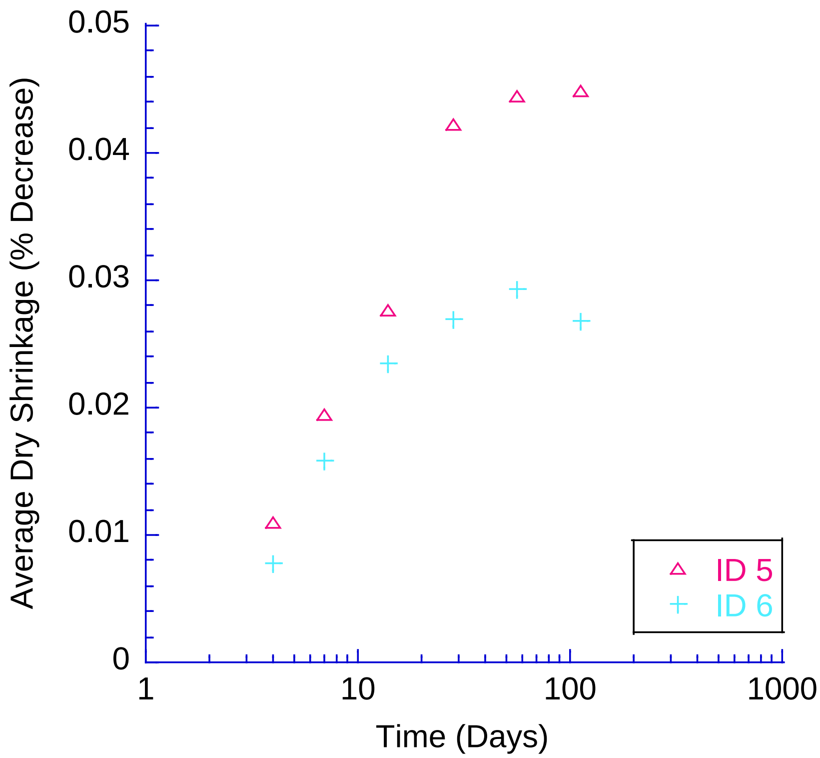



**Figure D-3:Average Dry Shrinkage Based on 752 lb/yd$^3$ Cement Factor**

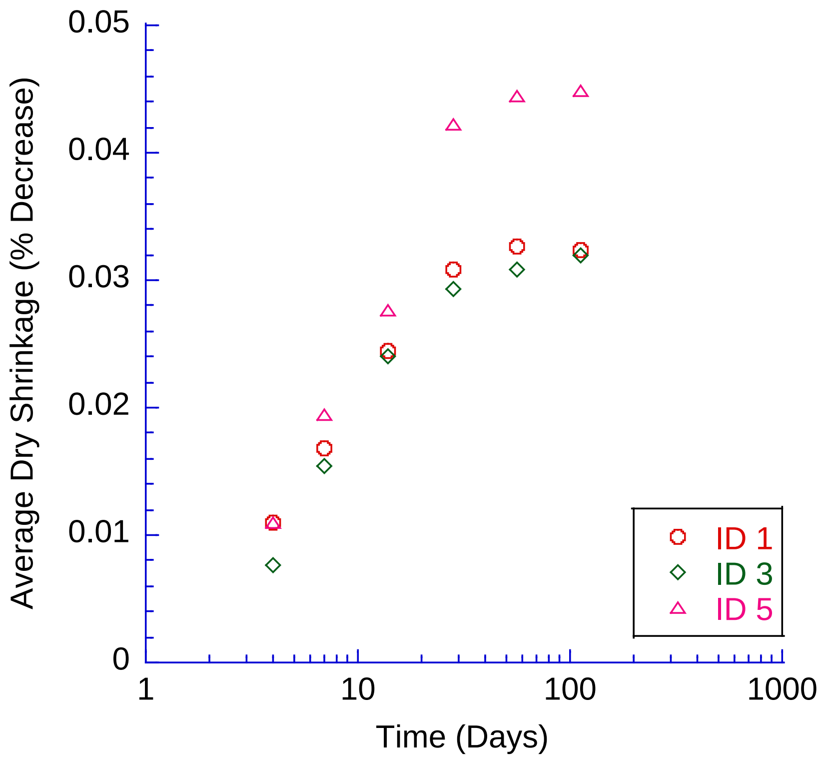



**Figure D-4:Average Dry Shrinkage Based on 1% Calcium Chloride**

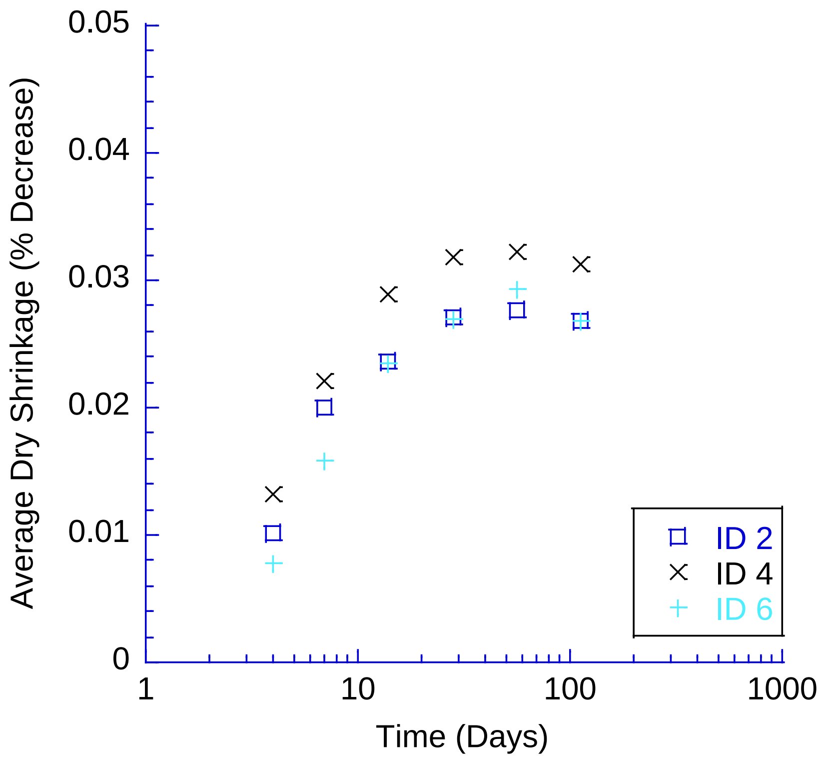



**Figure D-5: Average Dry Shrinkage Based on 2% Calcium Chloride**

# Appendix E - Surface Resistivity

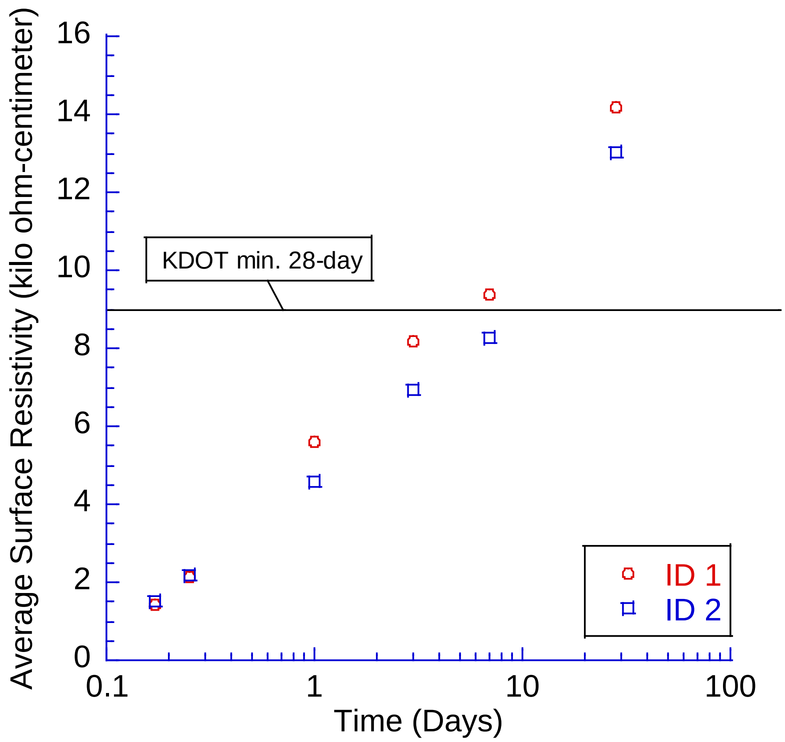



**Figure E-1: Surface Resistivity Development with Time Based on 564 lb/yd³ Cement Factor**

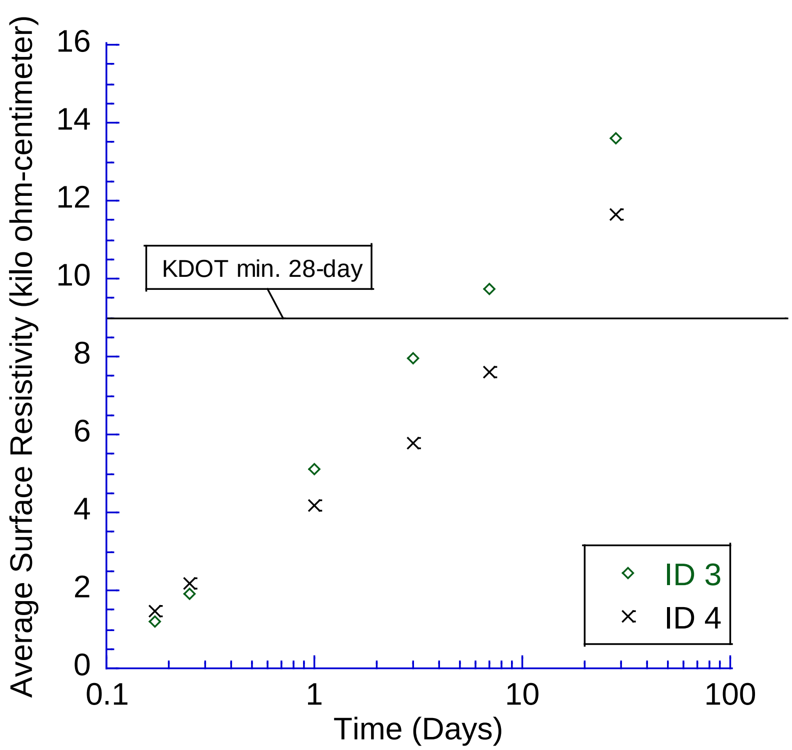



**Figure E-2: Surface Resistivity Development with Time Based on 658 lb/yd³ Cement Factor**

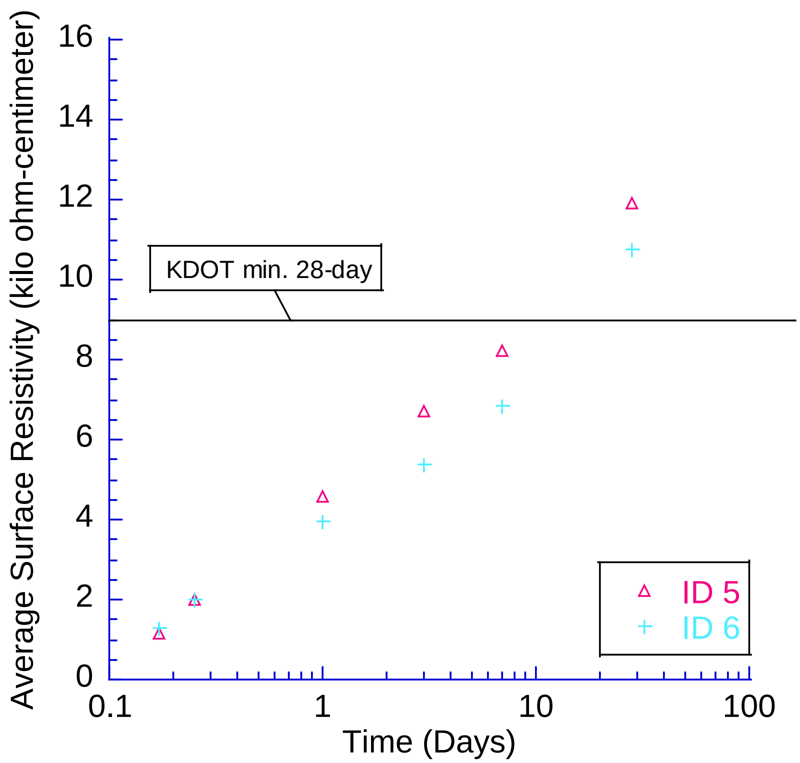



**Figure E-3: Surface Resistivity Development with Time Based on 752 lb/yd$^3$ Cement Factor**

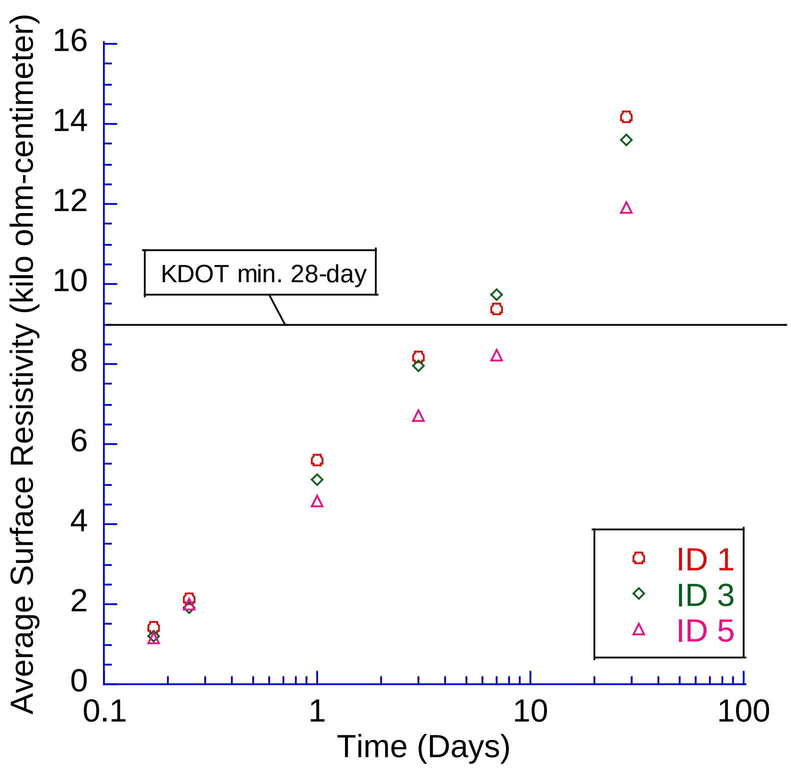



**Figure E-4: Surface Resistivity Development with Time Based on 1% Calcium Chloride**

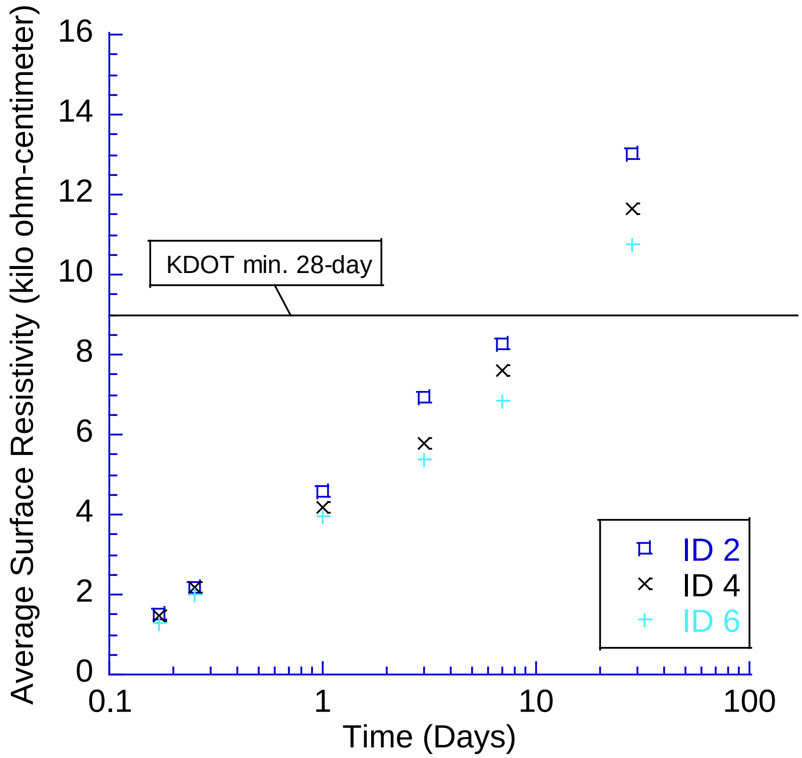


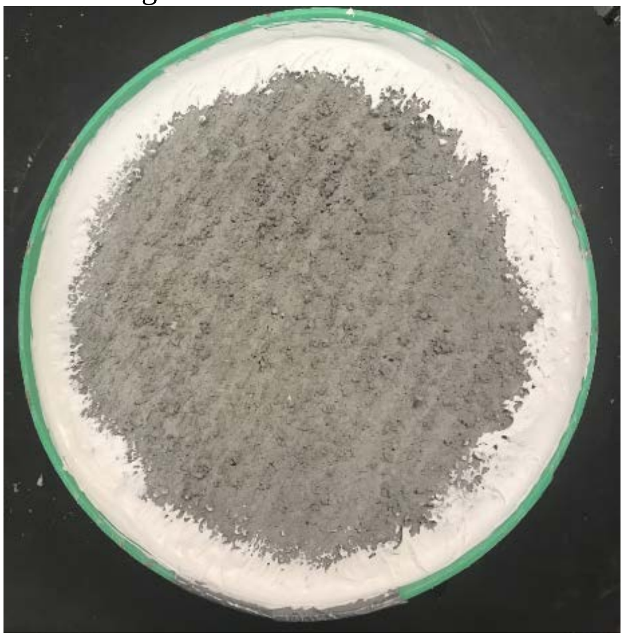

**Figure E-5: Surface Resistivity Development with Time Based on 2% Calcium Chloride**

# Appendix F - Scaling Resistance

PCCP ID 1

Cycle 0

Start Rating: 0



Cycle 55

End Rating: 2

PCCP ID 2

Cycle 0

Start Rating: 0

Cycle 55

End Rating: 2

PCCP ID 3

Cycle 0

Start Rating: 0

Cycle 55

End Rating: 2

PCCP ID 4

Cycle 0

Start Rating: 0

Cycle 55

End Rating: 2

PCCP ID 5

Cycle 0

Start Rating: 0

Cycle 55

End Rating: 2

PCCP ID 6

Cycle 0

Start Rating: 0

Cycle 55

End Rating: 2

# Appendix G - Freeze Thaw [Average Change in Mass]

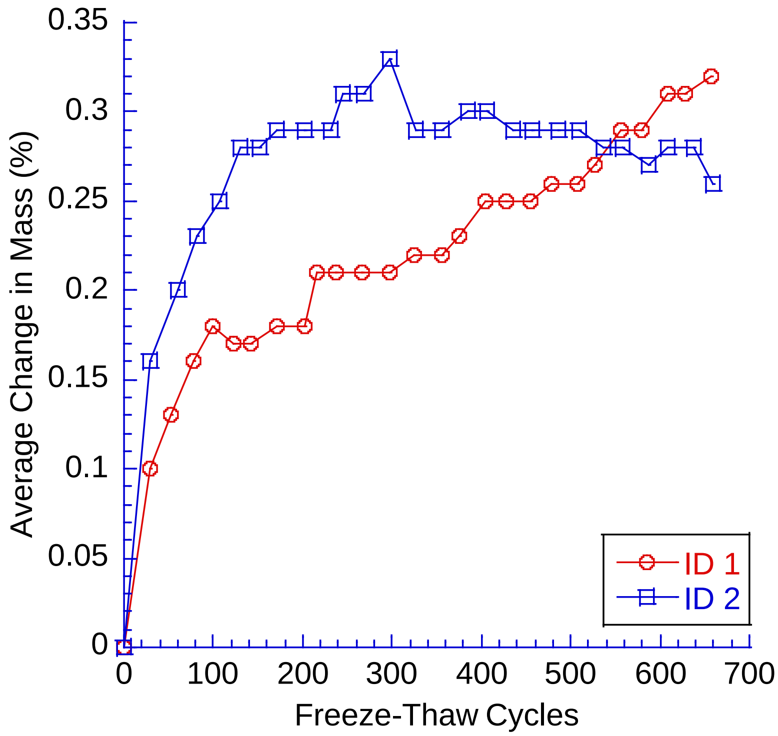



**Figure G-1: Average Change in Mass Based on 564 lb/yd³ Cement Factor**

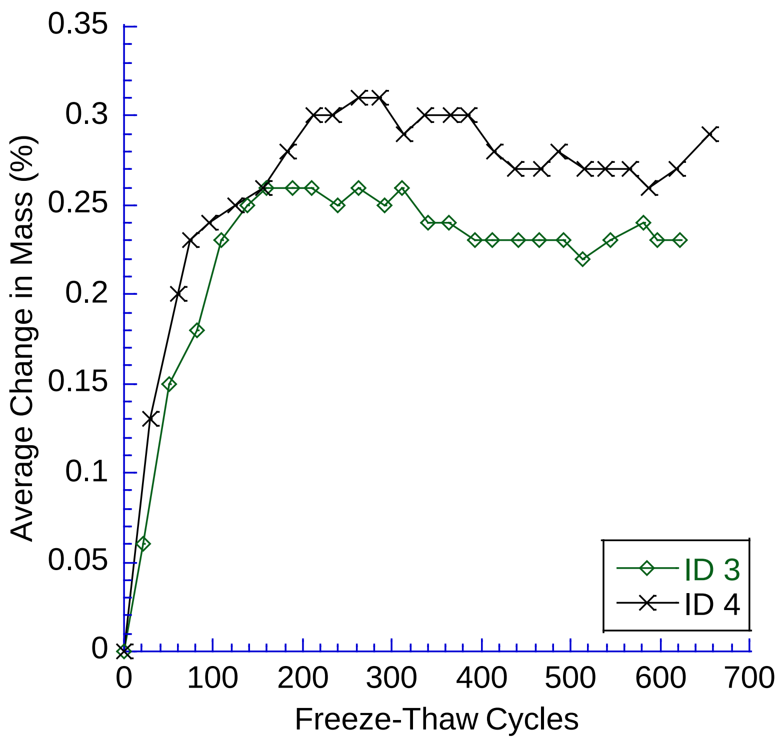



**Figure G-2: Average Change in Mass Based on 658 lb/yd³ Cement Factor**

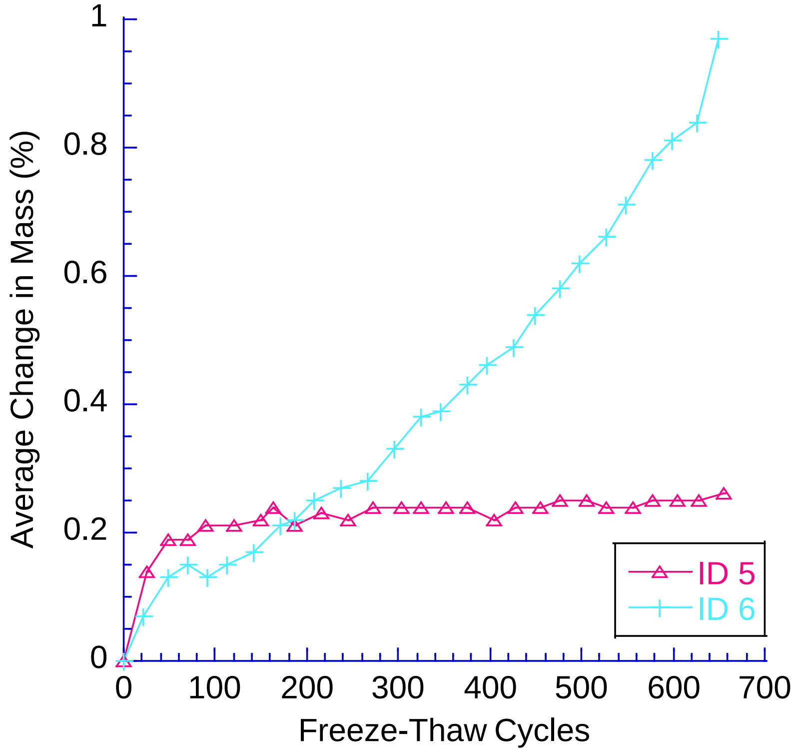



**Figure G-3:Average Change in Mass Based on 752 lb/yd$^3$ Cement Factor**

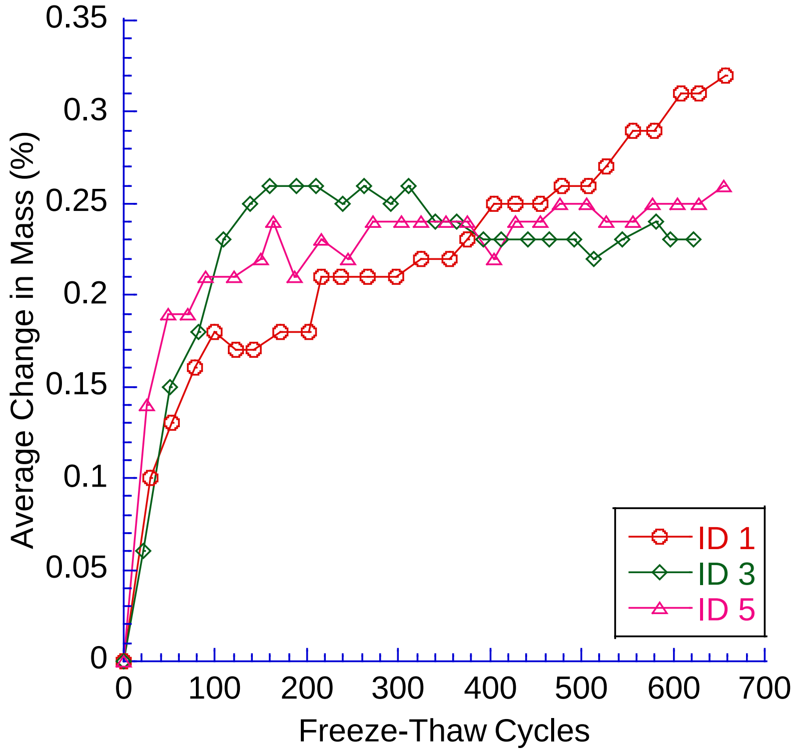



**Figure G-4:Average Change in Mass Based on 1% Calcium Chloride**

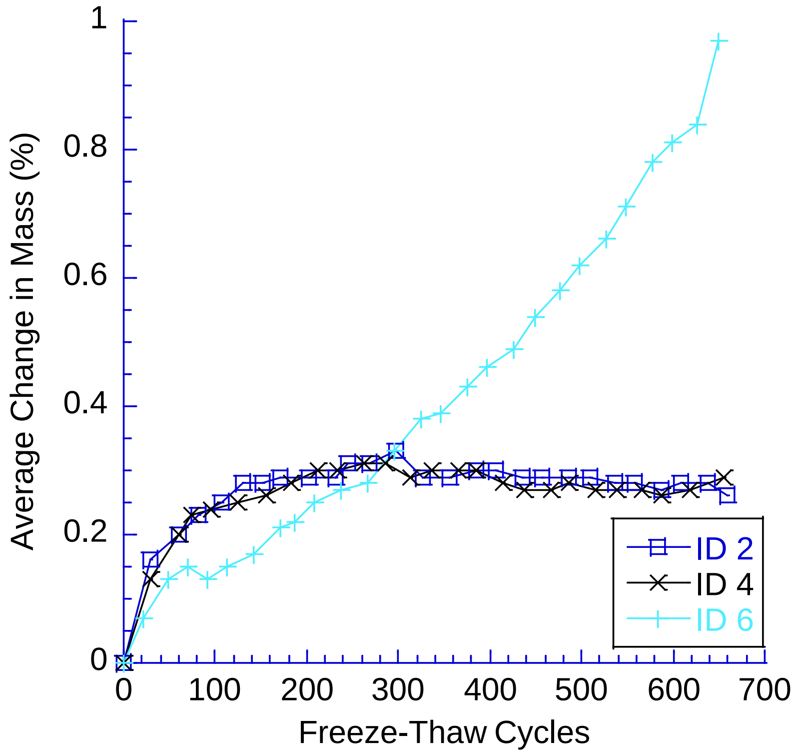



**Figure G-5: Average Change in Mass Based on 2% Calcium Chloride**

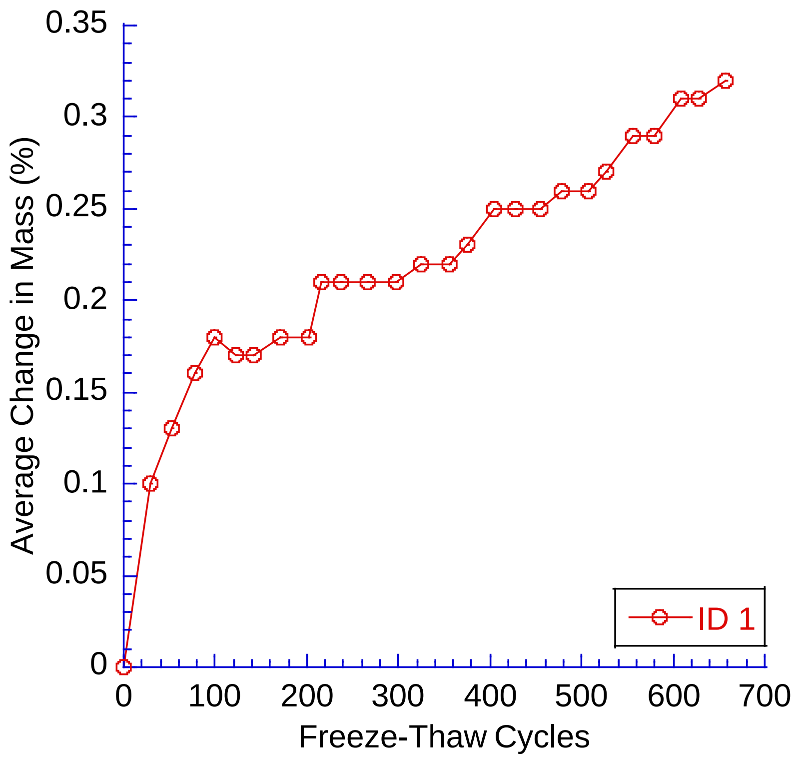



**Figure G-6:Average Change in Mass for PCCP ID 1**

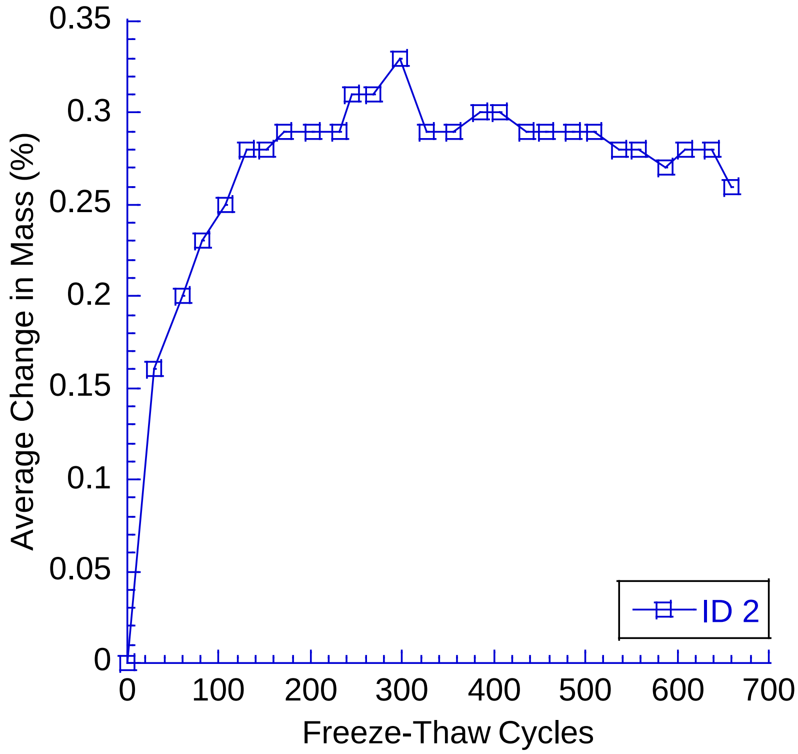



**Figure G-7: Average Change in Mass for PCCP ID 2**

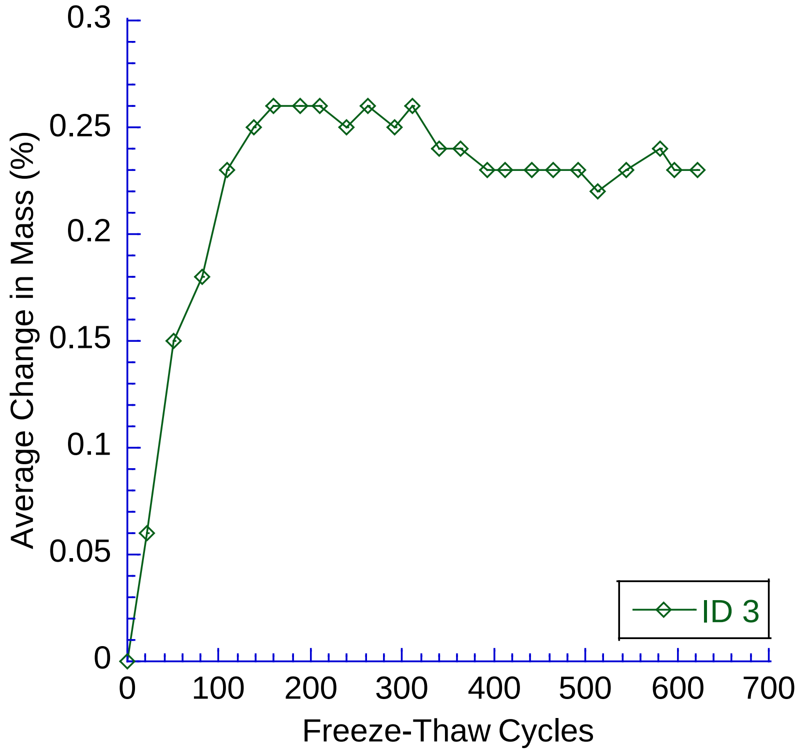



**Figure G-8: Average Change in Mass for PCCP ID 3**

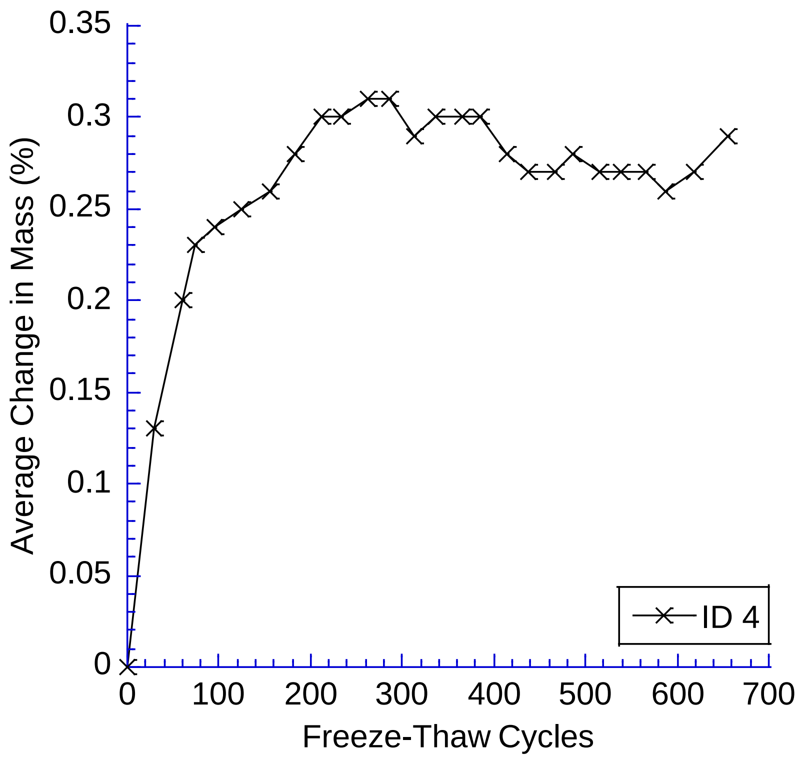



**Figure G-9: Average Change in Mass for PCCP ID 4**

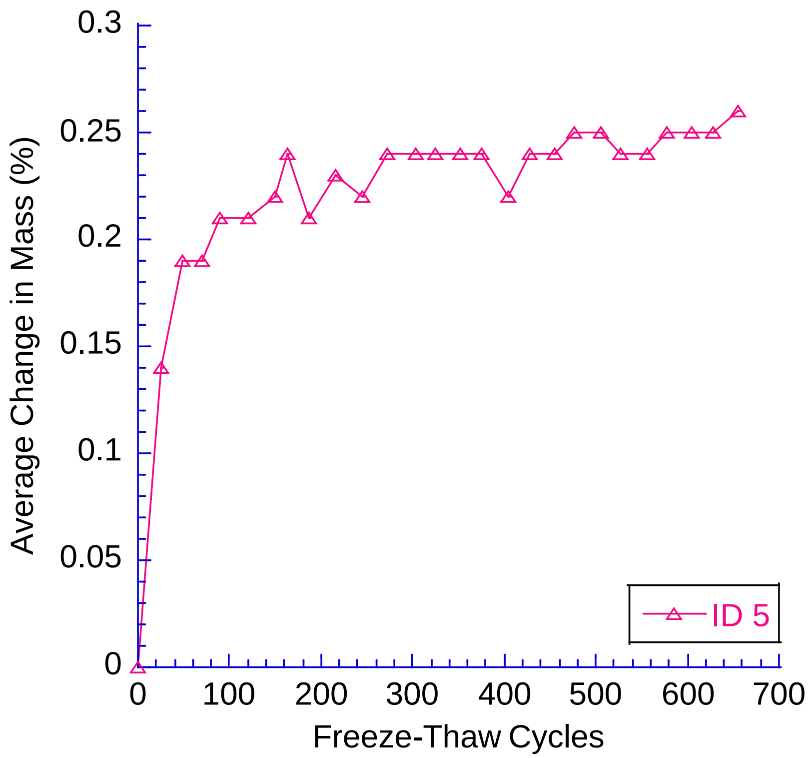



**Figure G-10: Average Change in Mass for PCCP ID 5**

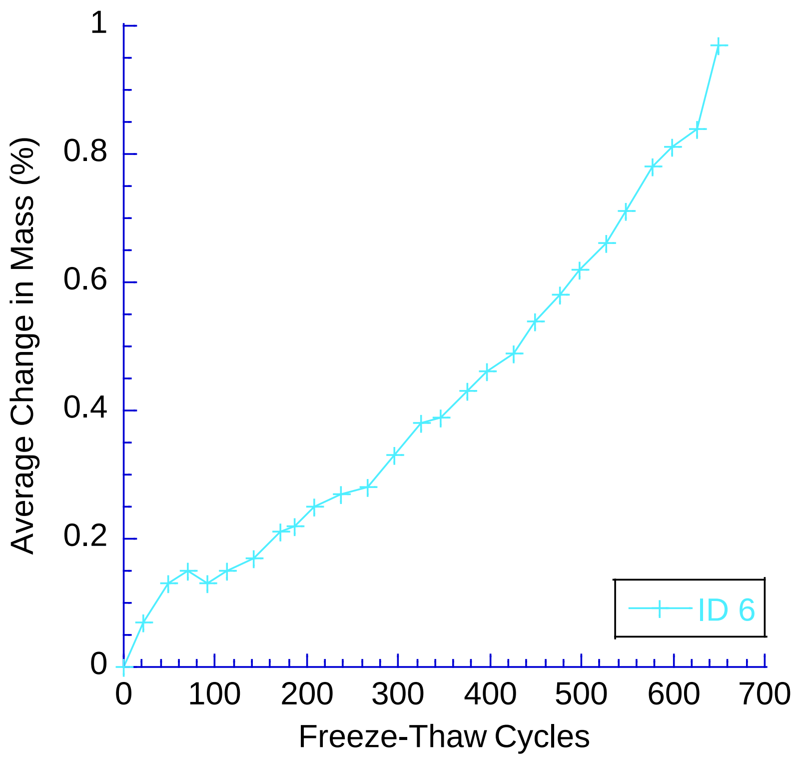



**Figure G-11: Average Change in Mass for PCCP ID 6**

# Appendix H - Freeze Thaw [Average Expansion]

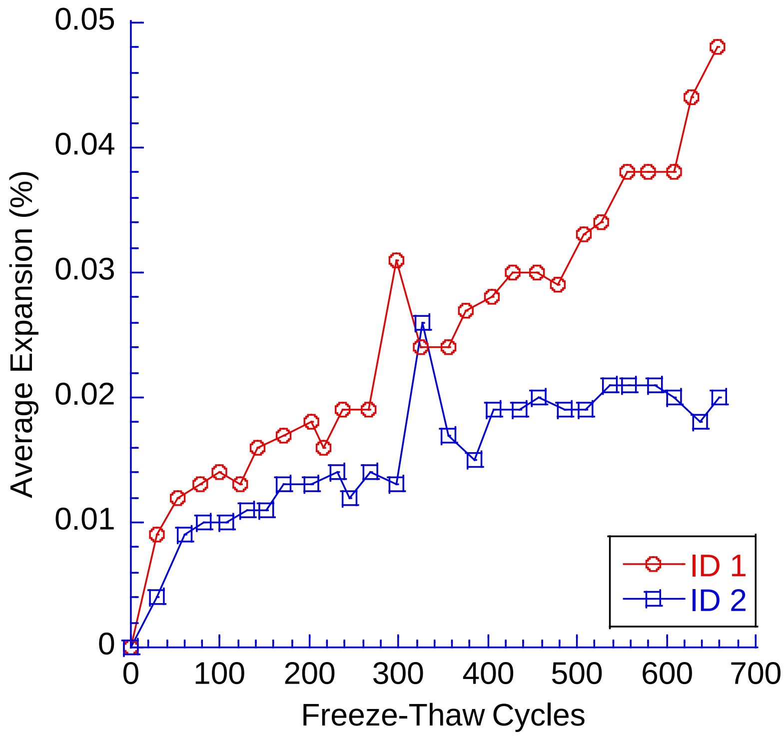



**Figure H-1: Average Expansion Based on 564 lb/yd$^3$ Cement Factor**

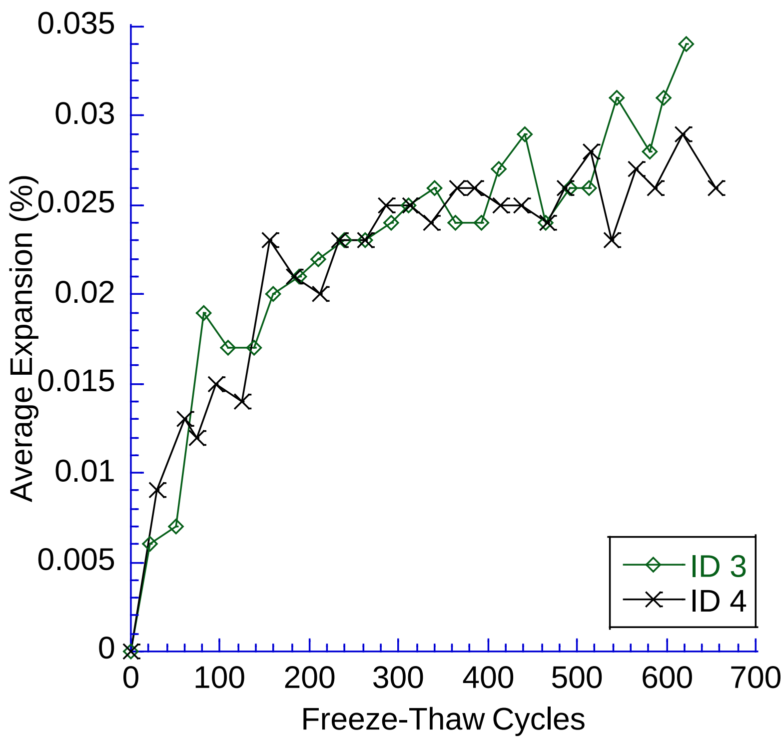



**Figure H-2: Average Expansion Based on 658 lb/yd$^3$ Cement Factor**

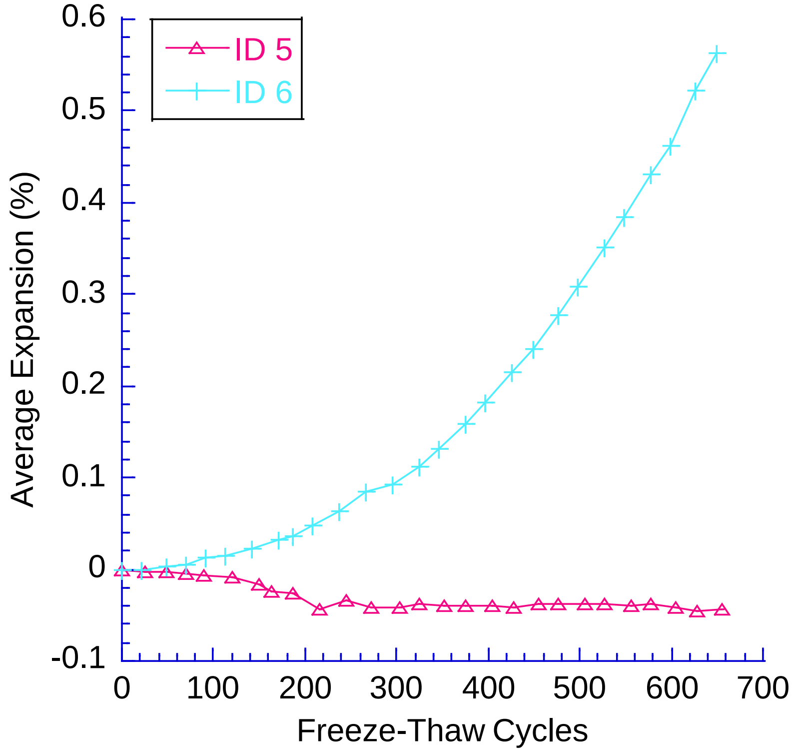



**Figure H-3: Average Expansion Based on 752 lb/yd$^3$ Cement Factor**

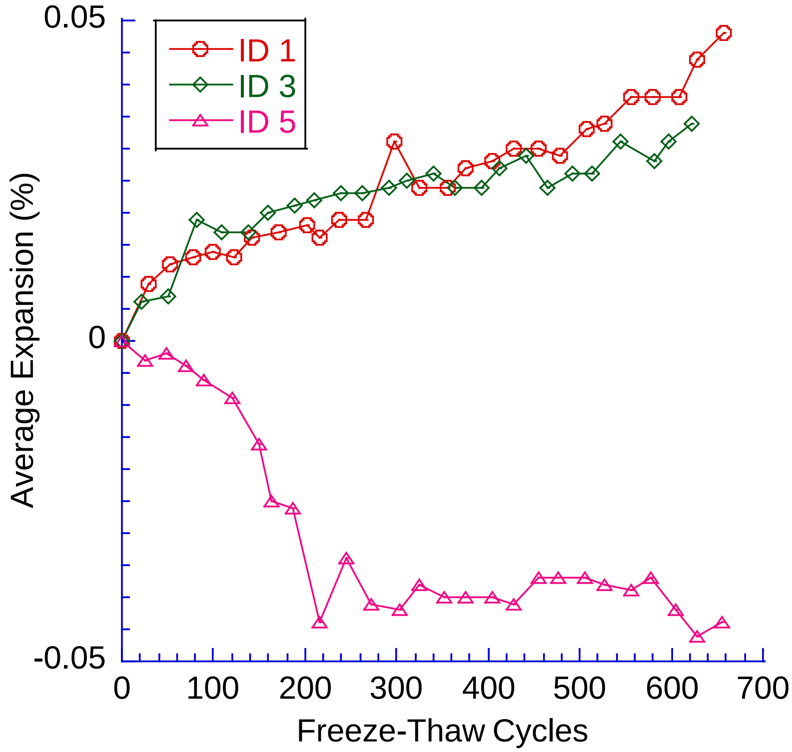



**Figure H-4: Average Expansion Based on 1% Calcium Chloride**

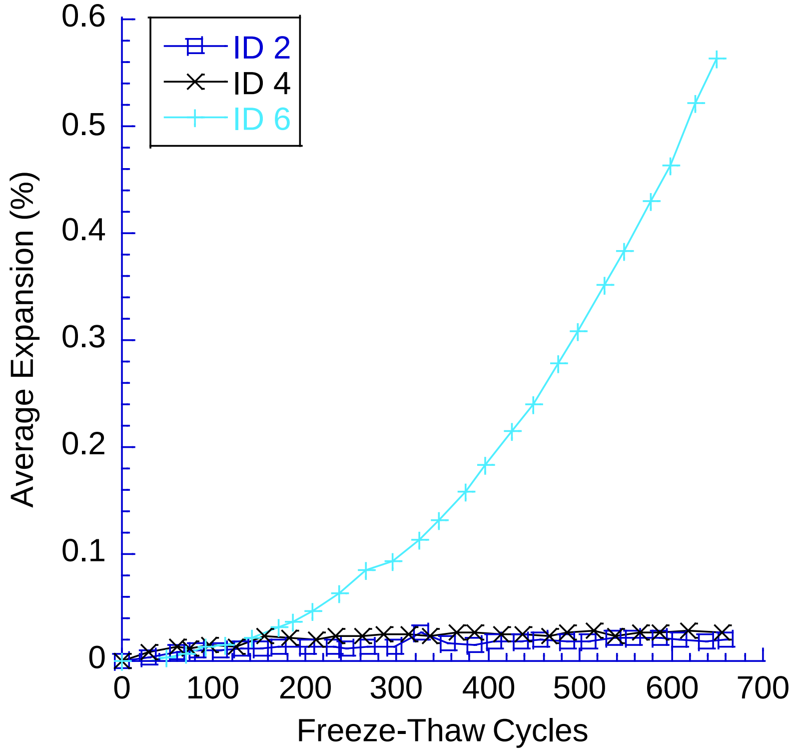



**Figure H-5: Average Expansion Based on 2% Calcium Chloride**

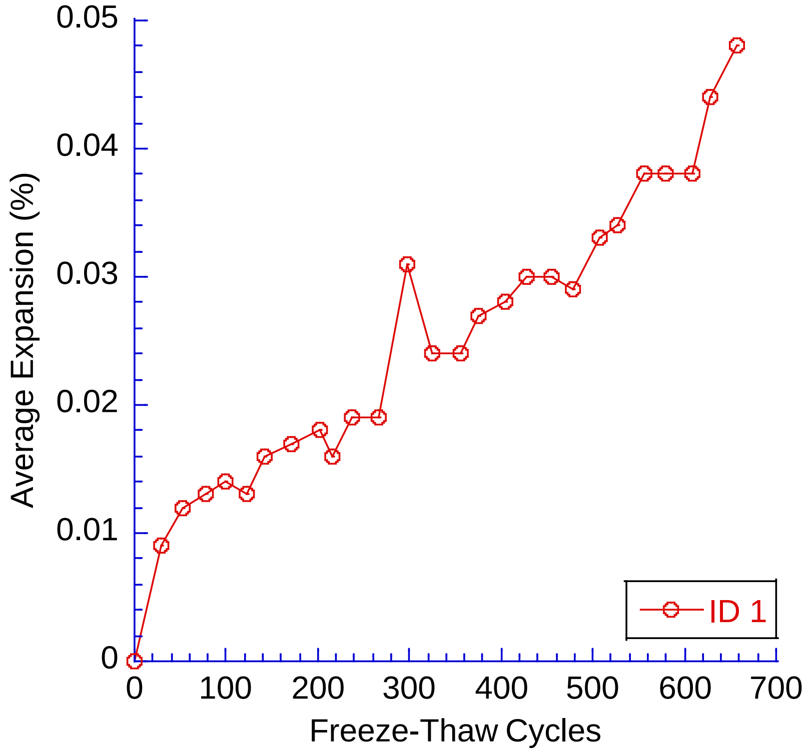



**Figure H-6: Average Expansion for PCCP ID 1**

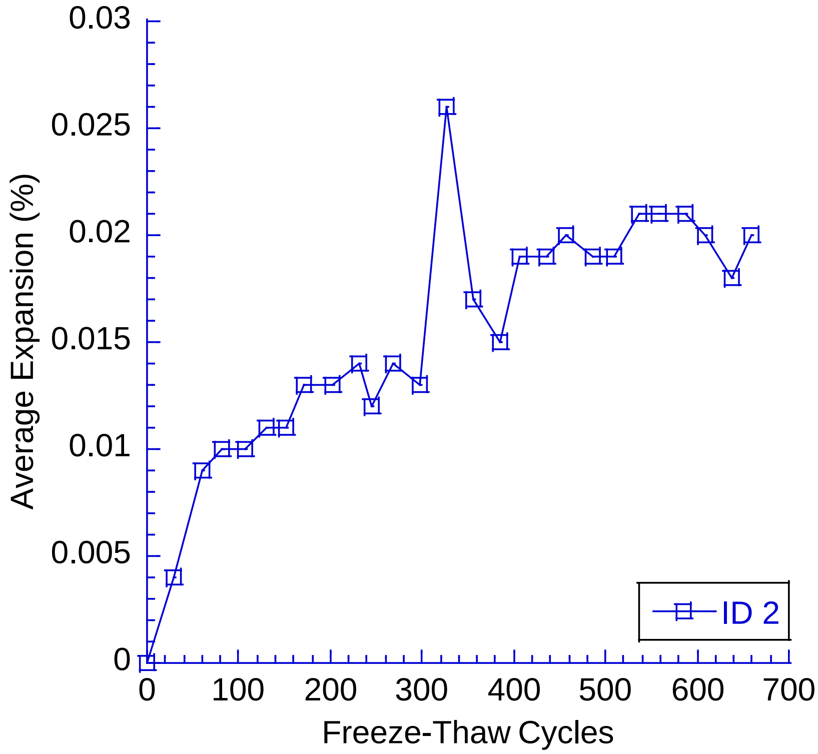



**Figure H-7: Average Expansion for PCCP ID 2**

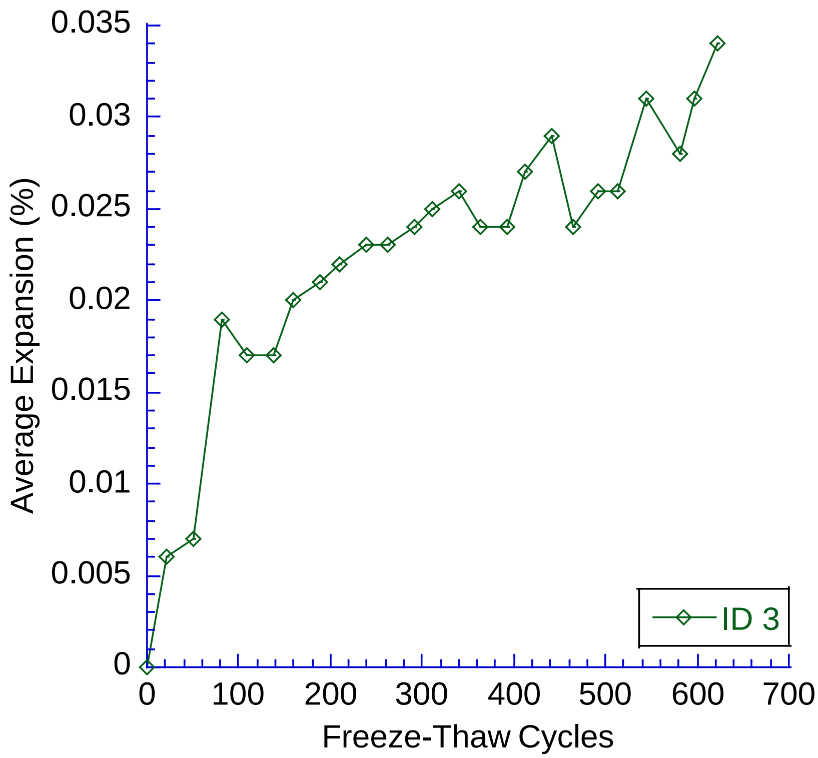



**Figure H-8: Average Expansion for PCCP ID 3**

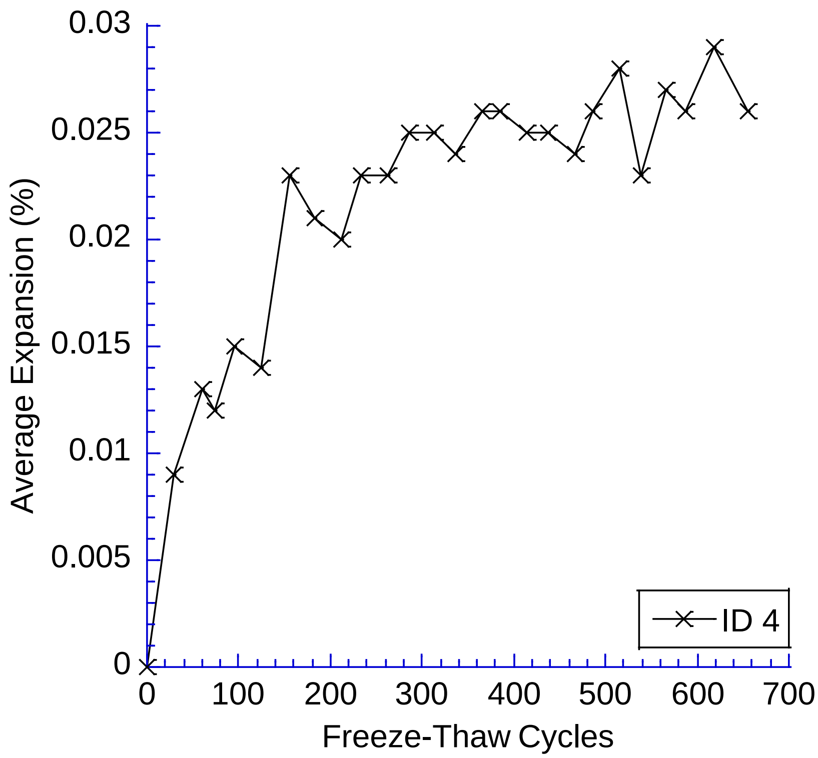



**Figure H-9: Average Expansion for PCCP ID 4**

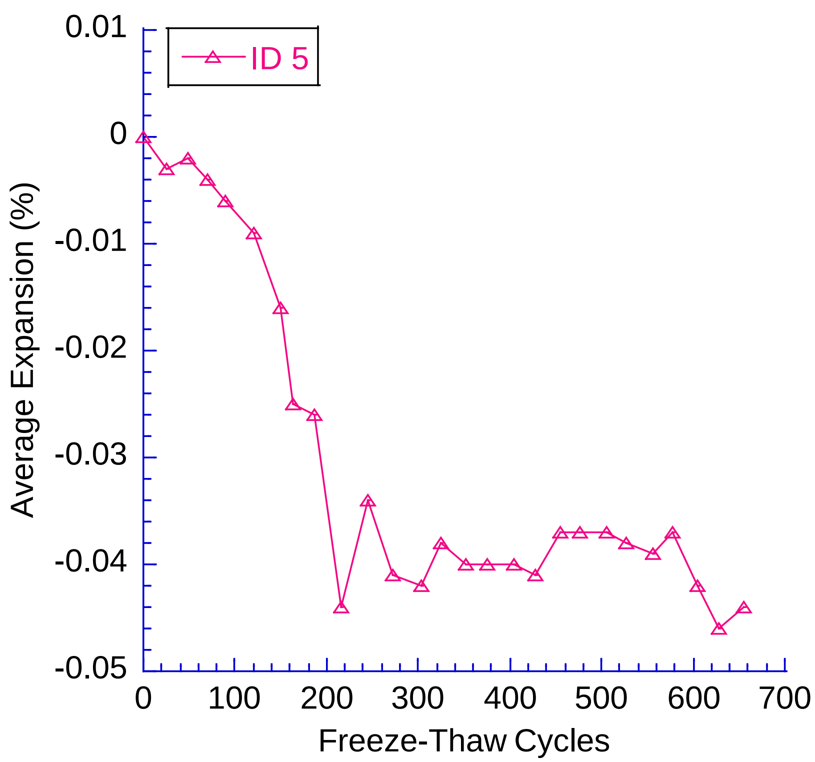



**Figure H-10: Average Expansion for PCCP ID 5**

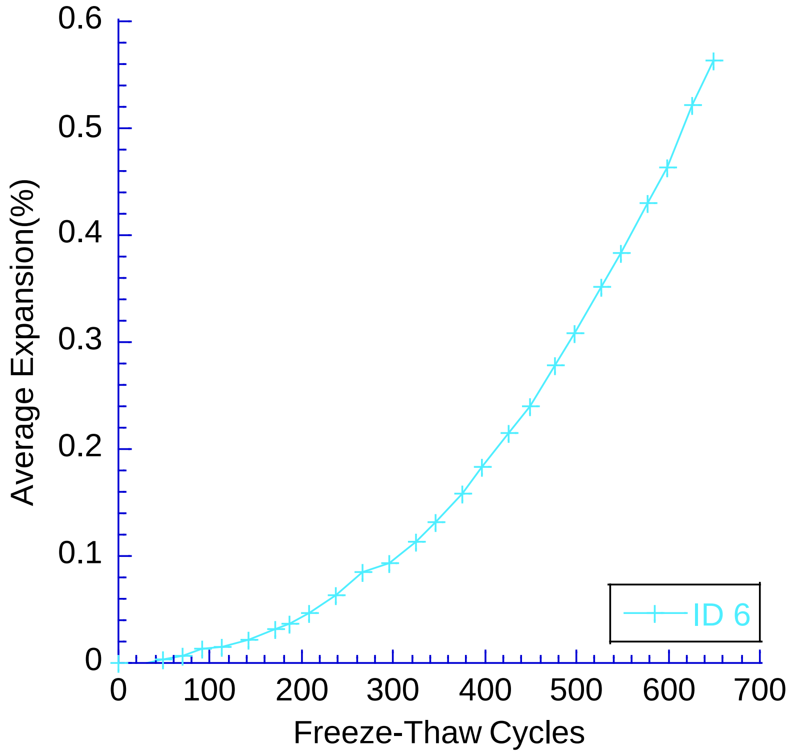



**Figure H-11: Average Expansion for PCCP ID 6**

# Appendix I - Freeze Thaw [Average RDME]

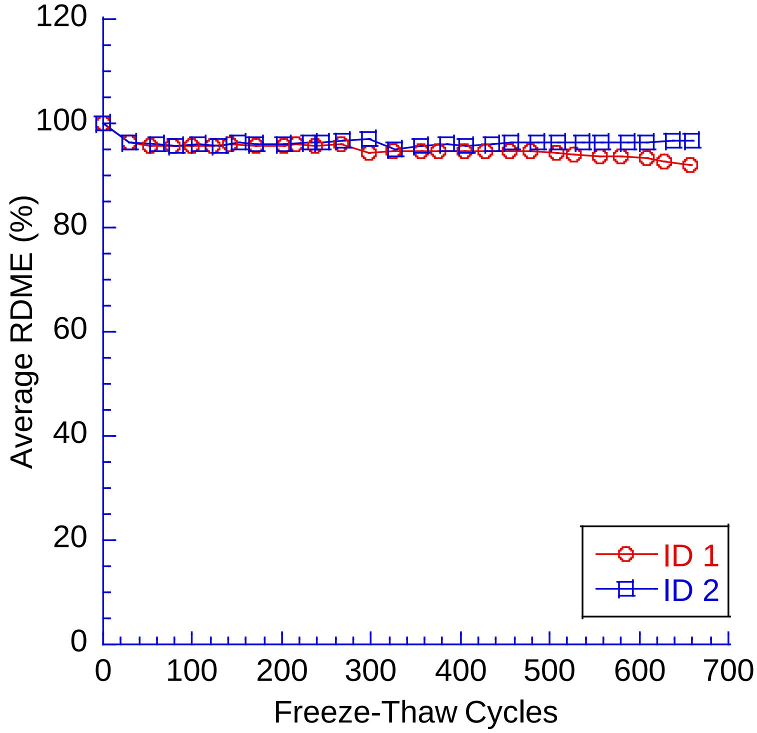



**Figure I-1: Average RDME Based on 564 lb/yd$^3$ Cement Factor**

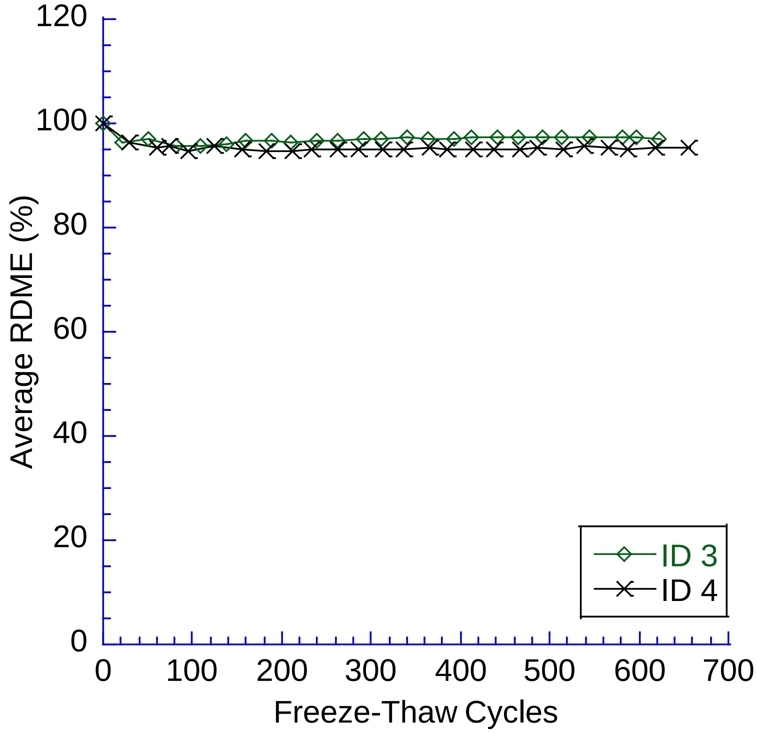



**Figure I-2: Average RDME Based on 658 lb/yd$^3$ Cement Factor**

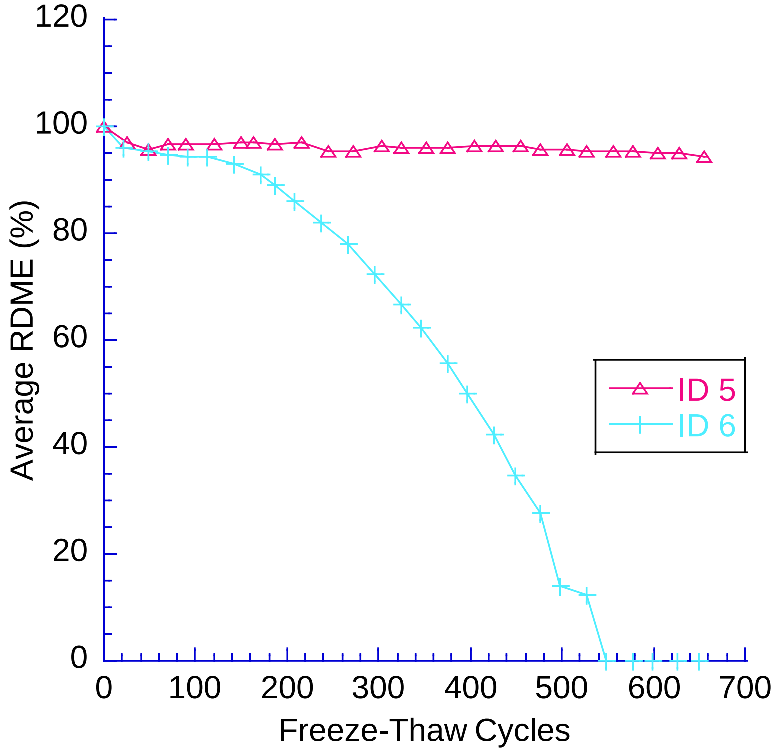



**Figure I-3: Average RDME Based on 752 lb/yd$^3$ Cement Factor**

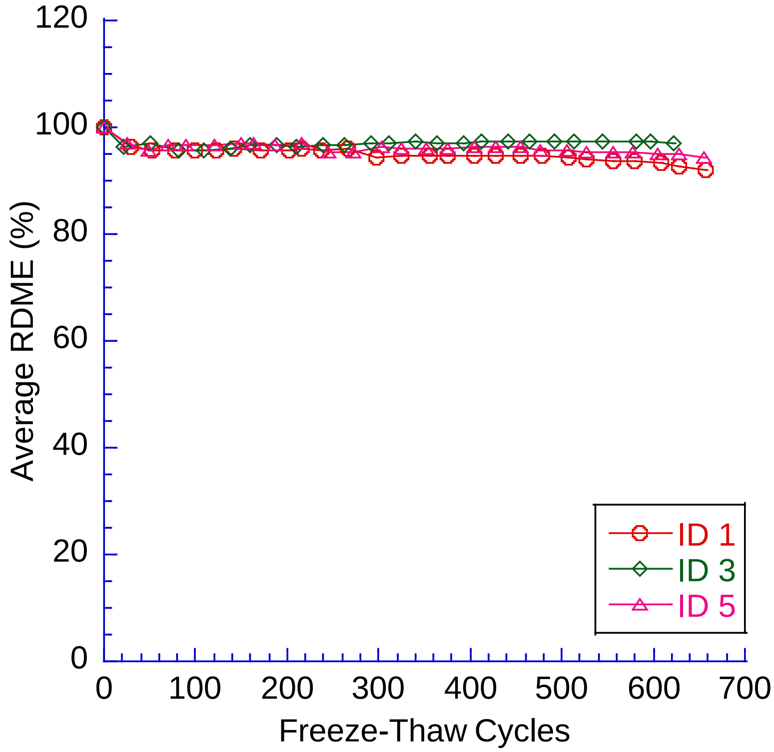



**Figure I-4: Average RDME Based on 1% Calcium Chloride**

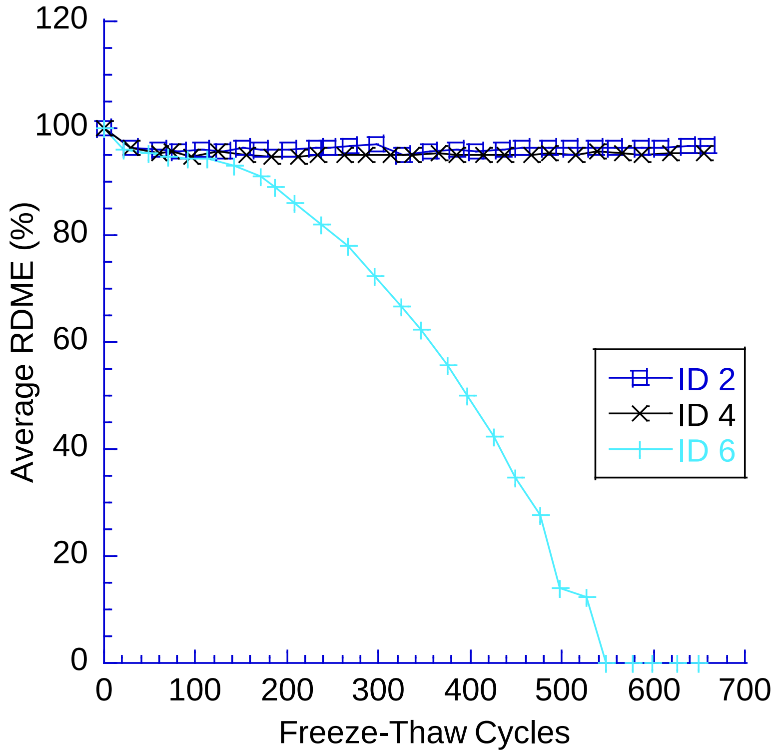



**Figure I-5: Average RDME Based on 2% Calcium Chloride**

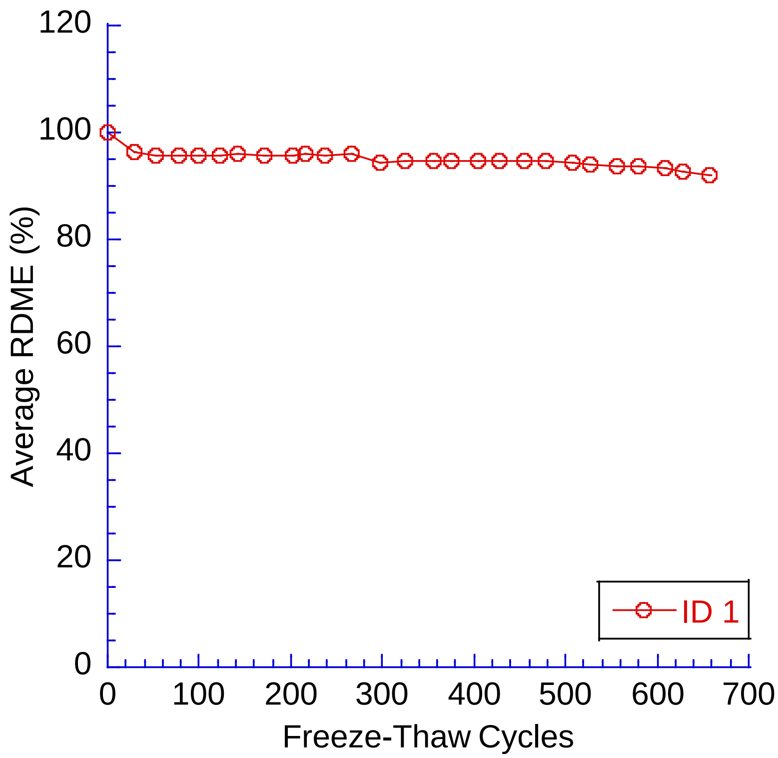



**Figure I-6: Average RDME for PCCP ID 1**

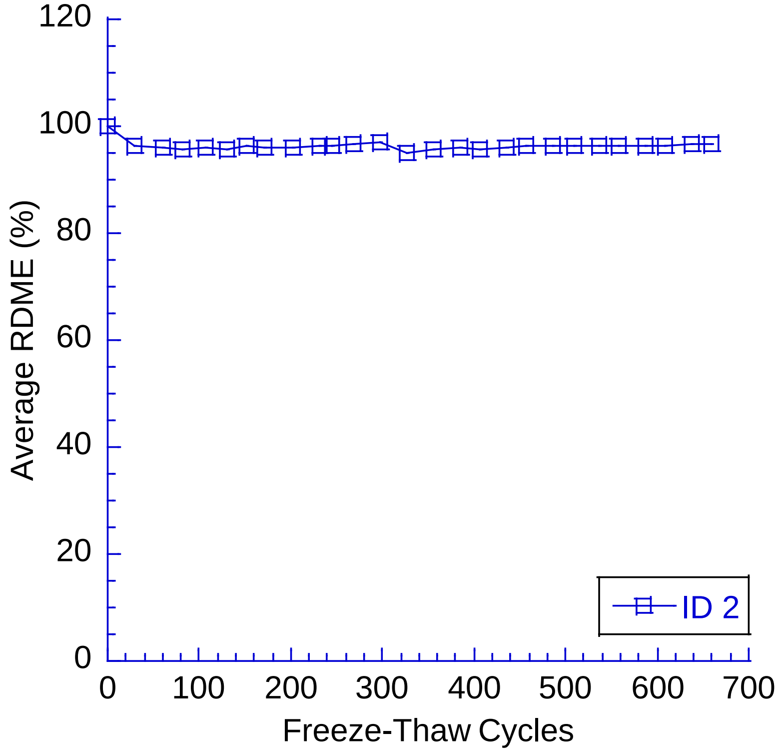



**Figure I-7: Average RDME for PCCP ID 2**

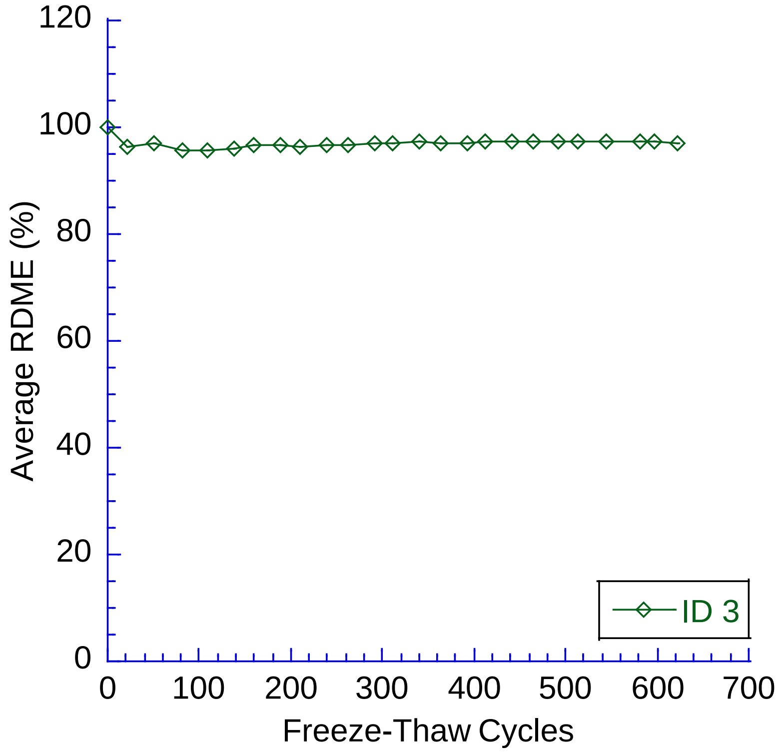



**Figure I-8: Average RDME for PCCP ID 3**

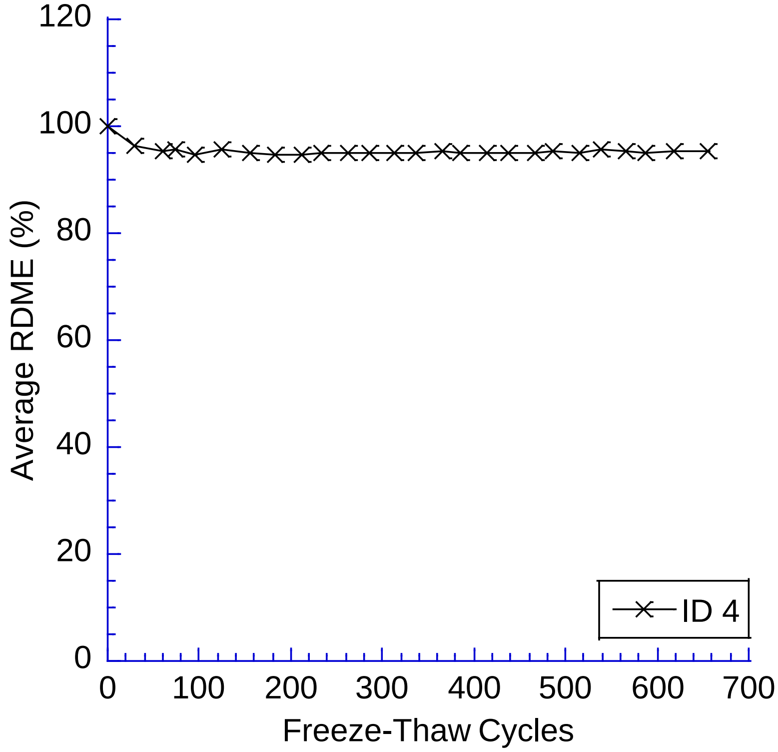



**Figure I-9: Average RDME for PCCP ID 4**

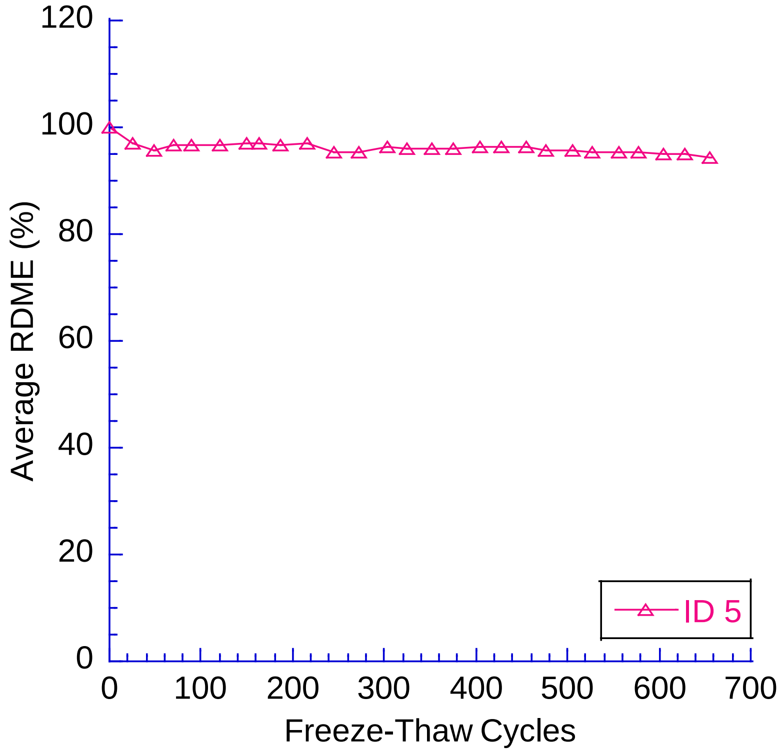



**Figure I-10: Average RDME for PCCP ID 5**

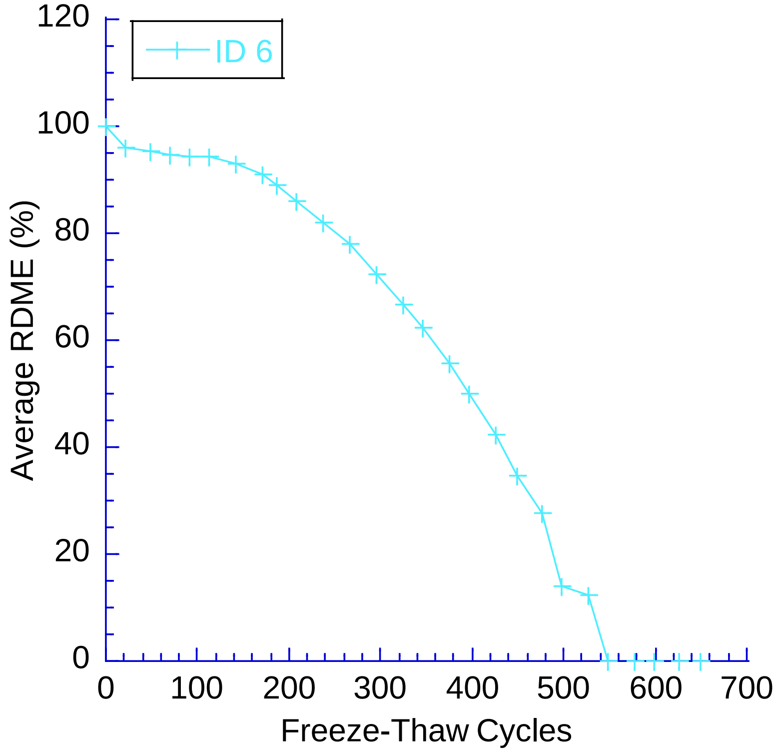



**Figure I-11: Average RDME for PCCP ID 6**