INTERNALLY CURED CONCRETE MIXTURES AND SPECIFICATIONS FOR HIGHWAY INFRASTRUCTURE

by

Jacob William Leach

A thesis submitted to the faculty of
The University of North Carolina at Charlotte
in partial fulfillment of the requirements
for the degree of Masters of Science in
Construction and Facilities Management

Charlotte

2017

Approved by:

____
Dr. Tara Cavalline

____
Dr. Brett Tempest

____
Dr. Thomas Nicholas

©2017
Jacob William Leach
ALL RIGHTS RESERVED

ABSTRACT

JACOB WILLIAM LEACH. Internally Cured Concrete Mixtures and Specifications for Highway Infrastructure. (Under the direction of DR. TARA L. CAVALLINE)

Internal curing is a relatively simple approach to reduce the shrinkage and cracking potential of concrete mixtures, accomplished by placing reservoirs of water within the concrete to provide additional moisture to facilitate hydration of the cement. These reservoirs are supplied by materials that can retain a significant amount of water, such as manufactured lightweight aggregates (LWA) added to the concrete in a prewetted condition. Benefits of internal curing, including reduced early-age cracking, are of particular interest in construction of concrete bridge decks and pavements, as agencies require longer-lasting, more durable highway infrastructure.

For this work, bridge deck and pavement concrete mixtures, produced using a variety of materials local to North Carolina, were batched and tested to provide insight into the benefits of internal curing. Two types of prewetted fine LWA were used to replace normalweight fine aggregate in different quantities to evaluate internal curing potential. Tests to evaluate the performance of fresh and hardened concrete were performed.

Compared to conventional concrete mixtures, internally cured concrete and mortar mixtures showed significant reductions in restrained drying and autogenous shrinkage (up to 56%). The mechanical properties and durability performance of internally cured mixtures was typically similar to that of conventional mixtures. Of note, internally cured mixtures had, on average, a 20% lower modulus of elasticity than control mixtures, which could further aid in reducing cracking potential. Additionally, the CTE was reduced in all

internally cured concrete mixtures (up to 11%) compared to control mixtures, indicating potential for improved thermal performance.

As this approach to concrete curing has not yet been used in concrete construction in North Carolina, specification provisions are presented that may be beneficial towards the creation of a Department of Transportation (DOT) specification developed to support successful implementation of internally cured concrete in future highway infrastructure projects.

ACKNOWLEDGEMENTS

I would like to acknowledge several individuals and organizations for their contributions to this research. I greatly appreciate all of the guidance and support provided to me by my advisor, Dr. Tara Cavalline, P.E. She has consistently provided me encouragement and support during my studies and has prepared me for success in my career and beyond. I would also like to thank the other members of my committee, Dr. Brett Tempest and Dr. Thomas Nicholas, for their guidance and support in my studies. This material is based upon work supported by the North Carolina Department of Transportation under Research Project 2016-06, and this support is greatly appreciated. I would also like to thank all of the companies that donated materials for the purpose of this research.

I would like to thank Gregory Loflin III for his continued effort working in conjunction with me on the batching and testing of concrete for this research project. I would also like to thank Wesley Maxwell, Blake Biggers, Patrick Phillips, Ross Newsome, Mark Fitzner and several others for their assistance in the laboratories as we batched and tested concrete mixtures.

I would like to thank my parents (Andrew and Jeri Leach) for all of the support, love and encouragement they have provided me along the way. I would also like to thank all of my friends and family who have kept me grounded and provided love and support as I performed this research.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
°C	degrees Celsius
CC	conventionally cured concrete
cf	cubic feet
CTE	coefficient of thermal expansion
cwt	hundred weight of cement
cy	cubic yard
dc	direct current
DOT	Department of Transportation
°F	degrees Fahrenheit
FHWA	Federal Highway Administration
ft	foot
g	gram
gal	gallon
hr	hour
ICC	internally cured concrete
in	inch
ITM	Indiana Testing Method
ITZ	interfacial transition zone
JPCP	jointed plain concrete pavement
kg	kilogram

lb	pound
LMC	latex-modified concrete
LRFD	load resistance factor design
LWA	lightweight aggregate
LWA1	lightweight fine aggregate 1
LWA2	lightweight fine aggregate 2
m	meter
μ	micro
mL	milliliter
mm	millimeter
MOE	modulus of elasticity
NCDOT	North Carolina Department of Transportation
NCHRP	National Cooperative Highway Research Program
OPC	ordinary portland cement
oz	ounce
PCC	portland cement concrete
pcy	pounds per cubic yard
pcf	pounds per cubic foot
psi	pounds per square inch
PSP	project special provisions
PVC	polyvinyl chloride
RCPT	rapid chloride permeability test
RH	relative humidity
rp	research project
rpm	revolutions per minute

S	siemens
ε	strain
SAM	super air meter
SCM	supplementary cementitious material
t_r	time to cracking
SSD	saturated surface dry
TRB	Transportation Research Board
V	volts
UNC	University of North Carolina
w/c	water to cementitious material ratio
yd	yard

CHAPTER 1: INTRODUCTION

1.1 Background and Significance

Like a number of state highway agencies, the NCDOT reports issues of early-age cracking of bridge deck concrete mixtures that include high cementitious contents and low water-to-cement ratios. A promising solution to mitigate these issues may exist in the form of internally cured concrete (ICC). Conventional concrete is cured in a more traditional method that essentially prevents moisture from leaving the surface, using either a physical or chemical barrier, or by replacing lost water with additional water. This is accomplished in the laboratory through the use of water storage tanks or a moist room and accomplished in the field through the use of wetted burlap, plastic sheeting, surface applied water, or other methods.

Adequate curing of concrete is of high importance to ensuring construction of durable concrete infrastructure, since improper curing can decrease the anticipated strength by 50% or more (NRMCA 2000). These forms of traditional curing, however, do not provide the necessary moisture to the interior of the concrete as moisture transport from the exterior to the interior of the concrete only has the potential to travel several millimeters. This is especially significant when discussing high cementitious content and low water-to-cement ratio concrete as there is typically not enough batch water available in the mixture to fully hydrate all the cement particles. When cement particles are not fully hydrated, a chemical shrinkage occurs within the concrete that, if restrained, may cause early-age cracking.

In contrast to conventional curing, internal curing (IC) is an approach that utilizes a material to deliver moisture internally to concrete, facilitating hydration throughout the concrete even after conventional curing methods are discontinued. ICC can be can be accomplished by placing reservoirs of water within the interior of the concrete that are able to provide moisture (in addition to that provided by the batch water) to the cement particles as the chemical reaction of hydration occurs. These internal reservoirs are supplied through materials that can retain a significant amount of water, such as manufactured lightweight aggregates (LWA) or absorbent polymer particles. These materials, when fully saturated, may provide enough moisture to fully hydrate the cement paste and potentially diminish or eliminate the shrinkage associated with the hydration process. If this shrinkage can be reduced, the potential for early-age cracking can be reduced as well.

Use of internally cured concrete (ICC) provides a potential means of mitigating the issue of early-age cracking of high-strength bridge deck concrete mixtures in North Carolina and in other states. Several states have successfully constructed IC concrete infrastructure using internal curing with prewetted LWA, since LWA is commercially available in many markets and has been shown to provide adequate characteristics to support internal curing (Bentur et al. 2001, Kovler et al. 2003, Geiker et al. 2004, Cusson and Hoogeveen 2008, and others). NCDOT desires to validate that the benefits of internal curing can be achieved for concrete mixtures typically utilized in North Carolina bridge decks and pavements, using locally available LWA. Additionally, NCDOT seeks guidance in preparing specifications for future ICC projects as well as identification of potential construction challenges associated with implementation of ICC in North Carolina projects.

1.2 Objectives and Scope

This research aims to provide verification that the benefits of ICC can be achieved using materials locally available to North Carolina through laboratory testing. A variety of concrete mixture designs typical of those utilized in North Carolina bridge decks and pavements were developed, batched, and tested in the laboratory to evaluate the benefits of internal curing using two different locally available LWA at two different replacement rates. A series of tests was performed to evaluate concrete characteristics such as mechanical properties, durability performance, and cracking potential.

The literature review and findings of laboratory testing performed as part of this project is used to establish a preliminary ICC specification – a project special provision (PSP) for a pilot project for ICC in North Carolina. This work supports development of recommendations for a general specification for NCDOT to be used in future ICC projects. Finally, a field testing and monitoring plan is established for the pilot project, including preliminary development of a low-cost field test to monitor in-situ concrete humidity for validation that IC is occurring.

1.3 Organization of Contents

Chapter 1 of this thesis includes a general introduction to the concept of internally cured concrete as well as the objectives of the research included in this thesis. Chapter 2 provides a literature review with a detailed background of IC materials, methods, and laboratory research findings, as well as a summary of the experiences of other researchers and state agencies that have implemented the use of ICC in field projects. Chapter 3 documents an in depth characterization of the materials that were utilized in this study. Chapter 4 describes the methodology and development of the mixture design and batching procedures of ICC. In Chapter 5 the details of the laboratory testing program are presented, along with the results and an evaluation of key findings. Chapter 6 includes a description of construction specification provisions developed for the pilot project, along with a field testing program and details of preliminary development of a low-cost field monitoring test for IC. Chapter 7 is the final summary of the project and includes lessons learned and recommendations for future work.

CHAPTER 2: LITERATURE REVIEW

2.1 Overview of Internal Curing

The object of curing is to keep concrete saturated until the originally water-filled space in the fresh cement paste has been filled to the desired extent by the products of hydration of cement and in the case of site concrete, active curing stops nearly always long before the maximum possible hydration has taken place (Neville 2011). In recent years, attention has been placed on providing extra water to the concrete to facilitate further hydration of the cement through a method known as internal curing. The American Concrete Institute (ACI) defines internal curing as “the use of absorptive materials in the mixture that supplement the standard curing practices by supplying moisture via internal reservoirs to the interior of the concrete” (ACI 2013). These internal reservoirs can be utilized throughout a variety of materials including prewetted lightweight aggregate (LWA), prewetted crushed returned concrete fines, superabsorbent polymers, or prewetted wood fibers and absorbent limestone aggregates (ACI 2013). For this study, prewetted lightweight manufactured aggregates were utilized to supply either additional hydration of the cement paste or water replacement for evaporation or self-desiccation, which leads to early-age cracking (Bentz and Weiss 2011). Therefore, for this literature review, focus is placed on use of LWA for internal curing of concrete.

Internally cured concrete has been inadvertently used over the last hundred years in lightweight concrete, however, until recently, this technology has not been intentionally incorporated into concrete mixtures at the proportioning stage (Bentz and Weiss 2011). One of the first to notice and quantify this phenomenon was renowned concrete technologist Robert Philleo. In his research, he states "…either the basic nature of Portland cement must be changed so that self-desiccation is reduced, or a way must be found to get curing water into the interior of high strength structural members." Philleo further states, "A partial replacement of fine aggregate with saturated lightweight fines might offer a promising solution" (Philleo 1991).

Over the last two decades, the benefits that could be gained through the relatively simple-to-implement technology of internally cured concrete has become more apparent. Several states, including Texas, Colorado, Utah, Illinois, Indiana, Ohio, Virginia and New York, have performed Division of Transportation (DOT) projects that utilize internally cured concrete. Research has shown that benefits of internally cured concrete exist (Bentz and Weiss 2011, ACI 2013, Rao and Darter 2013, Delatte et al. 2007, and others). However, many state highway agencies including NCDOT, require independent verification of these benefits using locally sourced materials and commonly utilized mixture designs, as well as cost and constructability assessments, before choosing to implement or otherwise specify internally cured concrete. This literature review provides a review of available information on internally cured concrete performance, specifications, and field experiences from other states.

It is important to clarify that the mixture design and material properties of concrete bridge decks is not the only factor that leads to cracking. As outlined in "Control of Cracking in Concrete: State of the Art," Transportation Research Circular E-C107 prepared by the Transportation Research Board (TRB 2006), factors that are generally accepted to be influential in bridge deck cracking include: bridge design, concrete mixture design, materials used in the concrete mixture, and placing, finishing and curing practices. A discussion on cracking of concrete bridge decks due to the design or construction of the bridge deck is not presented as part of this literature review. Focus is placed, rather, on the concrete mixture design and material properties of concrete, particularly characteristics which lead to increased (or decreased) cracking potential.

2.1.1 Mechanisms

While traditional curing is commonly specified in concrete construction, it is an additional step that is all too often overlooked or performed inadequately (Weiss et al. 2012). Traditional curing is performed through one or some combination of the following three methods. The first method is to maintain the presence of mixing water in the concrete during the early hardening period by use of ponding, spraying or saturated wet coverings. Another method is to reduce the loss of mixing water from the surface of the concrete by either covering it with impervious paper or plastic sheets or by use of membrane-forming curing compounds. The last method of traditional curing occurs during cold weather and is performed by supplying heat and additional moisture to the surface of the concrete by use of heating coils, steam, or electrically heated pads or blankets (University of Memphis 2007).

Adequate curing of the concrete is important to prevent two main negative consequences. First, without the availability of adequate moisture, the hydration reactions for cement slow and ultimately stop, limiting strength development and producing a more permeable final product. Secondly, the loss of water causes concrete to shrink and, if restrained, the concrete develops stresses that may lead to cracking (Weiss et al. 2012).

Cracking is dependent upon two factors: differential volume changes (shrinkage) and restraint to movement (ACI 2013). Adequate curing is essential to reducing permeability and preventing early-age cracking, ultimately extending service life and yielding low maintenance costs (Cusson et al. 2010).

A simple diagram explaining the comparison between external and internal curing is provided in Figure 2.1. It can be seen in the diagram that the effective cured zone is significantly higher in the internally cured concrete, and concrete throughout the body (not just near the surface) benefits from the presence of additional moisture. Also, it is noted that internally cured concrete is also cured conventionally (externally) and that internal curing occurs in addition to external curing, not as a substitution.

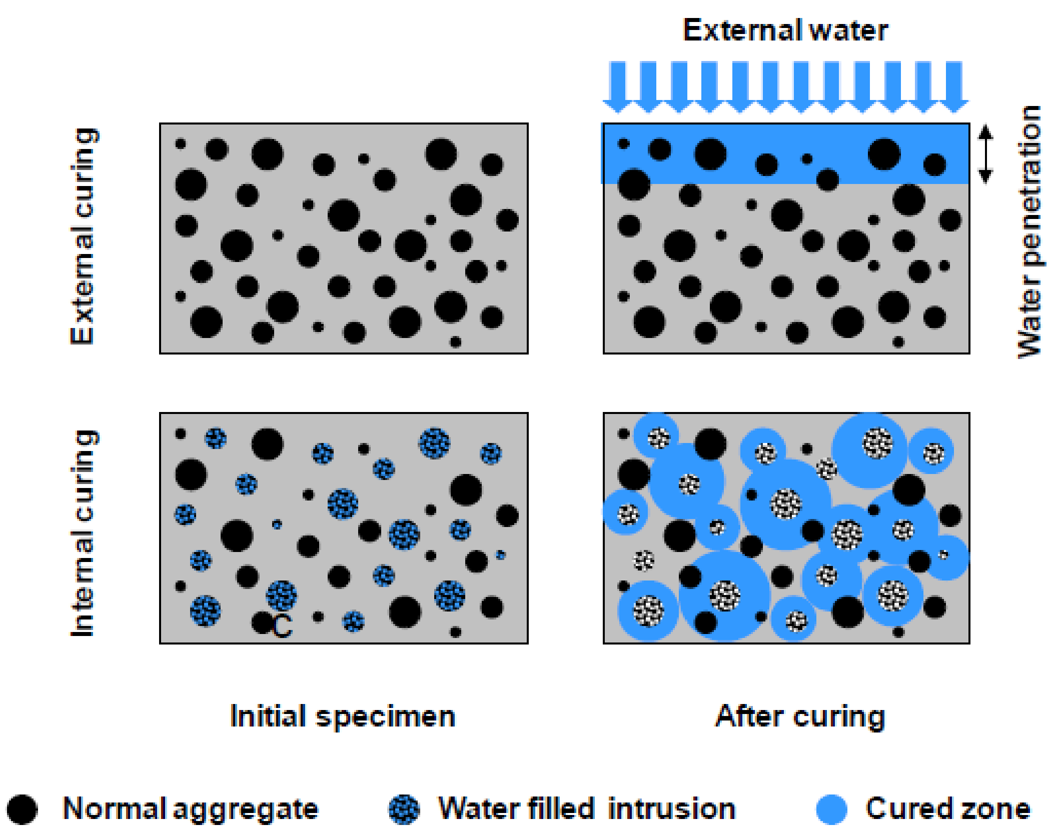


Figure 2.1: Comparison of external and internal curing (Castro et al. 2011)

Since the curing of concrete is directly related to its final properties, it is important to accurately predict how much water is required for curing. The amount of water necessary for curing is dependent upon many factors such as absorption capacities, desorption isotherms, evaporation, and saturation states (Bentz and Weiss 2011). Furthermore, Bentz et al. (2005) explain that concrete mixtures with a water-to-cementitious content ratio (w/c) of 0.36 or less have a higher likelihood of the mixing water not satisfying the curing requirements. Powers et al. (1959) also notes that in concrete with a w/cm of 0.42 or less, depercolation of the capillary pores by the products of hydration severely limits the amount of external water that can reach un-hydrated cement particles. Similarly, external surface curing has limited effectiveness after a few days beyond the cure affected zone which is typically the outer 0.2 to 0.3 in. (4 to 8 mm) at the surface of the concrete (Bentz and Weiss 2011).

Typically, high strength concrete mixtures utilize a low w/c (and often higher cement contents) to develop higher strength. These types of mixtures are becoming more popular in highway concrete projects due to their ability to rapidly gain strength and to potentially provide a reduction in structure weight due to requiring smaller supporting members. However, these high-performance concrete mixtures for bridge-deck applications have shown to develop shrinkage-related cracking (Wei and Hansen 2008). Therefore, many researchers and state highway agencies have thought the benefits of internal curing are ideal for these types of concrete mixtures.

According to Bentz (2007), over the past 50 years, portland cement has become finer and contains higher contents of tricalcium silicate and alkalis. These changes have led to generally faster hydrating cements that produce much of their strength in only a few days (ACI 2013). ACI 308-213 states that concretes that are made with these cements tend to be more prone to early-age cracking due to their increased heat of hydration and increased autogenous strains and stresses. To compensate for these negative effects, more moisture needs to be supplied internally to fully hydrate the cement paste and provide saturated conditions. According to Bentz and Weiss (2011), "…for each pound of tricalcium silicate (the primary component of cement) that reacts completely during hydration, we need to supply 0.07 lb. of extra curing water to maintain saturated conditions."

Saturated conditions effectively refer to a state of 100% humidity within the concrete. As the chemical process of hydration between water and cement begins, the products of hydration occupy less volume than the reacting materials (ACI 2013). This difference in volume produces a net chemical shrinkage that increases proportionally with the degree of hydration. After initial setting, the chemical shrinkage creates vapor-filled pores within the microstructure unless curing water is available to maintain saturation (ACI 2013). If curing water is not available, the internal humidity of the concrete will fall below 100% indicating that saturation conditions are not being maintained. This is known as self-dessication.

At this point, if internal curing reservoirs are available and well-dispersed, the water within the reservoirs will be desorbed into the cement paste via capillary suction, effectively restoring the internal humidity of the concrete to 100% indicating saturated conditions (Weber and Reinhardt 2003). The authors state that as new hydration products form, the capillary pores will be further reduced in size, further increasing the capillary suction and drawing more moisture from the LWA reservoirs (Weber and Reinhardt 2003).

Fortunately, the size of the pores within the LWA are typically much larger than the capillary pores within the paste microstructure, promoting hydration of numerous cement particles from the same water reservoir. This system is beneficial to the hydration of concrete, much like a well-dispersed system of structured entrained air voids is beneficial to protecting concrete in freezing-and-thawing conditions (Bentz and Snyder 1999). A more detailed analysis is performed on the characteristics of the aggregates before proportioning materials for internally cured concrete.

2.1.2 Materials

According to Bentz and Weiss (2011), the benefits of internal curing can be utilized throughout a variety of materials including prewetted LWAs, prewetted crushed returned concrete fines, superabsorbent polymers, and prewetted wood fibers. These materials are each able to provide an internal source of available moisture to replace that consumed by chemical shrinkage during hydration. This study will focus on the use of prewetted manufactured LWAs, and therefore this section of the literature review will be limited to these materials.

Most LWA is produced from materials such as clay, shale or slate. To produce LWA using these materials, the raw material is heated to high temperatures (in the range of 2000°F) until it expands to roughly twice the original volume (Holm and Ries 2007). One of the benefits of LWA when considering it for use in internally cured concrete are the generous sizes of the pores within the LWA created through the heating and expansion process. These pores, when saturated, are capable of storing water during the initial curing of concrete and slowly release it to supply either additional hydration of the cement paste or water replacement for evaporation or self-desiccation (Delatte et al. 2007).

In theory, internally curing concrete through the use of prewetted LWA seems relatively straightforward. However, in practice, the mechanical properties of LWAs can widely vary across different manufacturers. To determine the mixture proportions for internally cured concrete mixtures, information about the specific gravity, water absorption and water desorption properties of the LWA is needed (Castro et al. 2011). Not only do properties of LWAs vary between manufacturers, but the determination of the required desorption characteristics supporting release of moisture are not easy to obtain accurately. This is because LWAs have expansive networks of pore connectivity that yet again vary widely among different LWA manufacturers. Quantifying this porosity is critical for determining the water that can be absorbed (and later released) by the LWA (Castro et al. 2011). As mentioned earlier, this absorbed water does not influence the porosity of the paste during the setting of the concrete and therefore does not alter the water to cement ratio of the concrete.

Typically, when determining absorption properties of LWAs, absorption values are reported with a specific soaking duration length. This is because LWA can continue to absorb water for days and weeks due to the extensive pore connectivity within the material. Typically a 24, 48 or 72 hour absorption value is reported for LWAs. Therefore, the commonly used term SSD (Saturated Surface Dry) is not appropriate to use when describing LWAs because they have typically not reached full saturation within the 24, 48 or 72 hour time frame. Therefore, for internal curing purposes, the term prewetted LWA is used instead of SSD LWA. To illustrate the differences in LWAs by manufacturer, Holm et al. (2004) reported 24 hour absorption values for roughly a dozen different lightweight fine aggregates, ranging from 5 and 25% absorption by mass of dry aggregate. According to Castro et al. (2011), expanded slate LWAs typically have a 24 hour absorption between 6 and 12%, expanded shale LWAs typically have a 24 hour absorption between 10 and 20%, and expanded clay LWAs typically have a 24 hour absorption between 15 and 30%. The absorption capacity also varies with gradation of the aggregate as the larger the size of the aggregate the more porous space becomes inaccessible for water molecules (Henkensiefkin 2008). This significant difference in absorption percentages clearly justifies the detailed characterization of the LWA prior to mix proportioning for ICC.

In addition to characterizing the absorption properties of the LWA prior to use as an internal curing agent, it is of similar importance to verify the aggregate's ability to release the absorbed moisture to the surrounding hydrating cement paste. This process is referred to as desorption, and each LWA will possess a different desorption isotherm (Castro et al. 2011). Desorption of the moisture within the LWA can occur at high relative humidities, such as those present during the early ages of hardened concrete. The absorbed moisture is released as the internal humidity of the concrete drops below 100% to enhance and maximize the hydration of cement (ACI 2013). Unlike absorption rates, which differ widely among LWAs, desorption rates are virtually identical among LWAs with most desorbing at least 97% of their moisture at 94% or greater relative humidity (Henkensiefken et al. 2009). Castro et al. (2011) tested fifteen different LWAs to determine their ability to desorb absorbed moisture. Ideally, the LWA will release most of its absorbed moisture at a high relative humidity (93% or higher). The expanded shale, slate and clay aggregates tested all desorbed between 85% and 98% of their absorbed moisture at 93% relative humidity (Castro et al. 2011). It is noted by Castro et al. (2011) that each of the aggregates tested exhibited what can be considered 'good desorption behavior.'

As discussed, LWAs fall into three main categories, expanded slate LWAs, expanded shale LWAs and expanded clay LWAs. In North Carolina, expanded slate LWA and expanded shale LWA are commonly used in concrete construction. While the two materials have similar appearances, they are fundamentally different. Shale is a sedimentary rock while slate is a metamorphic rock formed from shale. Slate is considered much more durable than shale due to the metamorphic process it undergoes. Shale is the softer of the two and is more similar to clay (Holm and Ries 2007). As mentioned earlier, expanded shales tend to have a higher absorptive capacity than expanded slates. However, both materials have been demonstrated to provide sufficient performance as an internal curing agent in ICC (Rao and Darter 2013).

2.1.3 Methods

The methods used to develop internally cured concrete mixtures are not much different from those used to develop conventional concrete mixtures. In relatively early research in this area, Philleo (1991) recommended the replacement of normalweight fines with saturated lightweight fines. The use of fine LWA as the internal curing agent results in a final product that thoroughly distributes the internal curing reservoirs. The use of coarse LWA for internal curing purposes results in a microstructure similar to diagram (a) in Figure 2.2. Assuming that the moisture available within the aggregate can move approximately 2 mm into the surrounding paste, only a small portion of the paste is protected by internal curing. Using fine LWA as the internal curing agent on the other hand results in diagram (b) in Figure 3.1. The moisture reservoirs are now much more evenly distributed within the paste microstructure resulting in roughly 100% protection of the paste.

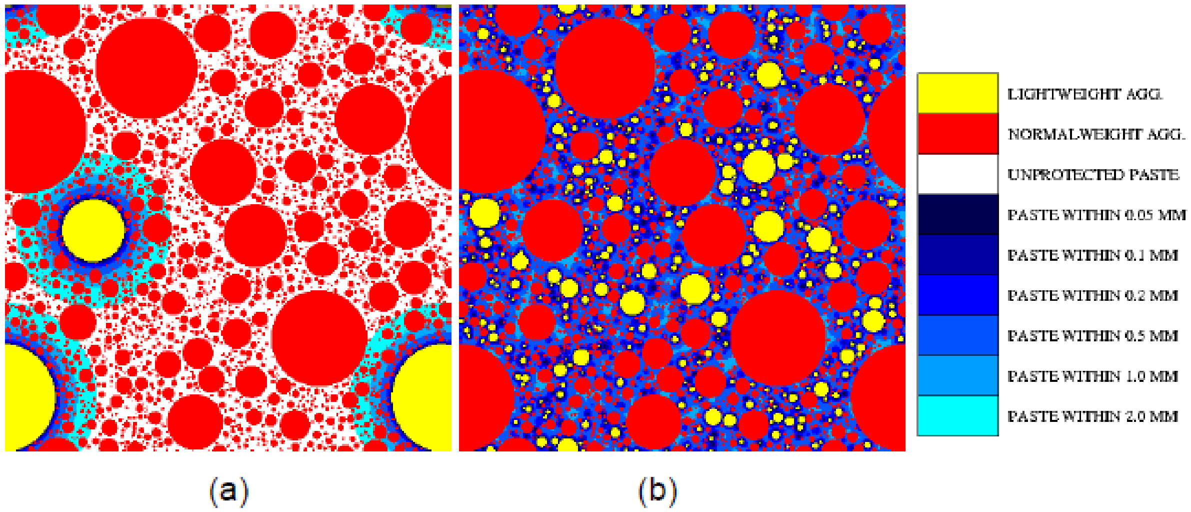


Figure 2.2: Spatial distribution of coarse LWA vs. fine LWA (Henkensiefken 2008)

This is the most commonly utilized method, since use of fine aggregate maximizes the dispersion of the internal curing agent, and consequently the volume of affected paste (Henkensiefkin 2008). Other state agencies utilize a replacement of both normalweight coarse and fine aggregate with saturated lightweight intermediate aggregate (Villareal and Crocker 2007). Traditional lightweight concrete construction has utilized replacement of normalweight coarse aggregate with saturated lightweight coarse aggregate. The coarse LWA is saturated in these instances so that it does not absorb mix water from the paste rather than for internal curing purposes.

Any quantity of additional water to facilitate internal curing will provide benefits to the surrounding cement paste. However, from a mixture proportioning standpoint, a procedure to determine the amount of LWA to provide adequate (or optimal) moisture to support internal curing is required. Bentz and Snyder (1999) developed a relationship between the water demand of the hydrating mixture and the supply that is readily available from the internal reservoirs of the LWA. The relationship is expressed below and will be referred to as Equation 2.1 throughout this report.

$$C_f{*}CS{*}\alpha_{max}=S{*}\Phi_{LWA}{*}M_{LWA} \qquad (2.1)$$

The left-hand side of the equation represents the water demand of the hydrating mixture and is composed of the cement (or binder) factor of the concrete mixture, C_f, the chemical shrinkage of the binder at 100% reaction, *CS*, and the expected maximum degree of reaction for the binder, α_{max}, ranging from 0 to 1. The right-hand side of the equation represents the water supplied from the internal reservoirs and is a product of the saturation level relative to a quantified ‘prewetted’ condition, *S*, the measured sorption capacity of the internal reservoirs in the prewetted condition, Φ_{LWA}, and the mass of saturated LWA required to meet the water demand, M_{LWA}. Nomographs have been developed to aid in internally cured concrete mixture designs incorporating LWA (Bentz 2009). Example nomographs in both SI and English units can be found in Appendix A in Figures A1 and A2.

Equation 2.1 provides only an initial estimate of the LWA required to supply absorbed moisture, as it does not account for a variety of effects that may result in the occurrence of autogenous shrinkage in concrete formulated with prewetted absorptive materials (ACI 2013). ACI 308-213 further explains that some of these effects include an insufficient spatial distribution of the LWA within the concrete microstructure or other strains such as thermal strains or drying shrinkage. For general estimating purposes, it is common to use 0.07 pounds of water per pound of cement for the chemical shrinkage coefficient, *CS* (Bentz and Snyder 1999, ACI 2013). The expected maximum degree of reaction of the binder, α_{max}, can be assumed to be 1 if the w/c is equal to or greater than

0.36. Otherwise, α_{max} is given by [(w/c)/0.36] for w/c<0.36. If the water to cement ratio and cement content are known, the water demand of the hydrating mixture (left side of Equation 1) can easily be determined. Similarly, after a thorough LWA characterization, the available water supply from the internal reservoirs can be chosen based on the necessary demand.

2.2 Laboratory Studies on Internal Curing for Highway Concrete

Over the past several decades, a number of studies have been performed to develop and validate the performance of internally cured concrete mixtures for highway concrete. These studies utilize a variety of materials, mixture proportions, and tests methods, and a few of the key findings of laboratory studies supporting this work are summarized in this section.

2.2.1 Lightweight Aggregate (LWA) Characteristics

Due to the inherent difference in geographical compositions of aggregates, the natural properties of aggregates differ across regions. The process of manufacturing LWA also varies by production facility, creating an even wider range of LWA properties. Similar to characterizing conventional aggregate characteristics for mixture proportioning, the unit weight, specific gravity and absorption of the LWA should be determined to aid in mixture proportioning. Additional attention needs to be given to the characteristics of absorption and desorption, for the reasons discussed in Section 2.1.3 above.

The use of prewetted LWA as an internal curing agent typically requires that the aggregate be in, or as close to, the saturated surface-dry (SSD) condition prior to mixing. This condition can be achieved in the laboratory by either the paper towel method (ASTM C1761) or by the centrifuge method (Miller, Barrett et al. 2014, Indiana Testing Method

(ITM) 222). SSD condition is achieved in the field, far less consistently, through the sprinkling and draining of aggregate stockpiles (NYSDOT 2009). Therefore, as discussed previously in Section 2.1.2, in this thesis, this condition will generally be referred to as the prewetted condition rather than the SSD condition.

It is important to determine the desorption qualities of LWA as this is how the water is transferred from the LWA to the hydrating paste. This is a function of the size of the pores and capillaries of the aggregate. If the pores and capillaries are too small, the release of water is too slow compared to what is required to hydrate the cement paste to the fullest (ACI 2013). For LWA to function successfully as an internal curing reservoir, the pores containing the water must be larger than those in the surrounding cement paste, so that water will preferentially move from the LWA to the hydrating cement (Bentz and Weiss 2011). Therefore, it is important to assess the speed and quantity of water being released from the aggregate. This is done by analyzing the aggregates desorption characteristics compared to industry standards according to ASTM C1761. Most LWAs possess similar desorption isotherms, 97% or greater (Henkensiefken et al. 2009) but some LWAs do not desorb this magnitude of water at 94% RH and therefore laboratory testing is still recommended.

2.2.2 Mixture Proportioning

The effectiveness of water from the LWA to hydrate the cement depends primarily on the following factors: the amount of water absorbed in the LWA, the LWA spatial distribution, the LWA pore structure and the strength and shape of the LWA (ACI 2013). Mixture proportioning with ICC is quite similar to conventional concrete with the primary difference being the determination of the quantity of prewetted LWA to be substituted for normalweight fines. It is noted by Lopez et al. (2006, 2008) that the possibility exists of overdosing prewetted LWA. As stated earlier, the addition of prewetted LWA in excess of what is needed for internal curing can reduce strength, especially at early ages when effects of the absorptive material are not fully developed as well as when supplementary cementitious materials are used (Bentz and Weiss 2011). Additionally, the amount of lightweight replacement has an effect on concrete's modulus of elasticity. Replacing small amounts of natural sand with lightweight fine aggregate can decrease the modulus of elasticity of the concrete (Hoff 2003), although, a lower modulus of elasticity has been shown to reduce cracking potential in some situations (Neville 2011, ACI 2013).

If too much prewetted LWA is used, the concrete mixture has more curing water than necessary. Additional water remaining in the pores of the LWA could cause concern in low temperatures, with freeze-thaw damage potentially occurring at early ages (Schlitter et al. 2010). An abundance of LWA within the mixture design may increase the permeability of the hardened concrete (Bentz and Weiss 2011). On the contrary, if an insufficient amount of prewetted LWA is used, especially in low water-to-cement ratio concretes, the effects of internal curing will be diminished. If this is the case, the cement paste will not be able to fully hydrate leaving concerns for autogenous shrinkage and self-dessication of the concrete (Bentz and Weiss 2011).

Following the LWA characterization and choice of cement content and water-to-cement ratio, the Bentz and Snyder approach can be utilized to determine an exact mass of saturated LWA needed to supply the water demand (ACI 2013). Nomographs provided in Appendix A of this report show how the recommended amount of LWA varies based on different chemical shrinkage coefficients, cement contents, water-to-cement ratios and absorption percentages of LWA (Bentz 2009). Once a mass of LWA is selected, it replaces the same volume of normalweight fine aggregate within the concrete mixture design. The selected mass of LWA will result in a specific replacement rate. This replacement rate can be adjusted for mixture simplification, as many state agencies tend to do (Bentz and Weiss 2011). For example, the New York State Division of Transportation (NYSDOT) specifies a 30% (by volume) substitution of normalweight fine aggregate with prewetted LWA as well as a minimum LWA absorption of 15% for all internal curing concrete, regardless of LWA selected. Following the replacement ratio determination, mixture proportioning of internally cured concrete can be performed in manner that is similar to mixture proportioning of conventional concrete (ACI 2013).

Wei and Hansen (2008) performed laboratory tests on concrete mixtures in Michigan containing 0.35 and 0.45 w/c ratios with 20% and 40% replacement of prewetted LWA for normalweight sand. The researchers found that the mixture with a w/c of 0.35 and 20% replacement did not completely eliminate autogenous shrinkage. However, the mixtures with 0.35 w/c and 40% replacement and 0.45 w/c and 20% replacement sufficiently prevented autogenous shrinkage. Mortar mixtures tested in Illinois were prepared using various amounts of LWA replacement including 0, 13, 25, and 38% substitutions (Ardeshirilajimi et al. 2016). The researchers found that as the replacement ratio increased autogenous shrinkage decreased. A 13% replacement slightly increased the compressive strength compared to 0% while 25% and 38% slightly decreased the compressive strength. Significant amounts of laboratory testing have been performed in Indiana regarding internally cured concrete for highway purposes. Through absorption and specific gravity testing of the LWA on the day of batching, INDOT determines an exact amount of prewetted LWA to use in their laboratory concrete mixtures (Barrett et al. 2015). Through this method, researchers can determine the ideal amount of LWA substitution to balance the effects of autogenous shrinkage and compressive strength reduction.

The significant differences in prewetted LWA replacement ratios used across the country in ICC indicate that the benefits of internal curing can be reached using a variety of materials and proportions. However, several key considerations need to be addressed in mixture proportioning when determining the quantity of prewetted LWA for inclusion. One consideration is that high replacement rates of LWA (particularly low unit weight LWA) could reduce the unit weight of the concrete below that of normalweight concrete, typically 135 – 155 pounds per cubic foot (AASHTO 2012), having implications on structural design. High replacement rates of LWA also have the potential of reducing both the compressive and splitting tensile strength of the concrete mixture (Bentz and Weiss 2011), and tests to confirm adequate mechanical properties should be performed. Finally, the amount of prewetted LWA included in the mixture needs to provide enough moisture within the reservoirs to adequately hydrate the remaining cement particles. When deficient, this can be compensated for by soaking the LWA for a longer period of time than typically recommended 48 – 72 hours (Ardeshirlajimi et al. 2016). In fact, Ardeshirilajimi et al. state that a 7 day absorption period of the LWA was shown to increase the early-age expansion of concrete mixtures and resulted in a lower net shrinkage at 28 days.

2.2.3 Internal Curing for Bridge Deck Concrete Mixtures

The NYSDOT has utilized internally cured concrete in many bridge decks, with varying superstructure types, span lengths, and other structural characteristics. Like bridges in many states, those in New York undergo a variety of severe conditions and environments caused by deicing chemicals, coastal conditions, freezing and thawing, wetting and drying, and heating and cooling (Streeter et al. 2012). As of 2015, internal curing concrete using prewetted LWA has been incorporated into dozens of bridge decks within New York State (Streeter 2017). Due to “bundling” of multiple bridges into single contracts, it is difficult to get an exact number. According to Streeter (2017), NYSDOT has also used ICC in other applications such as precast deck panels, sidewalks, bridge barriers and header repairs all under the premise of reducing cracking potential.

In 2013, INDOT constructed four bridges that utilized internally cured high performance concrete. Associated research studies provide details on LWA testing and characterization, mixture proportioning, laboratory testing and field implementation including sensor equipment embedded in the concrete. Researchers concluded that internal curing of concrete using prewetted LWA reduced early-age autogenous shrinkage by 80% compared to non-internally cured mixtures and estimate that the service life of an ICC bridge deck can triple that of a conventionally cured bridge deck (Barrett et al. 2015).

Four bridges were also constructed in northern Utah, two using conventional concrete and two containing prewetted LWA. Data from embedded sensors was collected as well. They observed a higher moisture content in the internally cured bridge decks, but an almost identical electrical conductivity – suggesting two types of concrete have similar diffusivity (Guthrie and Yaede 2013). At 28 days, the two types of concretes had similar compressive strengths while the internally cured concretes passed between 2 and 30 percent less current compared to conventional concrete using the RCPT test. After several months, cracking was observed at approximately 0.2 to 0.3 mm in the conventional decks while no visible cracks were found in the internally cured bridge decks.

Colorado also has experience utilizing internal curing concrete for bridge decks in the laboratory. The modified Colorado DOT bridge deck mixtures to be internally cured and focused on comparing fresh, mechanical, freeze-thaw and shrinkage properties. Their results, when comparing ICC to conventional concrete, showed a slightly reduced unit weight (2 – 6%), a slight increase in compressive strength, reduced MOE (10 – 20%), and enhanced durability performance in free-thaw testing (Jones et al. 2014). Furthermore, the authors state that ICC minimized autogenous shrinkage, showed improved drying shrinkage performance and reduced the residual stress build up in restrained shrinkage testing. As part of their research, a cost analysis was performed that estimated the initial cost of an ICC bridge deck to be 3 – 10 $\$/yd^3$ more than conventional concrete but consider this price increase marginal especially when considering the benefits of increased service life and reduced maintenance costs (Jones et al. 2014).

2.2.4 Internal Curing for Pavement Mixtures

Concrete mixtures typically utilized for rigid pavements typically have a lower cement content than bridge deck concrete mixtures (NCDOT 2012). Additionally, pavement mixtures tend to have lower slump and lower compressive and flexural strengths (NCDOT 2012). Internal curing technologies have been targeted for potential improvements to the performance of bridge deck concrete as well as pavement concrete in a number of states.

Rao and Darter (2013) have published findings on internally cured concrete for pavement use, and have identified two key benefits: structural longevity and durability. The authors state that structural longevity is improved in ICC due to a small reduction in unit weight, elastic modulus and CTE as well as a small increase in strength (both tensile and compressive). These effects when combined lead to significant positive impact on slab fatigue damage and slab cracking in jointed concrete pavements (Rao and Darter 2013). Durability performance benefits that the authors noticed include reductions in permeability as well as early age shrinkage, plastic shrinkage cracking and long term drying shrinkage. It has also been found that the interfacial transition zone (ITZ) between aggregate and cement paste was improved in ICC pavements that were subjected to freeze-thaw conditions (Peled et al. 2010).

Friggle and Reeves (2008) documented their work with internally cured pavements in Texas. Traditional paving methods were used including a slip-form machine, pavement train, carpet drag and tining, and the researchers do not detail any changes from that of conventional concrete paving (Friggle and Reeves 2008). Many benefits were noticed during placement and testing. It is believed, based upon observation, that the LWA provided bleed water throughout the entire finishing and curing process, which significantly improved finishing (Friggle and Reeves 2008). Testing of hardened concrete pavement properties in the field showed internal curing benefits in the pavement including 10% greater shrinkage in the control section at 28 days than the internally cured pavement. A dramatic difference was noticed when crack surveys were performed at approximately 3 months and 1 year. The internally cured pavement had less total cracks, reduced widths of cracks, and increased spacing of cracks. The authors concluded that the use of prewetted LWA in concrete pavement could provide benefits including a reduction of paste content and a reduction of drying shrinkage cracking.

Also in Texas, Villarreal (2008) documented the use of internally cured concrete pavement in construction of an intermodal transfer facility for the Union Pacific Railroad.

The rail yard is considered one of the most widely recognized internal curing paving projects using approximately 250,000 cubic yards of concrete. Testing of the concrete showed that the project was quite successful in benefiting from internal curing. Villarreal states, “The slow release of moisture from the LWA to the concrete matrix has resulted in the mitigation or elimination of plastic and drying shrinkage cracking, as well as limiting the effects of self-desiccation.” The TxDOT also noticed an improvement in compressive strength during testing of specimens of ICC versus conventional concrete. Furthermore, after 8 years of service, the internally cured pavement showed almost no cracking and no measureable curl in a random sampling of 20 slabs on the project (Villareal and Crocker 2007).

2.2.5 Internal Curing for Latex-Modified Concretes

Latex modified concrete (LMC) is primarily used as a bonded overlay that when hardened can protect bridge decks and other concrete structures. As LMC cures, the latex polymers form internal plastic films that can reduce permeability, reduce modulus of elasticity and increase durability (Choi and Yun 2014). A number of other state highway agencies, including NCDOT, have reported early-age cracking of LMC bridge deck overlays similar to high performance concrete that is not latex modified. Choi and Yun (2014) observed LMC mixtures exhibiting significant early age shrinkage indicating the need for substantial curing. Based upon a review of available literature, research investigating the potential of internal curing to improve cracking resistance in latex-modified concrete has not been performed.

2.3 Field Implementation of Internal Curing

In order to successfully implement internal curing in a field setting, proper preparation of the materials required for internal curing is essential, particularly the prewetted LWA. As mentioned earlier, this is typically done through the use of stockpiles and sprinkler systems on site or at the batch plant. However, this process is performed and specified in various ways depending on the state agency. For example, NYSDOT utilizes stockpiles at the batch plant and recommends a “continuous and uniform sprinkle for a minimum of 48 hours.” Afterwards, “stockpiles are allowed to drain for 12 to 15 hours immediately prior to use” (Streeter et al. 2012).

Prior to batching, Bentz and Weiss (2011) state that quality control tests must be performed on the aggregate stockpiles prior to mixing to ensure a moisture state as similar to SSD as possible. However, this is considered almost impossible without using methods such as the paper towel test or centrifuge, as the force of gravity will not overcome the force of some of the water to ‘stick’ to the surface of the aggregates within the stockpile. This is known as surface moisture and is common among aggregates stored in stockpiles at batch plants. Figure 2.2 below is a schematic that shows the presence of surface moisture on a LWA. LWA at SSD condition containing surface moisture is pictured on the left. If the surface moisture, pictured on the right, is removed, what remains is LWA in SSD condition, pictured in the middle.

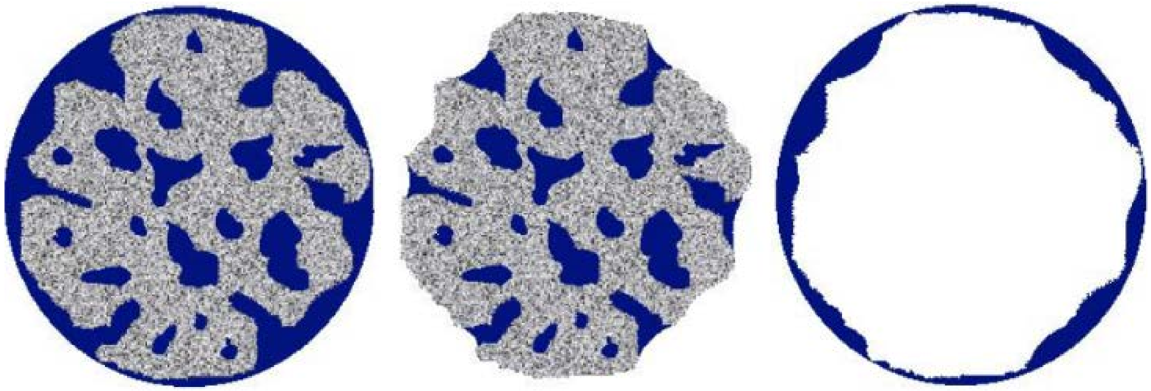


Figure 2.3: Schematic explaining surface moisture of aggregate (Barrett et al. 2015)

Since the presence of surface moisture within the prewetted LWA stockpile is inevitable, it must be accounted for in the batching stage. Typical values of surface moisture will range from 3 to 20 % by weight of prewetted LWA (Barrett. et al. 2015). The LWA used in the internally cured bridge deck mixtures placed in Indiana possessed absorption ranges of 18.7 to 20.2% with surface moisture ranging from 6.6 to 9.9%. Surface moisture is accounted for in the batch plant either through the use of ASTM testing procedures, calculations, and manual adjustments to the computers or through sensors placed within the stockpile that monitor the total moisture state and subtract surface moisture from the available mixing water. This procedure, of closely monitoring the moisture condition of the prewetted LWA, is the most critical step to the successful implementation of ICC (Bentz and Weiss 2011, ACI 2013, Barrett et al. 2015, and others).

When it comes to placing and finishing of internal curing concrete, many state agencies report no differences (Villareal and Crocker 2007, Streeter et al. 2012). Some noted a slight increase in bleed water which made placement easier (Villareal and Crocker 2007, Bentz and Weiss 2011). Researchers and state agencies stress that traditional curing methods (such as curing compounds, wet burlap, and plastic sheeting) should be utilized in addition to internal curing measures (Bentz and Weiss 2011, Streeter et al. 2012, Rao and Darter 2013, ACI 2013).

2.3.1 Specification Provisions

Based on the literature reviewed, it becomes clear that successful field implementation of ICC has been accomplished through development and implementation of specification provisions and other guidance for stakeholders to utilize. Specification provisions typically included for contractor and batch plant guidance on previous pilot projects include those associated with materials, mixture proportioning, batching, stockpile management, placement, instrumentation and access (Rao and Darter 2013, Barrett et al. 2015).

The benefits of laboratory concrete such as controlled environments and predictable outcomes are rarely present when transferring to the field. Therefore, specification requirements and other field guidance on methods for pre-wetting of the LWA are critical to the success of internally cured concrete. Therefore, an ICC specification is necessary for the concrete supplier to provide guidance on good batching techniques.

In actual practice, it has been found that a water sprinkler system works the best to moisture condition the LWA (Streeter et al. 2012, Barrett et al. 2015). The wicking action of the capillaries in the aggregate allow for a more uniform and faster saturation as compared to submerging the aggregates under water (Villareal and Crocker 2007). The key to successful implementation of ICC is to assure proper moisture conditioning of the LWA as without this, additional problems with variable unit weight, slump loss, pumpability and finishability will likely occur (Villareal and Crocker 2007). When the LWA is properly saturated, the pumpability of the concrete remains unaltered (Villarreal and Crocker 2007). It is recommended by Villarreal that a minimum 5 inch diameter pump line be used.

2.3.2 Field Instrumentation Methods and Testing

To validate that the benefits of ICC are being achieved in the field, instrumentation has been placed within bridge decks to monitor internal humidity and moisture as well as strain (Ardeshirilajimi et al. 2016). As mentioned previously, the internal humidity of internally cured concrete should effectively remain near 100% until complete hydration of the cement has occurred. On the contrary, the internal humidity of conventional concrete, especially in bridge deck mixtures with high cementitious contents, should fall below 100% as the cement paste hydrates (Bentz and Weiss 2011).

Commercially available humidity sensors have been utilized in such applications (Nantung 2016). However, other state agencies have reported a high failure rate of these sensors due to their lack of ruggedness when installed into fresh concrete during placement (Rupnow 2017). Therefore, some types of sensors are installed retroactively to avoid the initial impacts associated with concrete placement. For ease of installation of these sensors and to avoid drilling, greased dowels or other “plugs” are prepared and installed at the desired sensor location prior to concrete placement. After initial setting of the concrete, the plugs are removed, the sensors are installed, and the holes are filled with a temporary or permanent repair material.

To measure the strain within the bridge deck, vibrating wire strain gauges are commonly tied to the reinforcing steel prior to placement of the concrete. Each gauge has wires running from the gauge to an instrumentation box that is mounted near one of the bridge abutments. The strain observed in the ICC sections of the bridge deck tend to have less magnitude than the strain observed in the conventional concrete sections of the bridge deck. Ardeshirilajimi et al. (2016) performed this strain gage procedure within a bridge deck in Illinois in 2015.

2.3.3 Construction Challenges and Lessons Learned

As stated previously, internally cured concrete has been successfully used in the field by other state agencies, and the benefits observed have been well documented. However, some construction challenges and lessons learned have also been documented. Review of the literature indicates that additional construction considerations and challenges vary from state to state and project to project but can be generalized to include the following (ACI 2013, Bentz and Weiss 2011, Barrett et al. 2015):

- Determination of the quantity of prewetted LWA within the concrete mixture to ensure internal curing benefits
- Measuring and accounting for the moisture state of prewetted LWA stockpiles
- Providing adequate guidance to overcome concrete producer and contractor skepticism and resistance to change
- Observing the benefits of ICC in field projects
- Ensuring that cracking due to structural or construction issues does not interfere with the performance evaluation of ICC in the field
- Justifying increased initial costs due to addition of manufactured LWA

2.4 Research Needs

As presented in this chapter, extensive research to identify, quantify, and evaluate the benefits of use of ICC have been performed. Based upon the results of this literature review, the research needs for successful implementation of ICC in North Carolina can be broken down into three categories. First is the need to demonstrate that the benefits of ICC can be achieved using locally available materials and bridge and pavement mixture designs commonly used in North Carolina. The second research need is for specifications to guide the implementation of ICC in North Carolina concrete infrastructure. The third research need is the construction and assessment of a pilot project to validate, in the field that ICC is a beneficial technology for North Carolina and that adequate specification provisions support this effort.

CHAPTER 3: MATERIALS DESCRIPTION AND CHACTARIZATION

The purpose of this study was to investigate the benefits of internal curing using locally available materials and to develop specifications for the future use of internally cured concrete by the NCDOT. To achieve these goals, it is important to characterize and evaluate the materials used in the concrete. This chapter provides details on materials utilized in this study, including sources, physical properties, and other characteristics supporting use in concrete mixtures.

3.1 Lightweight Fine Aggregates

As mentioned in the literature review, the process of internal curing can be achieved through use of a variety of materials including prewetted lightweight aggregates, prewetted crushed returned concrete fines, superabsorbent polymers, and prewetted wood fibers. For this study, prewetted lightweight aggregates were chosen as the agent that will deliver the additional moisture to the concrete to facilitate hydration. Also for this study, fine lightweight aggregate was selected rather than intermediate or coarse lightweight aggregate. Fine lightweight aggregate was selected due to the increased benefits of spatial distribution of the internal curing reservoirs within the concrete matrix (Bentz and Snyder 1999, Castro et al. 2011).

As the lightweight aggregates are critical to the success of internally cured concrete, a fair amount of emphasis was placed on their material characterization. This is because their specific mechanical properties are utilized to adjust the various concrete mixture designs. For this study, two lightweight aggregate suppliers were chosen to provide material for the project based on their ability to supply the North Carolina market for lightweight aggregates. The first lightweight aggregate is an expanded lightweight slate produced in North Carolina, referred to subsequently as Lightweight Fine Aggregate #1 (LWA1). The second lightweight aggregate is an expanded shale produced in Kentucky, referred to subsequently as Lightweight Fine Aggregate #2 (LWA2).

3.1.1 Experimental Procedures and Results

In the following sections the experimental procedures and corresponding results that were determined for the lightweight aggregates are presented. Tests were performed to determine the following mechanical properties: density (dry rodded unit weight), relative density (specific gravity), absorption, gradation (sieve analysis) and desorption.

3.1.1.1 Density, Relative Density (Specific Gravity) and Absorption

For the lightweight aggregate, density, specific gravity and absorption were performed in accordance with ASTM C128, "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate." However, as Bentz and Weiss (2011) address in their 2010 summary of internal curing, ASTM C128 specifically states, "This test method is not intended to be used for lightweight aggregate." ASTM C128 presents a test method known as the 'cone method' to produce a sample of aggregate that is in SSD (Saturated Surface Dry) conditions. These conditions are easily achieved and reproduced with low absorptive normalweight aggregates but become significantly more difficult and variable with high absorptive lightweight materials (Miller et al. 2014).

Another testing standard, ASTM C1761, "Standard Specification for Lightweight Aggregate for Internal Curing of Concrete," documents an alternative method for determining the absorption of lightweight aggregate known as the ‘paper towel method.’ This method can provide similar information to the ‘cone method’ without the complexities associated with the angular nature of the particles (Castro et al. 2011). The ‘paper towel method’ involves taking a sample of soaked LWA and repeatedly drying the exterior of the aggregate with paper towels until the paper towels no longer collect moisture. At this point, the aggregate is deemed to be in the SSD condition.

This test method, however, is subject to large amounts of variability. Discrepancies can be found in the differences in absorptive capacity of the paper towels as well as the amount of drying pressure applied, both of which the standard lacks detail. Furthermore, the time of testing can be quite extensive. Therefore, the ‘centrifuge method’ may provide an alternative test which has recently been evaluated for internal curing applications (Miller et al. 2014). In this method, a sample of soaked LWA is placed in a centrifuge and surface moisture is extracted from the sample as the centrifuge rotates at 2000 rpm for three minutes. After the three minute rotation in the centrifuge, the sample is considered to be in the SSD condition. This test method has been shown to provide considerably less variability than the conventional paper towel method, and also requires significantly less time (Miller et al. 2014). Although a procedure for the ‘centrifuge method’ exists, the relationship between it and the ‘paper towel method’ have yet to be recognized by ASTM.

Absorption capacities were determined by the ‘paper towel method’ for 24 hour absorption and by the centrifuge method for 24 and 72 hour absorption. Absorption time varies because the absorption of many LWAs will continue over the course of many days. Therefore, absorption properties of lightweight aggregates are typically marked with a time descriptor, such as reporting the 24 h absorption capacity of a specific LWA (Bentz and

Weiss 2011). The average results of absorption testing in the laboratory are presented below in Table 3.1.

Table 3.1: Absorption properties of LWA used for this study

Method	**Paper Towel Method**	**Centrifuge Method**	
Absorption Time	**24 hour**	**24 hour**	**72 hour**
LWA1	13.02%	10.33%	11.33%
LWA2	22.40%	20.31%	23.94%

Despite the differences in absorption determination, the remainder of ASTM C128, "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate" is utilized to determine mechanical properties of LWAs including the density (dry rodded unit weight) and relative density (specific gravity). Testing of LWA1 determined that this material has a dry rodded unit weight of 65 pounds per cubic foot and a relative density (specific gravity) of 1.69. Testing of LWA2 yielded a density of 62 pounds per cubic foot and a relative density (specific gravity) of 1.52.

3.1.1.2 Sieve Analysis

A sieve analysis of fine aggregate provides a gradation curve which can be used to compute a fineness modulus. The fineness modulus of the fine aggregate is used in identification of the mixture proportions. These tests were performed in accordance with ASTM C136, "Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates." Both fine lightweight aggregates included in this study possess similar fineness moduli with 3.07 and 3.14 being the average results for LWA1 and LWA2, respectively. Due to this close fineness modulus, the research team elected to use a similar assumed fineness modulus for both materials when determining mixture proportions. The gradation curves for both lightweight aggregates as well as the normalweight sand are provided in Appendix B, Figure B.1.

3.1.1.3 Desorption

The ability of the LWA to release water at high relative humidity can be quantified by measuring the absorption/desorption properties of the LWA particles (Bentz and Weiss 2011). Desorption testing, as described in ASTM C1761, is used to determine the amount of absorbed water that will be released when lightweight aggregate that is initially in SSD condition is stored in an environment at 94% relative humidity. There are two methods presented in the standard as alternatives for obtaining a controlled relative humidity environment. The first of which is an environmental chamber capable of maintaining a relative humidity of 94.0 ± 0.5% and a temperature of 23.0 ± 1 °C. The second method was the alternative chosen for this work. This method involves using 300 grams of a supersaturated solution of potassium nitrate placed into a wide-mouth plastic or glass jar with a tightly fitting lid. A frame of non-corroding material was placed inside the jar to support the weighing pan holding the specimen. This setup provides similar conditions to a 94% relative humidity chamber and was held in a constant temperature environment of 23.0 ± 1 °C per the ASTM C1761 procedure. The results of desorption testing yielded 99.1% and 98.9% desorption at 94% relative humidity for LWA1 and LWA2, respectively, indicating that both aggregates are suitable for use as internal curing agents (Henkensiefkin 2008).

3.2 Normalweight Coarse and Fine Aggregates

As the purpose of this study is to compare the properties of conventional concrete and internally cured concrete, the normalweight aggregate materials (both fine and coarse) remained constant for all concrete mixtures prepared in the laboratory. Normalweight aggregate providers were selected based upon input from NCDOT personnel regarding frequently utilized aggregates for concrete mixtures used in bridge decks in the region of the pilot project. The normalweight coarse aggregate selected for the project comes from a quarry in the Piedmont region of the state in Cary, North Carolina. The product selected is No. 67 crushed granitic gneiss stone of which 100% passes the one inch sieve. The normalweight fine aggregate, a natural silica sand, selected for the project also comes from the Piedmont region of the state, from a pit in Lemon Springs, North Carolina.

3.2.1 Coarse Aggregate Density, Relative Density and Absorption

Density, relative density (specific gravity) and absorption tests for the normalweight coarse aggregate were performed in accordance with ASTM C127, “Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate.” The results of ASTM C127 testing of the coarse aggregate yielded a density (dry rodded unit weight) of 96.5 pounds per cubic foot, a relative density (specific gravity) of 2.62 and absorption capacity of 0.8%.

3.2.2 Fine Aggregate Gradation, Density, Relative Density and Absorption

Sieve analysis of the normalweight fine aggregate was performed in accordance with ASTM C136, “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates.” The sieve analysis resulted in a fineness modulus of 2.72. The gradation curve for the normalweight fine aggregates can be found in Appendix B, Figure B.1. Density, relative density (specific gravity) and absorption tests were performed in accordance with ASTM C128, “Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate.” The results of ASTM C128 testing on the normalweight fine aggregate yielded a density (dry rodded unit weight) of 97.2 pounds per cubic foot, a relative density (specific gravity) of 2.44 and absorption capacity of 0.18%.

3.3 Cementitious Materials

As the purpose of this study was to compare the properties of conventional concrete and internally cured concrete, similar to the normalweight aggregates, cementitious materials also remained constant for all concrete prepared in the laboratory. A description of the source of each cementitious material as well as characterization information is provided below.

3.3.1 Portland Cement

The portland cement chosen for this project was produced and manufactured by a plant located in Holly Hill, South Carolina. This cement is commonly used in concrete projects in the Piedmont region of North Carolina. The cement is considered Type I/II cement and meets the requirements of ASTM C150, “Standard Specification of Portland Cement.” The relative density (specific gravity) of the cement is 3.15 as provided by the manufacturer. The mill report for the cement is provided in Appendix B in Figure B.2.

3.3.2 Fly Ash

Five of the thirteen concrete mixtures prepared in the laboratory portion of this project utilized a partial replacement of portland cement for fly ash. The state of North Carolina allows for a 20% replacement of cement by mass in accordance with North Carolina Standard Specifications section 1024, “Materials for Portland Cement Concrete.” The specification states that for the Class F fly ash pozzolan, the replacement rate is “20% by weight of required cement content with 1.2 lb. Class F fly ash per lb. of cement replaced." The fly ash chosen for this project is a Class F fly ash sourced from the Roxboro power station in Roxboro, North Carolina. Further characterization information on this fly ash is provided in the Appendix B in Figure B.3.

3.4 Chemical Admixtures

Three commercially available admixtures were utilized in the laboratory concrete mixtures prepared as part of this study, an air entraining admixture and a high-range water-reducing admixture. The purpose of the use of chemical admixtures was to achieve the desired air content and workability (slump) for each fresh concrete mixture. NCDOT Standard Specifications for Roads and Structures state that for Class AA concrete mixtures the provided air content should be 5.0% ± 1.5% in the freshly mixed concrete. The specifications further state that the maximum slump for vibrated Class AA concrete is 3.5 inches. Laboratory concrete prepared as part of this study possessed a maximum slump of 3.5 inches in accordance with NCDOT specifications. The acceptable slump range for this project was set at 1.5 to 3.5 inches. Entrained air content for laboratory concrete was kept at a relatively tight tolerance of 5.5% ± 0.5% in order to ensure consistency between test results and to further ensure that differences in laboratory results were mostly attributed to changes in materials, rather than air content.

The air entraining admixture (AEA) chosen for use in this project was a synthetic AEA. All laboratory concrete mixtures prepared as part of this study used this AEA to achieve the desired air content, except the two mixtures which incorporated a latex (polymer) modification. The manufacturer of this air entraining admixture recommends using a dosage of 0.125 to 1.5 fluid oz/cwt. In order to achieve the desired air content in the laboratory, the actual dosage of AEA ranged between 0.5 fluid oz/cwt (typical for the internally cured pavement mixture) and 12.8 fluid oz/cwt (typical for mixtures where fly ash was incorporated).

The high-range water-reducer chosen for use in this project was a polycarboxylate-based high-range water reducer. This product was used in many of the concrete mixtures prepared in the laboratory to achieve the desired workability. The manufacturer recommends using a dosage of 2 to 10 fluid oz/cwt. To achieve a slump between 1.5 and 3.5 inches, the actual dosage in laboratory concrete mixtures ranged from 0 fluid oz/cwt (typical for internally cured pavement and latex-modified mixtures) and 3.2 fluid oz/cwt (typical in conventional concrete mixtures not utilizing internal curing).

The LMC prepared as part of this project utilized a commercially available latex modifying admixture that was used in virtually all approved LMC in North Carolina according to NCDOT data. The dosage rates of the latex-modifying admixture were determined through analysis of approved mixtures submitted for LMC provided by NCDOT personnel. Review of these submittals indicated that a typically utilized dosage of latex modifying admixture is 17.5 gallons per cubic yard, which was used in all LMC mixtures batched in this part of the study. The latex polymer provided is 48% solids, weighs 8.58 lb./gal and has a relative density (specific gravity) of 1.02. Further information on the latex polymer is provided in Appendix B in Figure B.4.

CHAPTER 4: LABORATORY BATCHING AND TESTING METHODOLOGY

4.1 Introduction

As discussed in Chapter 2, the benefits of internally cured concrete have become apparent to state highway agencies, and these mixtures have been targeted for use in different types of concrete infrastructure, including bridge decks, pavements, precast bridge deck components, and other applications. Of interest to NCDOT is use of internally cured concrete in bridge and pavement applications. In this chapter, the methodology behind identification of the types of concrete mixtures included in this study, as well as mixture proportioning, batching, and testing, is presented.

4.2 Overview of Laboratory Concrete Mixtures

In collaboration with the NCDOT, the research team targeted 13 different concrete mixtures to batch and test within the laboratory. Ten of those mixtures are proportioned to meet bridge deck Class AA mixture specifications, with materials (including cementitious materials and LWAs) varied to provide data comparing the performance of a variety of internally cured and control mixtures. Two of the other three mixtures are latex-modified concrete overlay mixtures (one internally cured mixture and one control mixture). One other mixture included in the matrix is an internally cured pavement mixture, with the control (not internally cured mixture) previously batched and tested as part of a previous research project, NCDOT RP 2015-03 (Blanchard 2016, Chimmula 2016, Medlin 2016). The mixture matrix provided in Figure 4.1 details the different variations in the 13 concrete mixtures included in this study.

Several parameters were held constant throughout the mixture matrix to aid in evaluating changes in performance of internal curing mixtures vs. non-internal curing mixtures. For bridge deck mixtures, it was determined that the water/cementitious material (w/cm) ratio would be held constant at 0.35 for all mixtures to ensure the required strength of 4,500 psi at 28 days was reached. NCDOT provided a range of cement contents commonly used in bridge deck mixtures within the state. This range spread from 639 to 715 pounds of cement per cubic yard of concrete. It was recommended by the NCDOT that the mixtures utilize a cement content in the higher range, noting that 715 pcy (pounds per cubic yard) is the most commonly utilized cement content among mixtures submitted for approval for construction. Therefore, 715 pcy was chosen as the cement content for the five straight cement concrete mixtures shown on the far left side of the mixture matrix in Figure 4.1, labeled as “PCC”.

The pilot project is located in a region of the state in which NCDOT requires that fly ash be included in all Class AA concrete for bridge decks. Therefore, five of thirteen concrete mixtures included in the laboratory program, labeled as “PCC w/ Fly Ash” in the mixture matrix, contain a 20% replacement of portland cement for Class F fly ash at a ratio of 1:1.2 (mass basis). This replacement rate results in cementitious contents of 572 pcy of portland cement and 172 pcy of supplementary cementitious materials (fly ash) per cubic yard of concrete, which are commonly utilized cementitious materials contents for bridge decks in the state per NCDOT mixture design submittals.

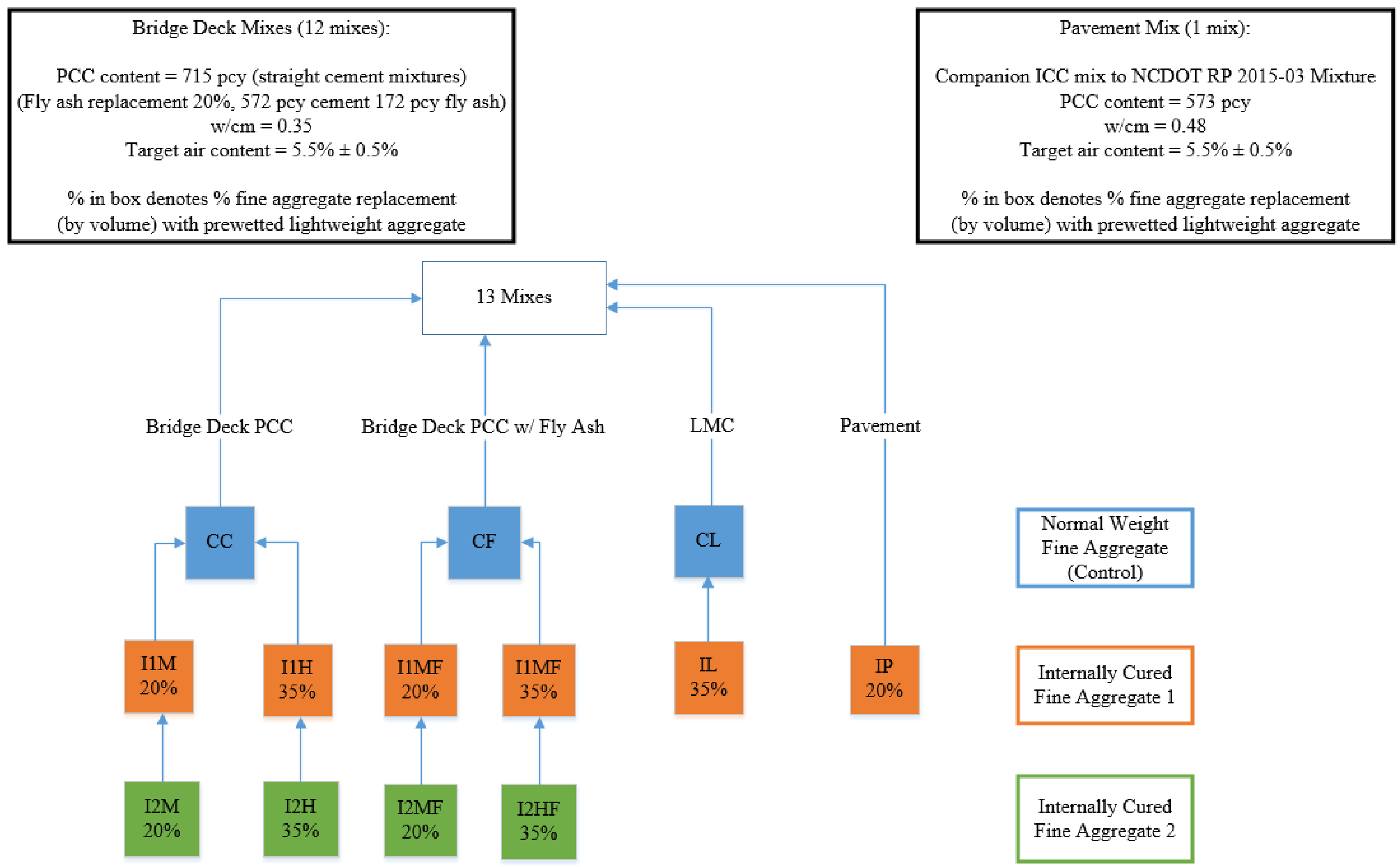


Figure 4.1: Mixture matrix for laboratory concrete for NCDOT RP 2016-06

As the fineness moduli of the LWA are similar and the normalweight sand remains constant for all mixtures, the coarse aggregate factor also remains constant for the ten class AA bridge deck mixture designs. This factor was selected from the ACI 211.1-91 mixture proportioning procedure. With a nominal maximum size of aggregate of one inch and an average fineness modulus of 2.90, a coarse aggregate factor of 0.66 was selected. Multiplying this factor by the dry rodded unit weight of the coarse aggregate (2600 pcy) provides the amount of coarse aggregate per cubic yard of concrete which equates to 1720 pcy. As mentioned earlier, this amount of material was held constant for the ten Class AA bridge deck mixtures included in this study.

The final variable within these ten bridge deck mixtures is the percent replacement of normalweight fine aggregate with prewetted lightweight aggregate. This, as discussed in Chapter 2, is the only significant difference between traditional concrete mixture designs and internal curing mixture designs. Two control conventional concrete mixtures, were developed with no replacement of normalweight fines for prewetted LWAs – one using only cement (or “straight cement”) (denoted as CC in this study) and one containing fly ash (denoted as CF in this study), shown in blue on the mixture matrix. Of the remaining eight mixtures, four utilized prewetted LWA1 (shown in orange on the mixture matrix) and four utilized prewetted LWA2 (shown in green on the mixture matrix). NCDOT desired that concrete mixtures be developed in a manner that contained both a moderate replacement level as well as a high replacement level of prewetted LWA fines for normalweight fines. This is to move towards identification of optimal quantities of LWA replacement for use in future specification provisions. Based off other state agencies experiences, it was determined by NCDOT to test internal curing mixtures in the laboratory with 20% and 35% replacement percentages of LWFA for normalweight sand. Further discussion on these replacement percentages in presented in Section 4.3.

The final three mixtures, shown on the right side of the mixture matrix, include two LMC mixtures, one conventionally cured (shown in blue) and one internally cured (shown in orange), as well as an internally cured concrete pavement mixture (shown in orange). The control (non-internally cured) concrete pavement mixture was batched and tested as part of a previous research project, NCDOT RP 2015-03. LWA1 was used for the internal curing mixtures for both the LMC and pavement concrete. Supporting information regarding these mixtures is presented in subsequent sections of this chapter.

4.3 Mixture Design Approach

As discussed in the literature review of this report, Bentz and Snyder developed a relationship in 1999 between the water demand of the hydrating mixture and the supply that is readily available from the internal reservoirs of the LWA. This relationship is expressed as Eq. 1 in Chapter 2, Section 2.1.3. In Eq. 4.1, the left-hand side of the equation represents the water demand of the hydrating mixture and is composed of the cement (or binder) factor of the concrete mixture, C_f, the chemical shrinkage of the binder at 100% reaction, *CS*, and the expected maximum degree of reaction for the binder, α_{max}, ranging from 0 to 1. According to Bentz et al. (2007), a typical CS (chemical shrinkage) for portland cement is on the order of 0.07 mL/g but values for fly ash can be 2 and 3 times greater. The expected maximum degree of reaction of the binder, α_{max}, can be assumed to be 1 if the w/c is equal to or greater than 0.36. Otherwise, α_{max} is given by [(w/c)/0.36] for w/c < 0.36. The cementitious content chosen for this study of (w/c = 0.35) provides an α_{max} of 0.972. The five mixtures on the left side of the mixture matrix utilize straight portland cement at

715 pcy and a CS factor of 0.07. Substituting this known information into the demand side of the equation results in the following equation.

$$(715\ pcy)\times(0.07\ mL/g)\times(0.972) = 48.65 = S\times\Phi_{LWA}\times M_{LWA} \tag{4.1}$$

Assuming saturated conditions of the LWA, S can be assumed to equal one. Φ_{LWA} is the absorptive capacity of LWA which is known. Substituting these values into the supply side of the equation provides the following two equations for LWA1 and LWA2 straight cement mixtures, respectively.

(1) $48.65 = (1)\times(11.33\%)\times M_{LWA}$

(2) $48.65 = (1)\times(23.94\%)\times M_{LWA}$

The suggested quantity of M_{LWA} can now be determined for each LWA. According to this approach, for the provided straight cement content mixtures, 429.4 pounds per cubic yard of dry LWA1 or 203.2 pounds per cubic yard of dry LWA2 are required in SSD condition to provide the maximum expected degree of cement hydration. However, this method, presented by Bentz and Snyder (1999), is used from here on out as a reference of comparison for the more simplified approach selected by the research team and NCDOT.

Using the masses of prewetted LWA to provide maximum cement hydration, mixture proportioning per ACI 211.1 procedure can be performed to determine quantities of other materials. As explained in Section 4.2, for the ten bridge deck mixtures, the selected cementitious contents of 715 pcy for straight cement and 572 pcy and 172 pcy for cement and fly ash mixes were utilized in the proportioning process of bridge deck mixtures. The selected w/cm ratio and the amount of coarse aggregate for bridge deck mixtures was held constant at 0.35 and 1720 pcy, respectively. Relative densities (specific gravities) are known for all materials and a target entrained air content was established at

5.5% for all mixtures. Using this information along with the ACI 211.1 procedure, a weight and volume analysis of the concrete matrix was performed to determine the volume of space that the fine aggregate (both normalweight and lightweight) would occupy in each mixture. Based on the aforementioned inputs, it was calculated that the total amount of fine aggregate encompasses 27.07% and 25.13% of the volume of the straight cement bridge deck mixtures and cement and fly ash bridge deck mixtures, respectively. These calculations are shown in Table 4.1 and Table 4.2, below.

Table 4.1: Mixture proportioning of straight cement bridge deck mixtures

Straight Cement	**Density (lb/cy)**	**Relative Density**	**Volume/cy of Concrete**
Cement	715	3.15	0.1347
Water	250	1	0.1484
Coarse Aggregate	1720	2.61	0.3911
Air	0	0	0.0550
Fine Aggregate			0.2707
Total			**1.0000**

Table 4.2: Mixture proportioning of cement and fly ash bridge deck mixtures

Straight Cement	**Density (lb/cy)**	**Relative Density**	**Volume/cy of Concrete**
Cement	572	3.15	0.1078
Fly Ash	172	2.2	0.0464
Water	250	1	0.1484
Coarse Aggregate	1720	2.61	0.3911
Air	0	0	0.0550
Fine Aggregate			0.2513
Total			**1.0000**

Based off the understanding that there is a finite amount of volume available for fine aggregates as seen above, the Bentz and Snyder approach becomes unfeasible for low absorptive LWA. This becomes apparent when observing the amount of dry LWA1 the Bentz and Snyder approach recommends, 429.4 pounds per cubic yard. Converting this weight of LWA to a volume within a cubic yard of (straight cement) concrete, similar to the method used in Table 4.1, LWA1 would consume 15.22% of the 27.07% available volume. Converting this volume to a percentage of LWA to replace normalweight fine aggregate results in a 56.22% replacement, which exceeds the LWA replacement quantities utilized by many state agencies currently specifying ICC for use in bridge deck concrete mixtures.

As discussed in Chapter 2, many state agencies tend to simplify the Bentz and Snyder approach to aid the ready-mix plants with design and field implementation of ICC. This simplification is a key difference between a number of state agencies' approaches to ICC mixture design. For example, as mentioned in Chapter 2, NYSDOT specifies that a 30% replacement of LWA for normalweight fines be utilized, regardless of the absorptive capacity of the LWA or the chosen cement content. However, NYSDOT also specifies the use of a LWA with a minimum absorption capacity of 15%. For research for Michigan, Wei and Hansen (2008) chose LWA replacement ratios of 20% and 40%. Speck (2013) recommended LWA replacement ratios of 10 to 25% for internally cured concrete mixtures to be used in Illinois. In a study evaluating ICC for pavement applications, Rao and Darter (2013) studied ICC pavements with a 30 – 33% LWA replacement for normalweight fine aggregate.

It is clear that the ideal replacement ratio varies based on many factors including cement content, LWA absorption capacities and saturation conditions, which as discussed earlier, can vary drastically based on the method of conditioning. Based on preferences of NCDOT personnel, the simplified approach of selecting two replacement ratios (in lieu of the Bentz and Snyder approach) was utilized for the laboratory testing program: 20%

(moderate) and 35% (high) replacement by volume of prewetted LWA for normalweight sand. Assuming that the LWA is in SSD condition and the normalweight aggregate (coarse and fine) is oven dry, the concrete mixture proportions presented in Table 4.3 were utilized for the laboratory portion of this study. For latex-modified concrete (LMC), 210.2 pcy of latex modifying admixture (48% polymer, 52% water) was used in addition to the stated amount of water. Furthermore, #78 coarse aggregate was used in the LMC, rather than #67 stone, since LMC overlays are typically shallow (approximately 2”) in thickness. These changes are denoted with an asterisk (*) in the table below. The subsequent sections discuss batching procedures and final mixture proportions.

Table 4.3: Mixture proportions for laboratory testing

Mixture Type	ID	Description	Weight (lb/cy)					
			Cement	Fly Ash	Water	CA	NWFA	PWLA
Bridge Deck	CC	Conventional Concrete, Straight Cement	715	0	266.0	1720	1113	0
	I1M	Internal Curing, LWA #1, Moderate Replacement, Straight Cement	715	0	265.6	1720	890	154
	I2M	Internal Curing, LWA #2, Moderate Replacement, Straight Cement	715	0	265.6	1720	890	139
	I1H	Internal Curing, LWA #1, High Replacement, Straight Cement	715	0	265.3	1720	723	270
	I2H	Internal Curing, LWA #2, High Replacement, Straight Cement	715	0	265.3	1720	723	243
	CF	Conventional Concrete, Cement and Fly Ash	572	172	266.0	1720	1113	0
	I1MF	Internal Curing, LWA #1, Moderate Replacement, Cement and Fly Ash	572	172	265.6	1720	890	154
	I2MF	Internal Curing, LWA #2, Moderate Replacement, Cement and Fly Ash	572	172	265.6	1720	890	139
	I1HF	Internal Curing, LWA #1, High Replacement, Cement and Fly Ash	572	172	265.3	1720	723	270
	I2HF	Internal Curing, LWA #2, High Replacement, Cement and Fly Ash	572	172	265.3	1720	723	243
Pavement	IP*	Internal Curing Pavement Mixture, LWA #1, Moderate Replacement	573	0	298.2	1798	770	252
LMC	CL**	Conventional Latex-Modified Concrete	658	0	153.3*	1304*	1510	0
	IL**	Internally Cured Latex-Modified Concrete, LWA #1, High Replacement	658	0	152.4*	1304*	921	345

* denotes concrete pavement mixture, ** denotes LMC mixture

4.4 Batching Procedure

The batching, mixing, transportation, placing and finishing of ICC is not significantly different from any other common concrete practice (ACI 308-213). As detailed in the literature review, other researchers and stakeholders have indicated that prewetting of the lightweight aggregate is the only significant difference in ICC construction, and monitoring and controlling this additional moisture in field stockpiles and during batching is the key construction challenge for successful ICC implementation. Methods of prewetting were discussed in Chapter 2 of this report and include sprinkling of a stockpile as well as full emersion of the LWA.

For batching of concrete mixtures in the laboratory, a sufficient amount of LWA, enough to create a small stockpile (3 – 5 cubic feet), was placed in a container. The container was then filled with water until the LWA was completely submerged. The LWA remained in this condition for a minimum of 48 – 72 hours. As batching of laboratory specimens typically occurred in the morning, the LWA was drained at the end of the previous work day. While draining the excess water, care was taken to ensure loss of only a minimal quantity of fines. After a fair amount of water was drained, the LWA was removed from the container and placed on a piece of plastic sheeting (to avoid contamination) in the shape of a small stockpile. The LWA was then allowed to further drain overnight, generally for a period of 12 – 15 hours.

During the morning of batching, the small LWA stockpile was then “turned” several times using a shovel. Care was taken to not disturb the bottom 2 – 3 inches of the stockpile as this was typically where the drained water collects. The quantity of LWA needed for each concrete mixture that day was then sampled from the stockpile. However, as discussed previously in Chapter 2, the LWA still contains some free (surface) moisture that must be accounted for prior to mixing. This was done through the use of the centrifuge as described by Miller et al. (2014). After several iterations of laboratory batching and mixing, it was observed that the laboratory stockpile management procedures consistently utilized resulted in reasonably consistent surface moistures for each type of LWA.

Therefore, an average surface moisture for each LWA used in this study was calculated and utilized throughout this study. For laboratory stockpiles, it was determined that on average LWA1 retained 5.5% surface moisture after the draining period and LWA2 retained 4.0% surface moisture after the draining period. Batch water quantities were adjusted for this excess surface water to ensure the target w/cm was met.

Once the LWA was properly conditioned, batching was performed in general accordance with ASTM C685, "Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing." While batching, the LWA and a portion of the mixing water were placed in the mixer first, per guidance in ACI 308-213 (ACI 2013). Next, the coarse aggregate and normalweight fine aggregate were placed in the mixture. Following the normalweight aggregate, a portion of the mixing water was combined with the necessary admixtures and added to the mixer. Lastly, the cement, fly ash and remaining mix water were added to the mixer. For laboratory testing, the drum mixer was run for 3 minutes immediately after the addition of the cement. The mixture was then allowed to rest for three minutes, and then underwent a final mixing period of two minutes.

Due to laboratory constraints, including a six cubic foot portable concrete mixer with a maximum capacity of 2.5 cubic feet for proper mixing, each mixture was batched three times to produce enough concrete to facilitate casting of all of the desired test specimens. The ideal batch size was determined to be 2.3 cubic feet. Batch 1 of each mixture was utilized to produce test specimens for mechanical properties, including seventeen 4 in. x 8 in. cylinders for testing of compressive strength, surface resistivity, rapid chloride permeability testing (RCPT) and coefficient of thermal expansion (CTE), two 6 in. x 12 in. cylinders to test for Modulus of Elasticity (MOE) and Poisson's Ratio as well as three 4 in. x 4 in. x 11 in. ASTM C157 drying shrinkage bars. Batch 2 was utilized to provide fresh concrete for two Super Air Meter (SAM tests) and to prepare three freeze/thaw test beams (ASTM C666), test specimens for hardened air void analysis, and several 4 in. x 8 in. cylinders to test compressive strength and evaluate conformity with other similar batches. Batch 3 was utilized to prepare three restrained shrinkage rings for ASTM C1581 testing, as well as several 4 in. x 8 in. cylinders tested for conformance. Further details on testing procedures and the corresponding results are presented in Chapter 5 of this report.

CHAPTER 5: TESTING PROGRAM FOR LABORATORY CONCRETE

5.1 Testing of Fresh Concrete Properties

Prior to the preparation of specimens for hardened concrete testing, fresh concrete property tests were performed on each batch of concrete to ensure compliance with NCDOT specifications for bridge deck concrete, as well as to ensure uniformity between different batches of the same concrete mixture. Tests that were performed included slump, unit weight, air content and temperature of the concrete.

5.1.1 Design and Predicted Performance

The methods used for fresh concrete tests, along with the test results, are provided in subsequent sections. All fresh concrete testing was performed in controlled laboratory conditions at UNC Charlotte, in accordance with the ASTM standards as presented below.

5.1.1.1 Slump

Slump tests were performed according to ASTM C143, "Standard Test Method for Slump of Hydraulic-Cement Concrete" on each batch to confirm that the batch met the target slump for NCDOT Class AA concrete. The slump test also provided a simple method of quality control for ensuring consistency between batches of the same mixture. The slump requirements for Class AA concrete state that the maximum allowed slump is 3.5 inches. This maximum slump is not to be exceeded as excessive slump will yield a difficult final product for bridge deck concrete due to the inability to construct it with a slight crest (camber) to aid in drainage. The target slump for the laboratory concrete testing program was set at 1.5 to 3.5 inches. However, to aid in performance comparisons between mixtures, it was desired to keep a constant water-to-cement ratio for mixtures of each type (e.g. bridge deck mixtures, latex modified mixtures, and pavement mixtures) across the laboratory testing program. Due to the differences in materials used, and proportioning of each concrete mixture, water-reducing admixtures (described in Section 3.4) were utilized to achieve a slump within the target range.

5.1.1.2 Entrained Air Content

It is important that bridge deck concrete be appropriately air entrained to adequately resist the stresses associated with freezing and thawing. For all concrete mixtures in this study, air content testing was performed in accordance with ASTM C231, “Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.” Per NCDOT Standard Specifications, the required air content of all NCDOT mixtures is 6.0 ± 1.5% (NCDOT 2012). However, due to the research team’s desire to minimize variability within the laboratory concrete testing program that could potentially be attributed to changes in air content (rather than differences in materials), a tight requirement on entrained air was held constant at 5.5 ± 0.5%. Mixtures not meeting this air content requirement were discarded.

5.1.1.3 Unit Weight

Measuring the fresh density of concrete provided another relatively easy method of quality control between multiple batches of the same mixture. The fresh density of the concrete directly correlates to the air content of the concrete and can be used to provide early indication of proper mixture proportioning and air entrainment. Fresh density testing was performed with the same apparatus as the entrained air content using the pressure method; a container of known volume. Fresh density testing was performed in accordance with ASTM C138, “Standard Test Method for Density (Unit Weight), Yield and Air Content (Gravimetric) of Concrete.”

5.1.2 Experimental Results

Several trial batches were prepared and tested until fresh property targets (air content and slump) were consistently met. Once the appropriate admixture proportions were established, specimens were cast and allowed to cure per ASTM C192, “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory,” and tested at various ages. As mentioned previously in Chapter 4 and shown in Table 4.3, the following Mixture IDs are used in tables and figures within this chapter. Table 5.1, below, provides a list of Mixture IDs and a description of each concrete mixture.

Table 5.1: Identification of each concrete mixture and description

Mixture ID	**Concrete Mixture Description**
CC	Conventional Concrete Straight Cement
I1M	ICC LWA1 Moderate Replacement Straight Cement
I2M	ICC LWA2 Moderate Replacement Straight Cement
I1H	ICC LWA1 High Replacement Straight Cement
I2H	ICC LWA2 High Replacement Straight Cement
CF	Conventional Concrete w/ Fly Ash
I1MF	ICC LWA1 Moderate Replacement w/ Fly Ash
I2MF	ICC LWA2 Moderate Replacement w/ Fly Ash
I1HF	ICC LWA1 High Replacement w/ Fly Ash
I2HF	ICC LWA2 High Replacement w/ Fly Ash
IP1	ICC Pavement Mixture with LWA #1
PANN	Conventional Pavement Mixture (from NCDOT RP 2015-03)
CLMC	Conventional Latex Modified Concrete
ILMC	ICC Latex Modified Concrete

5.1.2.1 Slump

Slump test results were consistent with specifications for Class AA concrete. The target range of slump for this project was 1.5 to 3.5 inches. Each mixture required various amounts of water-reducing admixtures to achieve the desired slump, however, water-reducing admixture dosages were within the range of manufacturers' recommendations. Table 5.2, below, provides the slump results for the ten base mixtures for each batch of laboratory concrete along with results for air content and unit weight (discussed in Sections 5.1.2.2 and 5.1.2.3, respectively).

Table 5.2: Slump, air content and unit weight results of bridge deck mixtures

Mixture ID	**Slump (in.)**				**Air Content (%)**			**Fresh Unit Weight (pcf)**			
	Batch 1	**Batch 2**	**Batch 3**	**Ave.**	**Batch 1**	**Batch 3**	**Ave.**	**Batch 1**	**Batch 2**	**Batch 3**	**Ave.**
CC	2.50	2.50	2.25	2.42	5.5%	5.1%	5.3%	144.6	146.9	145.0	145.5
I1M	2.50	2.75	2.50	2.58	5.3%	5.9%	5.6%	141.0	141.4	141.0	141.1
I2M	2.25	2.50	1.75	2.17	5.2%	5.1%	5.2%	141.6	143.0	142.8	142.5
I1H	3.25	3.50	3.00	3.25	5.8%	5.0%	5.4%	138.8	140.2	141.2	140.1
I2H	3.00	2.75	2.50	2.75	5.0%	5.5%	5.3%	139.1	142.2	138.7	140.0
CF	3.25	2.75	2.25	2.75	6.0%	5.1%	5.6%	140.2	142.2	143.2	141.9
I1MF	2.00	1.50	2.25	1.92	5.0%	5.5%	5.3%	141.0	140.2	140.1	140.4
I2MF	2.00	***	2.00	2.00	5.0%	5.1%	5.1%	141.0	***	142.0	141.5
I1HF	2.25	***	1.75	2.00	5.0%	5.2%	5.1%	139.4	***	141.0	140.2
I2HF	2.00	2.75	***	2.38	5.1%	***	5.1%	138.7	137.5	***	138.1

*** indicates mixture not completed at time of thesis submission

5.1.2.2 Entrained Air Content

As discussed previously, air content was held constant at a target range of 5.5 ± 0.5%, and mixtures not meeting this tightly targeted air content were discarded. Results of ASTM C231 are also presented in Table 5.2, above. The amount of air entraining admixture required to provide the desired air content varied between mixtures. As stated in Chapter

3, most of the concrete mixtures that incorporated fly ash required the use of significantly more air entraining admixture. For some fly ash mixtures, the amount of air entraining admixture required to achieve the desired air content the manufacturer recommendations.

5.1.2.3 Unit Weight

Results for the unit weight of fresh concrete are also included in Table 5.2. As was expected, a correlation between quantity of LWA and fresh unit weight can be observed. Non-internally cured concretes (CC and CF) possessed the highest unit weights while high replacements of LWA for normalweight sand mixtures (I1H, I2H, I1HF, I2HF) possessed the lowest unit weights. LWA1 has a higher unit weight than LWA2 but this was not as noticeable in the concrete mixtures with moderate replacement. However, the slight <1% difference in air content shown in Figure 5.1 help to explain that. Furthermore, mixtures that contained fly ash had a slightly lower unit weight than mixtures that did not contain fly ash. A graphic illustrating the differences in fresh unit weight and average air content is provided below in Figure 5.1. Of note, the fresh unit weight of all mixtures was above the unit weight of 135 pcf, which is the limit for normalweight concrete in the AASHTO LRFD Bridge Design Specifications (2012), and also results in use of the lightweight concrete modification factor (λ) for strength reduction in concrete structural design per ACI 318.

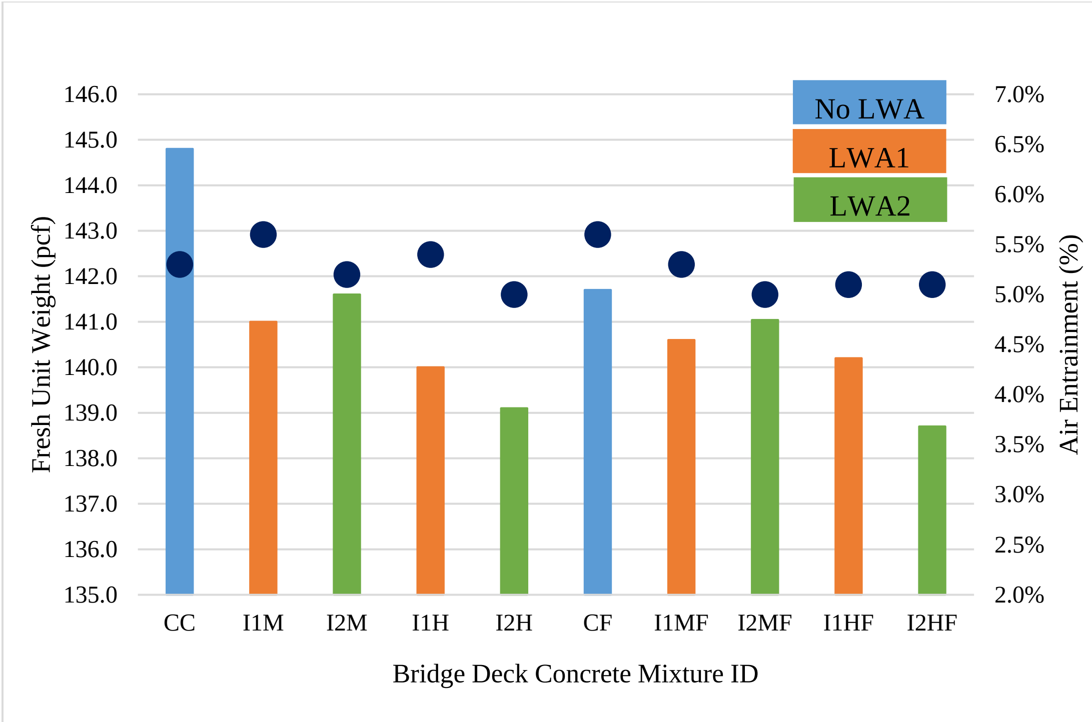


Figure 5.1: Fresh unit weight and average air content for bridge deck mixtures

5.2 Testing of Hardened Concrete Properties

The subsequent sections detail the results of hardened concrete testing. The results of the ten bridge deck concrete mixtures are included in this section. Testing results from the internally cured pavement concrete mixture and testing results of latex-modified concrete mixtures are presented in section 5.3.

5.2.1 Experimental Procedures

The test methods and procedures performed on conventional concrete and ICC specimens are presented in the following sections. Specimens were prepared and stored in accordance with ASTM C192, “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory.” The test methods are divided into three categories: mechanical properties, durability performance and shrinkage.

5.2.1.1 Mechanical Properties

The first category of tests performed were those to determine the mechanical properties of the concrete, which included compressive strength, modulus of elasticity, and Poisson's ratio. Test results for the coefficient of thermal expansion of these mixtures is also included in this section. These tests were performed on each of the 13 mixtures developed as part of the laboratory batching program, but as mentioned previously, results for the 10 bridge deck mixtures is presented in this section.

5.2.1.1.1 Compressive Strength

Tests to determine the compressive strength of each concrete mixture were performed in accordance with ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." Traditional 4 in. by 8 in. cylinders were cast for testing. Three cylinders were tested at each of the following ages: 3, 7, 28, and 90 days after casting.

5.2.1.1.2 Modulus of Elasticity and Poisson's Ratio

Tests to determine the modulus of elasticity (Young's Modulus) and Poisson's ratio of concrete specimens was performed in accordance with ASTM C469, "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." For each of the mixtures, two 6 in. by 12 in. cylinders were tested at 28 days after the day of casting.

5.2.1.1.3 Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) of materials represents the change in length per change in temperature. The CTE of concrete is measured by repeatedly evaluating the length change of cylindrical concrete specimens during a period of time where temperature changes are known and specified. Exposed to a temperature change, concrete with a higher CTE will expand and contract more than concrete with a lower CTE. According to Gudimettla et al. (2012), the CTE of portland cement concrete (PCC) can be influenced by many factors including aggregate types, coarse aggregate proportions, relative humidity, cement content, w/c ratio, porosity and the degree of hydration. In regards to bridge decks, which are generally constructed with high strength concrete that is restrained, a high CTE value can influence early-age cracking (Castrodale and Cavalline 2017).

The currently accepted standard for measuring the CTE of concrete is AASHTO T 336-11, "Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete", which was the procedure utilized for this study. In this test procedure, the change in length of a concrete cylinder is measured while submerged in a water bath which temperature fluctuates between 10°C and 50°C. Specimens are 4 ± 0.1 in. in diameter and 7 ± 0.1 in. in length and are tested in a consistent moisture state. Testing was performed at the UNC Charlotte laboratory utilizing the AFCT2 equipment, manufactured by Pine Instruments, which is capable of testing three specimens simultaneously.

5.2.1.2 Durability Performance Testing

The second type of tests performed as part of this work are those used to evaluate concrete's durability performance. Adequate durability is essential to obtaining a long service life for concrete bridges and pavements. Durability performance testing included surface resistivity, bulk conductivity, and rapid chloride permeability testing (RCPT). Tests of these concrete mixtures to evaluate freeze/thaw resistance are being performed as part of the larger research study supporting this work, but are not detailed in this thesis.

5.2.1.2.1 Surface Resistivity

Surface resistivity testing was performed in general accordance with AASHTO TP95-11, "Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration." The surface resistivity test is a non-destructive test method that uses electricity to estimate the permeability of hardened concrete, and can provide an indication of the ability of chlorides and other aggressive agents to penetrate the concrete and potentially cause corrosion and other materials related distress. Surface resistivity results have been shown to correlate with the results of rapid chloride permeability testing (RCPT) (Rupnow and Icenogle 2011). The equipment used for surface resistivity testing was the Resipod surface resistivity meter, manufactured by Proceq. Three 4 in. by 8 in. cylindrical concrete specimens were tested at ages of 3, 7, 28 and 90 days after being cast.

5.2.1.2.2 Bulk Conductivity

The bulk electrical conductivity of concrete specimens also provides a rapid indication of the concrete's resistance to the penetration of chloride ions by diffusion, and is the inverse of resistivity. The test method was performed in general accordance with ASTM C1760, "Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete." The procedure includes measuring the electrical current through a saturated concrete specimen with a potential difference of 60 V dc maintained across the ends of the specimen. Three 4 in. by 8 in. cylindrical concrete specimens were tested at 28 and 90 days after being cast.

5.2.1.2.3 Rapid Chloride Permeability

The Rapid Chloride Permeability Test (RCPT) is a method of determining the electrical conductance of concrete to provide a rapid indication of its resistance to the penetration of chloride ions. The test method was performed in general accordance with ASTM C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." The procedure includes monitoring the amount of electrical current passed through 2 in. by 4 in. cylindrical concrete specimens during a six hour period. The current is supplied by 60 V dc maintained across the ends of the specimens (with voltage cells at each end of the 2 in. thick disk-shaped specimen). One end of the specimen is surrounded by a sodium chloride solution and the other end is surrounded by a sodium hydroxide solution. The total charge passed, measured in Coulombs, has been found to correlate with the resistance of concrete to chloride ion penetration (Whiting 1981). A lower charge passed indicates a higher resistance to chloride ion penetration while a higher charge passed indicates a low resistance to chloride penetration.

5.2.1.3 Shrinkage Testing

The main benefit of internally cured concrete is seen as the potential to reduce shrinkage and cracking (Bentz and Weiss 2011). Concrete shrinks due to several reasons but most of the shrinkage is the result of the chemical reaction between cement and water which causes a reduction in volume (Neville 2011). However, if the extra water available within the LWA is drawn out by capillary suction, the volume reduction of the concrete can be diminished which theoretically reduces shrinkage and the potential for cracking (Henkensiefkin et al. 2009).

Three different types of shrinkage testing were performed on both conventional and internally cured concrete and mortar specimens as part of this study. The tests included unrestrained shrinkage, restrained shrinkage, and autogenous shrinkage. In the subsequent sections of this report, each test method is explained and results are presented.

5.2.1.3.1 Unrestrained Shrinkage

A commonly utilized method of determining drying shrinkage characteristics of concrete is the unrestrained shrinkage length change test. Testing for unrestrained drying shrinkage was performed in general accordance with ASTM C157, "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete." As stated in ASTM C157, this test method correlates unrestrained length change with the potential for volumetric contraction of mortar and concrete specimens due to causes other than applied forces or temperature changes. Specimens for unrestrained shrinkage testing were three concrete beams with approximately 4 in. by 4 in. by 11 in. dimensions. At each end of the beam, a gage stud was cast in the center so that the length can be measured using a digital length comparator calibrated by a reference bar. Specimens were cured in a wet tank for 28 days after being cast. After curing, specimens were moved to a room of constant temperature and humidity conditions of 73 ± °3 F and 50 ± 4% relative humidity. Specimens were stored so that there was a clearance of at least 1 in. on all sides except for small areas where they were supported on rollers. Length measurements were made at 4, 7, 14, and 28 days and after 8, 14, and 64 weeks of air storage.

5.2.1.3.2 Restrained Shrinkage

Restrained shrinkage tests, on the other hand, are performed to determine the likelihood of early-age cracking of different concrete mixtures. These tests are evaluated by determining a time to cracking for each specimen, as well as the rate of tensile stress development. Restrained shrinkage testing for this study was performed in general accordance with ASTM C1581, “Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage.” However, the ASTM standard calls for coarse aggregate to be used of maximum nominal size of 0.5 in. or less. The maximum nominal size of aggregate used in this study was 0.75 in. in an attempt to reduce variability within this test and other tests performed. Specimens for this test are commonly referred to as “cracking rings” due to their circular geometry, which is shown below in Figure 5.2.

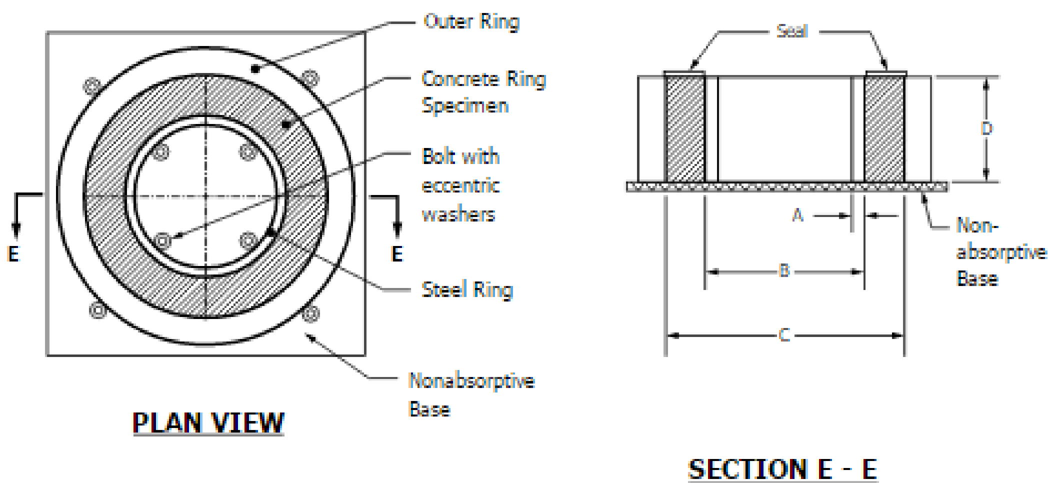


Figure Dimensions	SI Units	Inch-Pound Units
A	13 ± 1 mm	0.50 ± 0.05 in.
B	330 ± 3 mm	13.0 ± 0.12 in.
C	405 ± 3 mm	16.0 ± 0.12 in.
D	150 ± 6 mm	6.0 ± 0.25 in.

Notes:

1. Not to scale.

Figure 5.2: Testing apparatus geometry (from ASTM C1581)

On the interior of the inner steel ring, 4 strain gages were mounted at mid-height and opposite one another at 90° angles to monitor strain applied on the steel from the concrete. For each concrete mixture, three specimens were cast inside a room of constant temperature and humidity conditions of 73 ± °3 F and 50 ± 4% relative humidity. Specimens were covered in wet burlap and polyethylene sheeting for 1 day. On the following day, the wet burlap, polyethylene sheeting and outer ring were removed. Lastly, the top surface of each specimen was sealed using paraffin wax to ensure that the specimens dried from the outer circumferential surface only. Specimens were allowed to remain in place for a minimum of 28 days, with strain gage readings taken every 15 minutes by a computer-controlled data acquisition system. The specimens were periodically visually inspected for cracks.

5.2.1.3.3 Autogenous Shrinkage

Autogenous shrinkage is most significant in concrete and mortars that have a low w/c or w/cm ratio, such as the bridge deck concrete mixtures studied as part of this research (Neville 2011). Autogenous strain refers to the developed strain from a sealed specimen kept at constant temperature with no external forces from the time of first set until a certain age. Autogenous shrinkage testing was performed in general accordance with ASTM C1698, “Standard Test Method for Autogenous Strain of Cement Paste and Mortar.” This test method utilizes mortar specimens which are measured frequently for length change using a dilatometer, pictured below in Figure 5.3. The mortar specimens consist of the same proportions of materials as the concrete mixture designs described in Chapter 4, with the exceptions being the removal of the coarse aggregate and the use of no chemical admixtures. Autogenous shrinkage specimens have a geometry similar to that shown below in Figure 5.4. Three specimens were cast of each mortar mixture and stored in a room of constant temperature and humidity conditions of 73 ± °3 F and 50 ± 4% relative humidity.

Length measurements were taken at ages of 1, 4, 7, 14, and 28 days after casting. The difference in 1 day and 28 day readings was used to calculate autogenous strain per the ASTM C1698 standard.

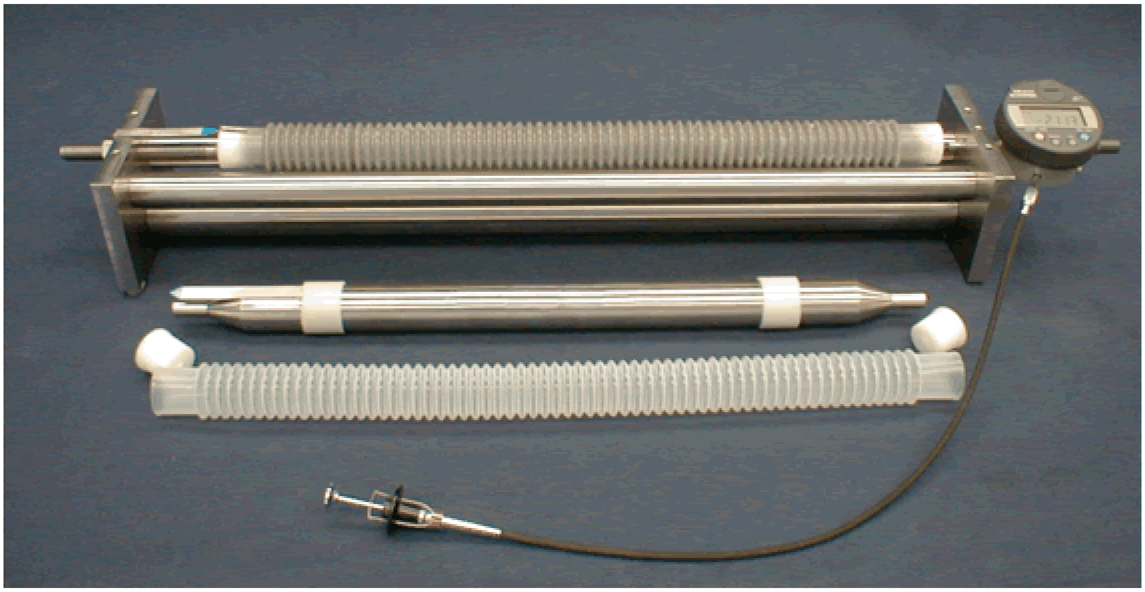


Figure 5.3: Autogenous shrinkage testing apparatus (from ASTM C1698)

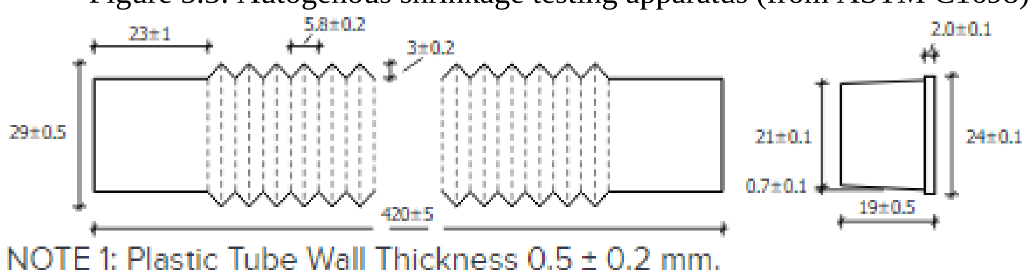


Figure 5.4: Geometry of Autogenous Shrinkage Specimens (from ASTM C1698)

5.2.2 Experimental Results

In this section, the results from testing of hardened concrete are presented. Results are presented in the same three general categories utilized in Section 5.2 of this chapter: mechanical properties, durability performance and shrinkage.

5.2.2.1 Mechanical Properties

As described in Section 5.2.1.1, tests were performed to determine the compressive strength, MOE, Poisson's ratio, and CTE of concrete mixtures included in this study. In Table 5.3, below, a summary of the results is provided, with additional analysis and discussion of these test results presented in the subsequent sections. Further information including ranges and standard deviations of test results are provided in Appendix D, Figures D.1 through D.4.

Table 5.3: Summary of mechanical property test results

Designation	**Compressive Strength (psi)**				**MOE (psi)**	**Poisson's Ratio**	**CTE (in./in./°F)**
	3 Day	**7 Day**	**28 Day**	**90 Day**			
CC	5,670	5,990	7,780	8,340	4,390,000	0.20	5.900×10^{-6}
I1M	3,910	5,460	5,250	6,320	3,190,000	0.26	5.544×10^{-6}
I2M	4,840	5,390	6,930	6,290	3,120,000	0.22	5.456×10^{-6}
I1H	4,380	4,400	5,440	5,720	3,070,000	0.21	5.222×10^{-6}
I2H	4,520	4,340	4,940	5,870	3,120,000	0.22	5.407×10^{-6}
CF	3,850	4,050	4,940	6,280	3,430,000	0.23	5.421×10^{-6}
I1MF	3,650	4,310	5,250	6,420	3,680,000	0.23	5.239×10^{-6}
I2MF	3,870	4,700	5,560	6,920	3,050,000	0.24	5.285×10^{-6}
I1HF	3,580	4,110	5,220	6,250	3,290,000	0.22	5.087×10^{-6}
I2HF	3,740	3,840	5,500	6,100	3,360,000	0.21	5.056×10^{-6}

5.2.2.1.1 Compressive Strength

As noted in Section 5.2.1.1.1, compressive strength was tested for each mixture at ages of 3, 7, 28 and 90 days and the average of three specimens are shown in Table 5.3. NCDOT specifications for Class AA concrete for bridge deck purposes require a minimum 28-day compressive strength of 4,500 psi. All ten of the bridge deck mixtures met this requirement. However, some differences in compressive strength and general trends were observed. Figure 5.5 provides a plot of compressive strength versus time for the bridge deck mixtures. Of note, the control conventional concrete mixture (CC) exhibited a notably higher compressive strength than other mixtures at all ages, as well as significantly higher MOE. To confirm these results, mixture 1 of CC was batched and tested again, and the test results for these mechanical properties almost identical. Results also showed that mixtures with fly ash showed a significant development of additional compressive strength between 28 and 90 days, which is consistent with the fact that fly ash shows hydration benefits at later ages. On the other hand, the mixtures without fly ash did not gain nearly as much compressive strength between 28 and 90 days of curing.

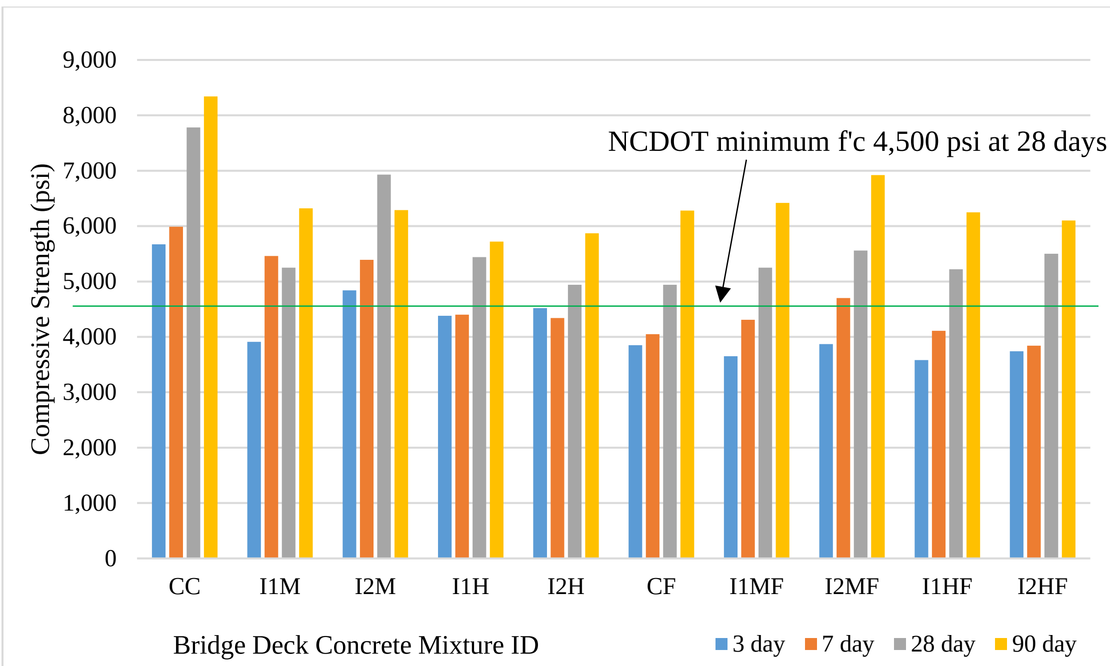


Figure 5.5: Compressive strength results of bridge deck mixtures

5.2.2.1.2 Modulus of Elasticity and Poisson's Ratio

Results of testing for the Modulus of Elasticity (MOE) and Poisson's Ratio are also provided in Table 5.3. The results of MOE were relatively consistent with a range of 3,050,000 psi to 4,390,000 psi. If the conventional concrete without fly ash mixture, which had gained reasonable strength by 28 days, is omitted, the range of MOE becomes significantly tighter at 3,050,000 psi to 3,680,000 psi for the remaining nine mixtures. Similarly, the results of Poisson's Ratio testing were fairly consistent with a range of 0.20 to 0.26. There does not seem to be a significant difference between the MOE and Poisson's ratio of the conventional concrete mixtures and the internal curing concrete mixtures. The MOE does tend to drop slightly due to the addition of LWA. This is consistent with findings of other researchers (Delatte et al. 2007, Barrett et al. 2015).

In Figure 5.6, the measured MOE of laboratory concrete mixtures from this work is compared to the MOE calculated using the equation provided in ACI 318 (Equation 5.1) that relates unit weight and compressive strength to MOE. In this equation, Ec is predicted MOE, w_c is the unit weight and f'_c is 28-day compressive strength. To compute the predicted MOE values in Figure 5.6, the measured w_c and f'_c were used.

$$E_c = w_c^{1.5} \times 33 \times f'_c{}^{1/2} \tag{5.1}$$

As can be seen in the figure, the ACI 318 prediction equation consistently calculates an MOE higher than the MOE measured using ASTM C469 for both conventional concrete and internally cured concrete. The reduction in measured MOE compared to theoretical MOE further increases with the addition of LWA for internal curing. This finding could be useful to designers planning to utilize ICC in future structures and pavements.

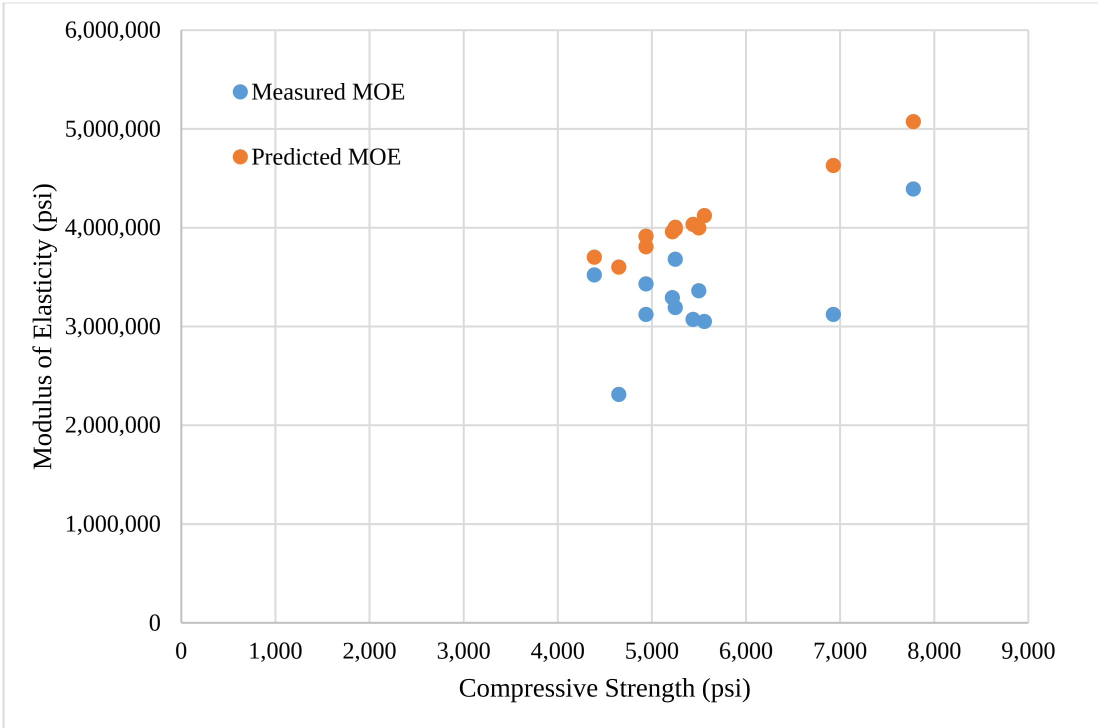


Figure 5.6: Measured MOE compared to predicted MOE calculated from ACI 318

5.2.2.1.3 Coefficient of Thermal Expansion

CTE test results are provided in Table 5.3, and a figure illustrating the differences in CTE, including range bars, is presented in Figure 5.7. As stated in Chapter 2, mortars using lightweight aggregates tend to have a lower CTE than mortars using normalweight aggregate (Maruyama and Teramoto 2012). The results from this study confirm this, as the conventional concrete mixture without fly ash had a CTE value of 5.900×10^{-6} which was significantly higher than the internally cured mixture with the next highest CTE of 5.544×10^{-6} (I1M). Additionally, the CTE was further decreased in concrete mixtures that utilized a high replacement of LWA for normalweight fines. The reduction in CTE ranged from approximately 3 to 11% in internally cured mixtures compared to conventional curing. It is unclear without further study whether the CTE was influenced by internal curing or more influenced by the presence of LWA. The presence of fly ash also appears to be linked to slightly reduced CTE values of these mixtures. This is perhaps due to the smaller size of fly ash particles compared to cement particles which provide more nucleation sites for the hydration of cement (Hemalatha and Ramaswamy 2017).

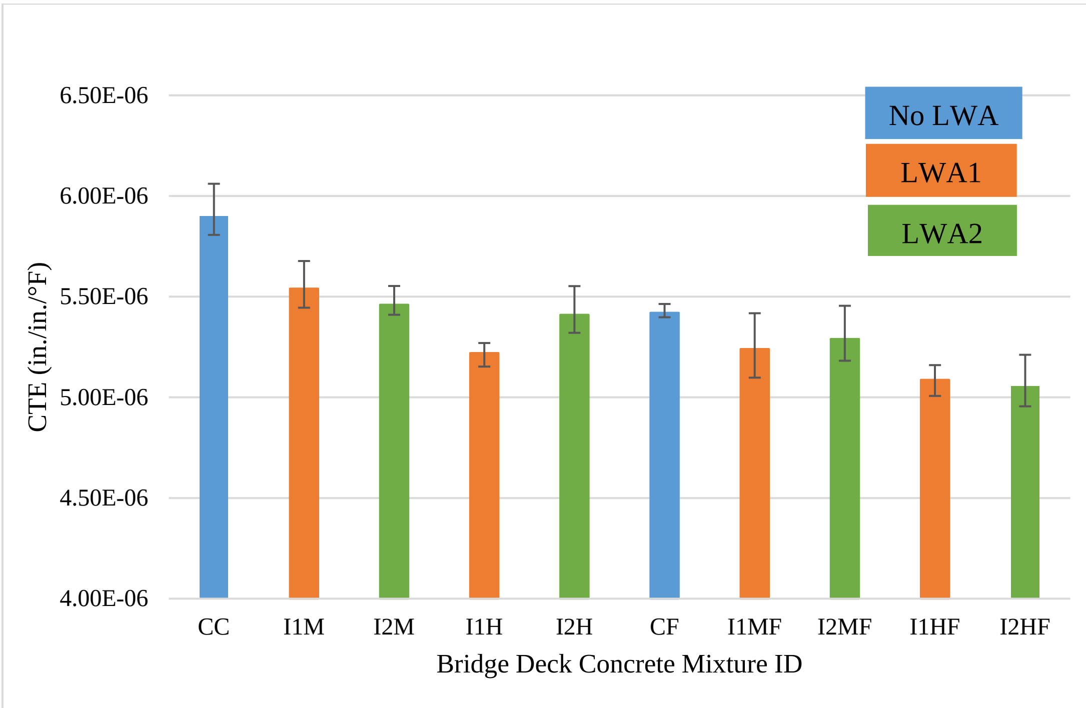


Figure 5.7: Coefficient of thermal expansion (CTE) results

5.2.2.2 Durability Performance Tests

As described in Section 5.2.1.2, durability performance tests included surface resistivity at 3, 7, 28 and 90 days, bulk conductivity at 28 and 90 days and rapid chloride permeability testing (RCPT) at 28 and 90 days. Results of these durability performance tests are summarized below in Table 5.4. As detailed in Chapter 2, the results of these three testing methods for durability performance often correlate well with each other. Additional analysis and discussion of the durability performance results are presented in subsequent sections.

Table 5.4: Summary of durability performance test results

Designation	**Surface Resistivity (kΩ-cm)**				**Bulk Conductivity (S×m/m)**		**RCPT (Coulombs)**	
	3 Day	**7 Day**	**28 Day**	**90 Day**	**28 Day**	**90 Day**	**28 Day**	**90 Day**
CC	13.3	12.7	16.3	17.9	9.93	6.09	2,930	2,190
I1M	9.9	10.6	13.7	14.7	13.92	6.28	3,380	2,620
I2M	9.3	10.0	13.3	16.7	12.54	9.12	3,180	2,550
I1H	9.3	10.7	13.6	14.7	10.22	6.22	3,220	2,890
I2H	8.1	9.1	12.5	14.4	11.55	5.23	2,920	2,790
CF	10.4	11.1	18.1	46.1	9.93	3.68	2,440	590
I1MF	8.8	10.3	17.5	45.7	9.50	3.33	2,140	590
I2MF	7.4	8.1	17.0	36.9	9.26	4.12	2,710	960
I1HF	7.1	9.2	17.0	43.3	11.64	3.59	2,470	810
I2HF	6.9	7.6	16.5	43.6	10.14	3.41	2,320	750

5.2.2.2.1 Surface Resistivity

Surface resistivity test results are provided in Table 5.4, and are graphically summarized below in Figure 5.8. A higher surface resistivity indicates that the concrete has a better resistance to electrical current, and is less penetrable to aggressive agents such as chlorides. Therefore, concrete with a high surface resistivity is predicted to have better durability performance.

Results show that the conventional concrete mixture for this study possessed a slightly higher surface resistivity than the internal curing concrete at each of the testing ages. However, the slight difference could likely be attributed to the extra water contained in the LWA within the internal curing mixtures, since water is known to conduct electricity. The results also show that the addition of fly ash has significant benefits on increased surface resistivity at 28 and 90 days. All concrete mixtures tested in this study, both conventional and internal curing, fell into an AASHTO TP 95-11 (Table 1) permeability classification of moderate at 28 days as seen in Figure 5.8. At 90 days, the mixtures that did not contain fly ash remained in the moderate permeability range while those mixes with fly ash reached a permeability classification of very low.

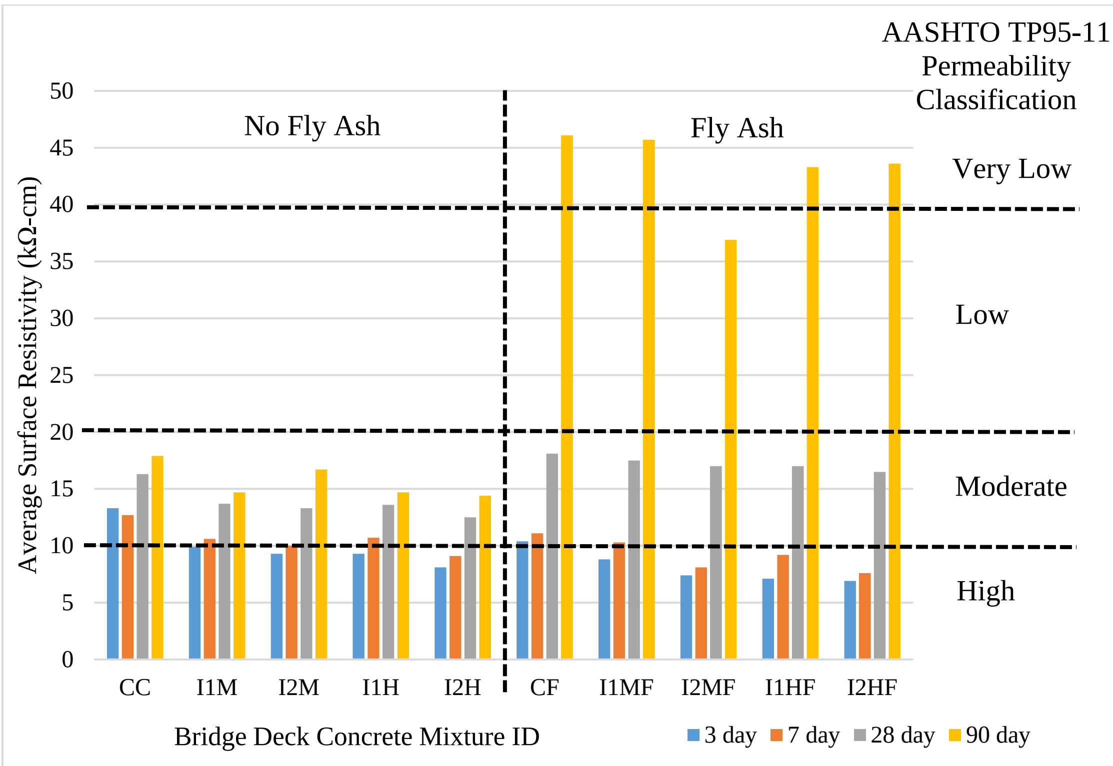


Figure 5.8: Average surface resistivity test results

5.2.2.2.2 Bulk Conductivity

Bulk conductivity results are provided in Table 5.4, with a graphical summary presented in Figure 5.9. The average of three specimens is plotted at each age. Bulk conductivity is the opposite of resistivity; it is a measure of the concrete's ability to conduct current rather than resist it. Therefore, a lower bulk conductivity indicates the concrete does not conduct electricity very well. Concrete that does not conduct electricity well tends to have a higher resistance to electricity as well as a higher resistance to permeability.

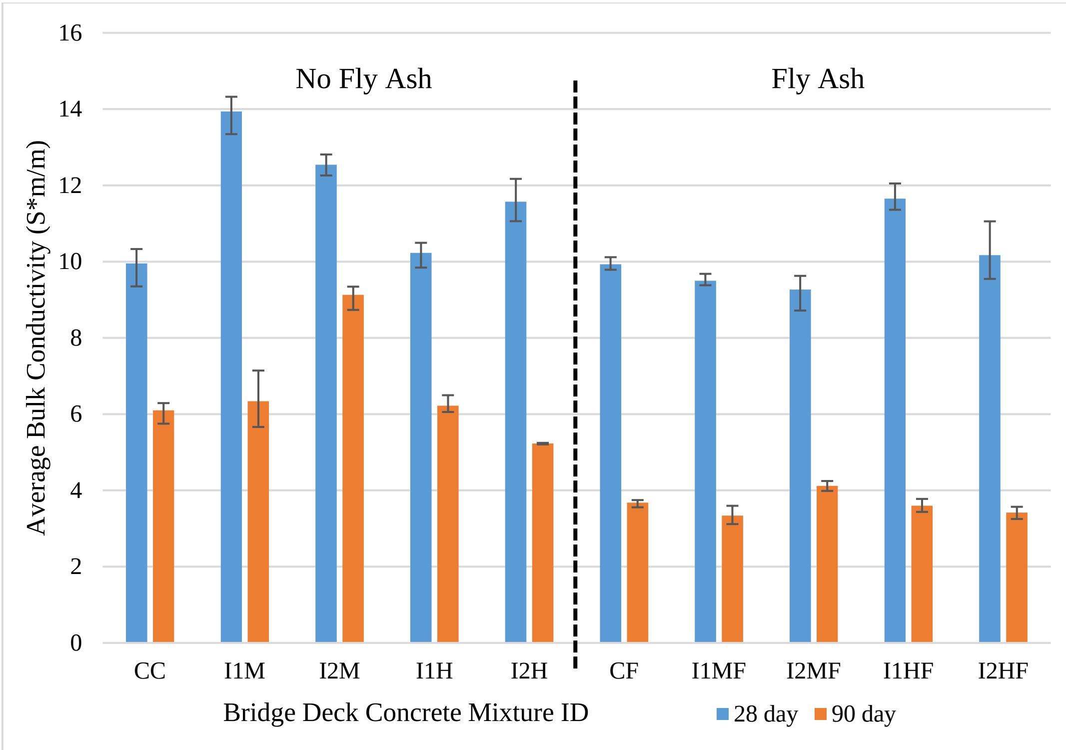


Figure 5.9: Average bulk conductivity test results

The test results show that bulk conductivity did not significantly change due to incorporation of prewetted LWA to promote internal curing. Similar to surface resistivity results, several of the internal curing mixtures possessed slightly higher bulk conductivity results, likely due to the extra water located within the pores of the LWA conducting electricity. The results at 28 days were fairly constant for all ten base mixtures with a range of 9.27 mS/m to 13.94 mS/m. At 90 days, the concrete mixtures that contained fly ash possessed significantly lower bulk conductivity (3.34 mS/m to 4.12 mS/m) than those mixtures that did not contain fly ash (5.23 mS/m to 9.13 mS/m) as seen below in Figure 5.7. According to ASTM C1760, a bulk conductivity range of 3 mS/m to 20 mS/m correlates with an RCPT (ASTM C1202) range of 500 to 4000 Coulombs passed. Rapid chloride penetration testing (RCPT) results, which compare well with both surface resistivity and bulk conductivity results, are discussed in the subsequent section.

5.2.2.2.3 Rapid Chloride Permeability Test

RCPT results are provided in Table 5.4, with a graphical summary presented below in Figure 5.10. The rapid chloride permeability test is a measure of charge passed (measured in Coulombs) between two sides of a 2 in. thick concrete specimen. As discussed previously in Section 5.2.1.2.3 for each concrete mixture, three specimens were tested and the total Coulombs passed was averaged. This occurred at both 28 days and 90 days with different test specimens cut from designated cylinders. The results indicate no significant difference in rapid chloride permeability between conventionally cured concrete mixtures and internal curing concrete mixtures included in this study. However, the inclusion of fly ash in both conventionally and internally cured concrete mixtures appears to result in a significant increase in resistance to chloride ion penetration (lower charged passed) at 90 days of age. Figure 5.10 provides an analysis of RCPT results and the corresponding permeability classifications, and it can be seen that RCPT test results correlate well with the results of surface resistivity and bulk conductivity testing.

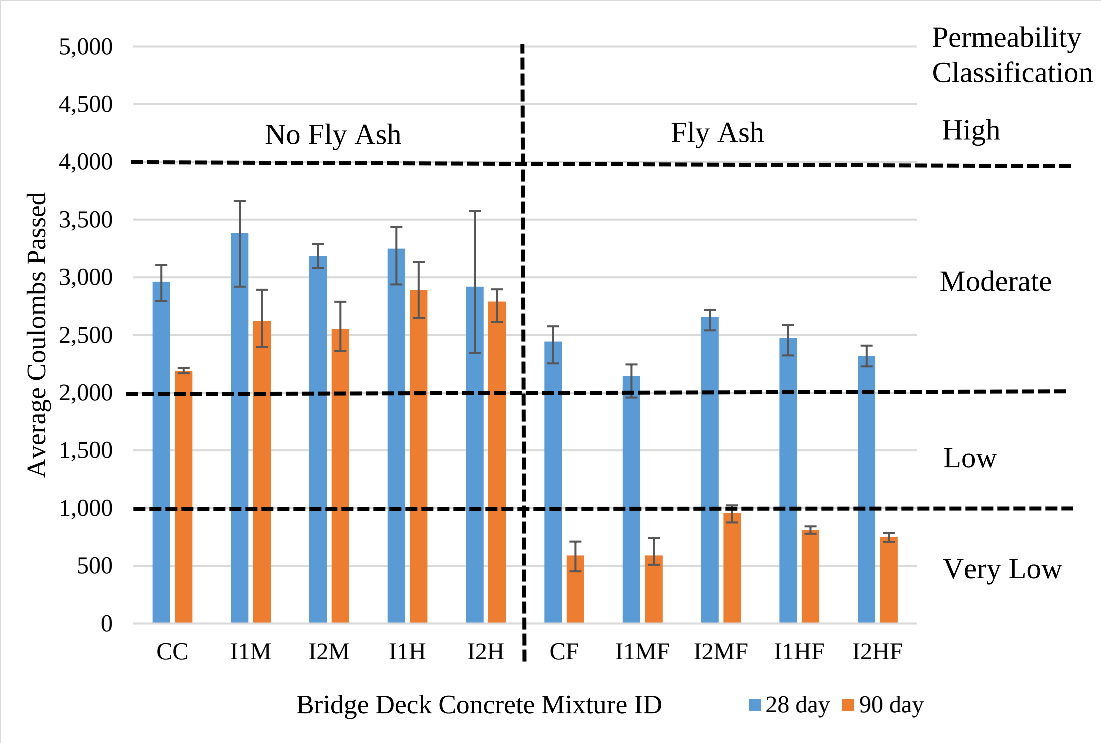


Figure 5.10: Average rapid chloride permeability test results

All mixtures were within the range of 2,000 to 4,000 Coulombs passed at 28 days which is classified as moderate permeability. The average charge passed decreased for all specimens between 28 and 90 days. Concrete mixtures that did not contain fly ash remained in the moderate permeability classification at 90 days while mixtures with fly ash fell into the very low permeability classification (100-1000 Coulombs passed). In Figure 5.11, a direct comparison of RCPT and bulk conductivity testing results is shown. As stated previously, 500 Coulombs passed in RCPT equates to a bulk conductivity of 3 mS/m and 4000 Coulombs passed equates to a bulk conductivity of 20 mS/m. The results of this study show a strong correlation between the two tests with those ranges. It seems clear that the presence or absence of fly ash is driving the change in durability performance rather than type or quantity (moderate or high) of LWA.

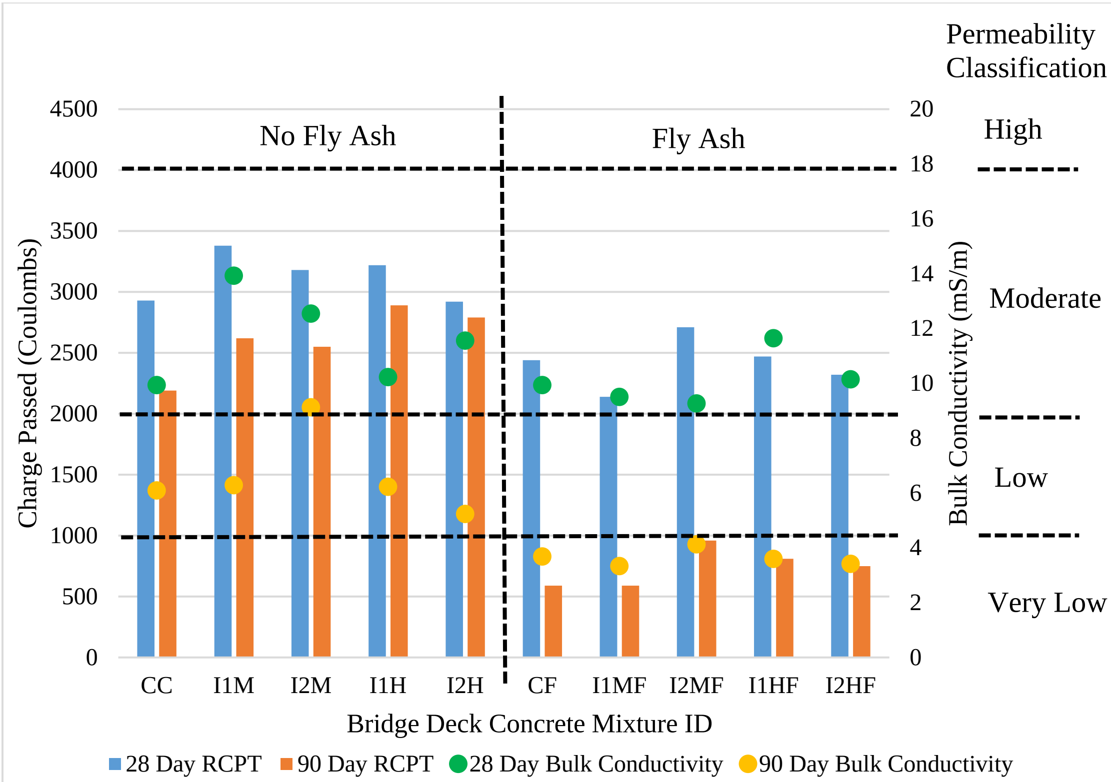


Figure 5.11: RCPT and bulk conductivity comparison

RCPT values from this work were compared to surface resistivity results obtained by Rupnow and Icenogle (2011), who found a strong correlation between these two tests. Results from Rupnow and Icenogle (2011) included a power curve relationship between RCPT and surface resistivity, shown in Figure 5.12. Plotting results of testing from this work against the relationship proposed by Rupnow and Icenogle (2011) (Figure 5.12) shows a similar trend, although surface resistivity readings between 10 and 20 kΩ-cm tended to be associated with RCPT values higher than that predicted by the Rupnow and Icenogle model. This finding is likely due to the difference in materials (coarse and fine aggregate) and mixture proportions used in this study compared to those used in the Rupnow and Icenogle study.

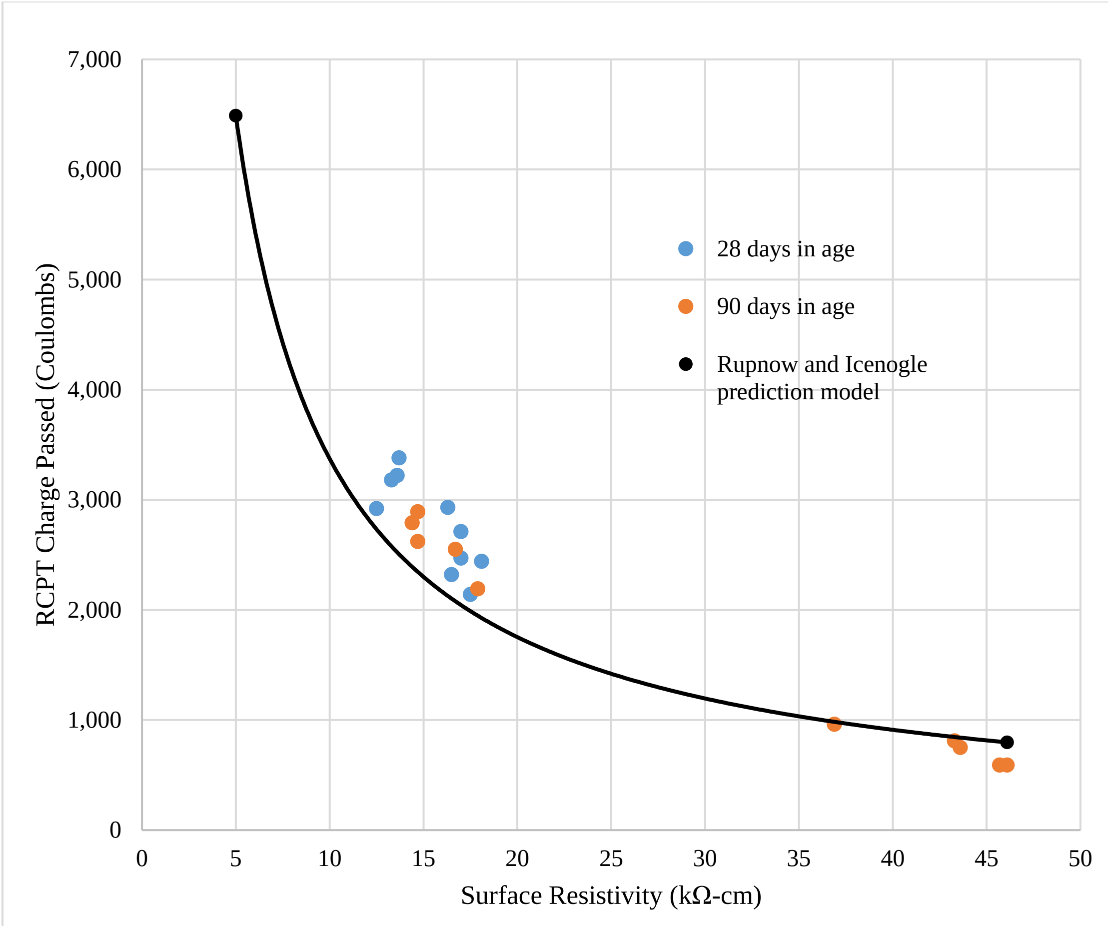


Figure 5.12: Rupnow and Icenogle prediction model (2011)

5.2.2.3 Shrinkage Testing

The results of shrinkage testing are discussed in the subsequent sections. In Table 5.5, below, a summary of the results is provided. Discussion and analysis of results from each of the three shrinkage tests are presented in the subsequent sections.

Table 5.5: Summary of average shrinkage test results

Designation	Unrestrained Drying Shrinkage After 28 Days of Wet Cure (% Change in Length)				Restrained Shrinkage Stress Rate (psi/day)	Autogenous Strain (μm/m)
	14 Day	**28 Day**	**56 Day**	**112 Day**	**28 Day**	**28 Day**
CC	0.0215	0.0318	0.0409	0.0509	12.14	433.6
I1M	0.0227	0.0327	0.0430	0.0561	10.22	263.8
I2M	0.0179	0.0315	0.0442	0.0539	8.53	272.8
I1H	0.0252	0.0333	0.0430	0.0479	10.04	298.9
I2H	0.0173	0.0294	0.0442	0.0579	4.64	173.3
CF	0.0248	0.0348	0.0427	0.0539	7.92	384.7
I1MF	0.0197	0.0282	0.0382	0.0476	8.56	261.5
I2MF	0.0185	0.0279	0.0400	0.0445	7.50	227.1
I1HF	0.0255	0.0291	0.0403	0.0488	6.08	271.5
I2HF*	0.0158	0.0255	0.0382	0.0576	***	185.2

***Restrained shrinkage results of this mixture were not complete by time of publication

5.2.2.3.1 Unrestrained Shrinkage

Unrestrained drying shrinkage results are presented in Table 5.5, with a graphical summary provided in Figure 5.13. Results are provided for the average of three specimens at ages of 14, 28, 56 and 112 days of drying. ASTM C157 also calls for measurements to be taken at 224 and 448 days (32 and 64 weeks, respectively) of drying shrinkage which will occur after the completion of this thesis.

The results do not show a significant difference in unrestrained drying shrinkage between conventionally cured concrete and internal curing concrete. Some internal curing mixtures performed slightly better than the conventional curing mixtures (showing relatively smaller lengths changes than the control mixtures) and some performed slightly worse (showing length changes higher than the control mixtures). As discussed in the literature, internal curing may not have an effect on unrestrained drying shrinkage and in some instances may increase drying shrinkage (Ardeshirilajimi et al. 2016). Additionally, the specimen curing protocol prescribed in ASTM C157 includes 28-day wet curing, which likely eliminates the potential for the test measurements to reflect the advantages in early-age moisture delivery provided by ICC. The results of this study corroborate the findings of the previous study cited. Mokarem et al. (2004) reported that the change in length due to drying shrinkage should be less than 0.04% at 28 days and 0.05% at 90 days to reduce the probability of cracking. All ten bridge deck mixtures in this study had unrestrained shrinkage test results that fell below these thresholds. Furthermore, AASHTO MP XX-17, “Standard Specification with Commentary for Performance Engineered Concrete Pavement Mixtures” states a prescriptive limit of 420 με at 28 days of drying for pavement applications. The ten bridge deck mixtures range from 255 to 348 με at 28 days which, although a pavement specification, passes the guidance presented in this draft specification.

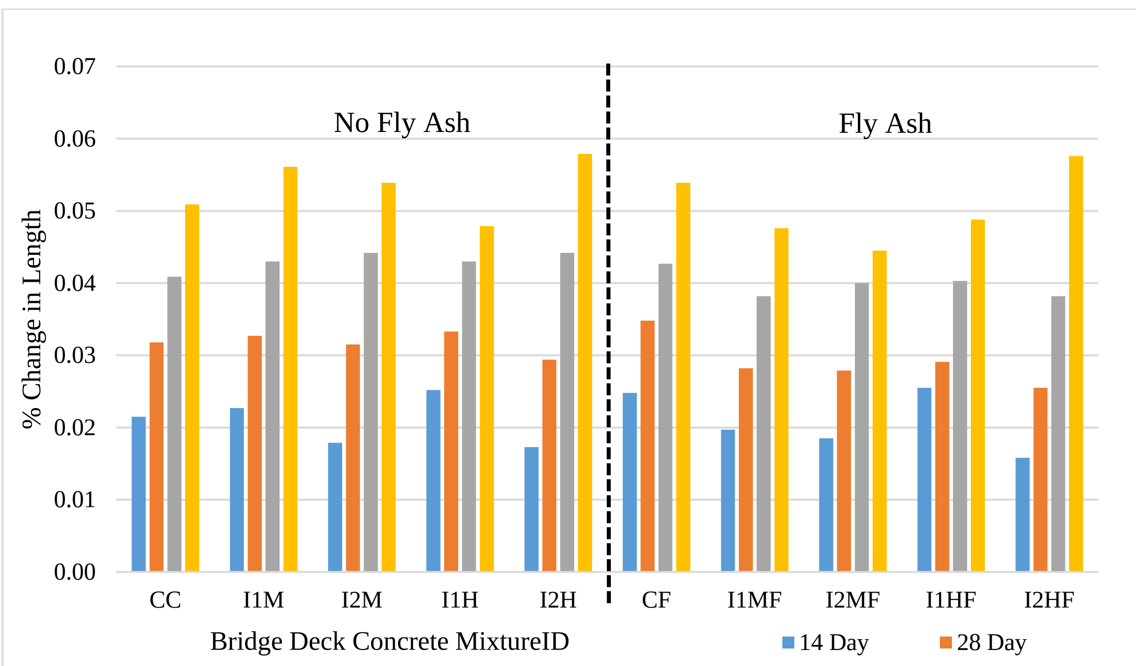


Figure 5.13: Average unrestrained drying shrinkage after 28 days of curing

5.2.2.3.2 Restrained Shrinkage

Restrained drying shrinkage test results are summarized in Table 5.5. Additional description of treatment of the test data, as well as analysis and discussion of the results is presented in this section. As described in Section 5.1.2.3.2, three specimens of each concrete mixture were prepared and tested for a period of 28 days. The average stress rate (psi/day) of the three specimens is presented in Table 5.4. Each specimen testing apparatus had four strain gauges, and strain data for each specimen was plotted against time. An example of this plot is provided in Figure 5.14 (for Mixture CC3, Specimen 2), with plots for all specimens presented in Appendix D, in Figures D.1 through D.54. The time of cracking of the specimen was determined when a sudden decrease in strain was noticed and recorded to the nearest 0.25 days.

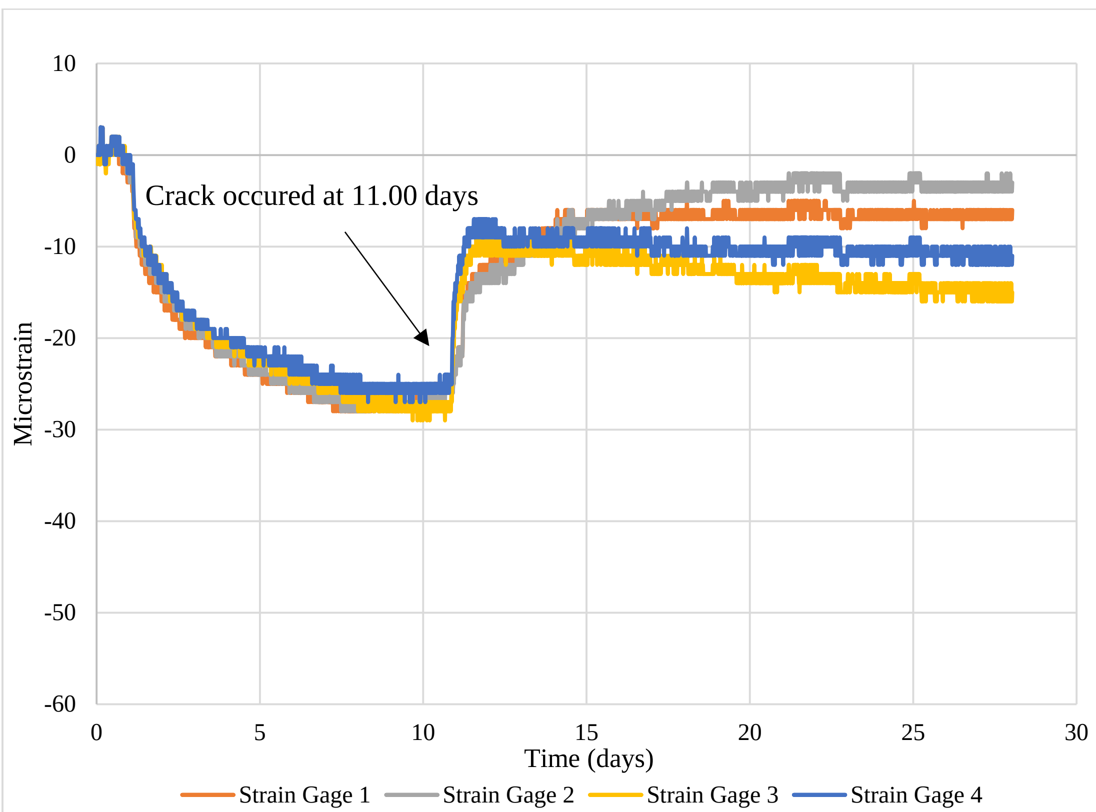


Figure 5.14: Example plot of microstrain vs. time for a typical test specimen

Following ASTM C1581 procedures, the strain was then plotted against the square root of time from approximately the time of first set to the time of cracking. According to the ASTM standard, the square root function has been found to consistently provide a good fit to the test data. Linear regression was then performed for each strain gage. An example of this plot (for Mixture CC3, Specimen 2) is provided in Figure 5.15. Plots for all specimens are presented in Appendix D, in Figures D.1 through D.54.

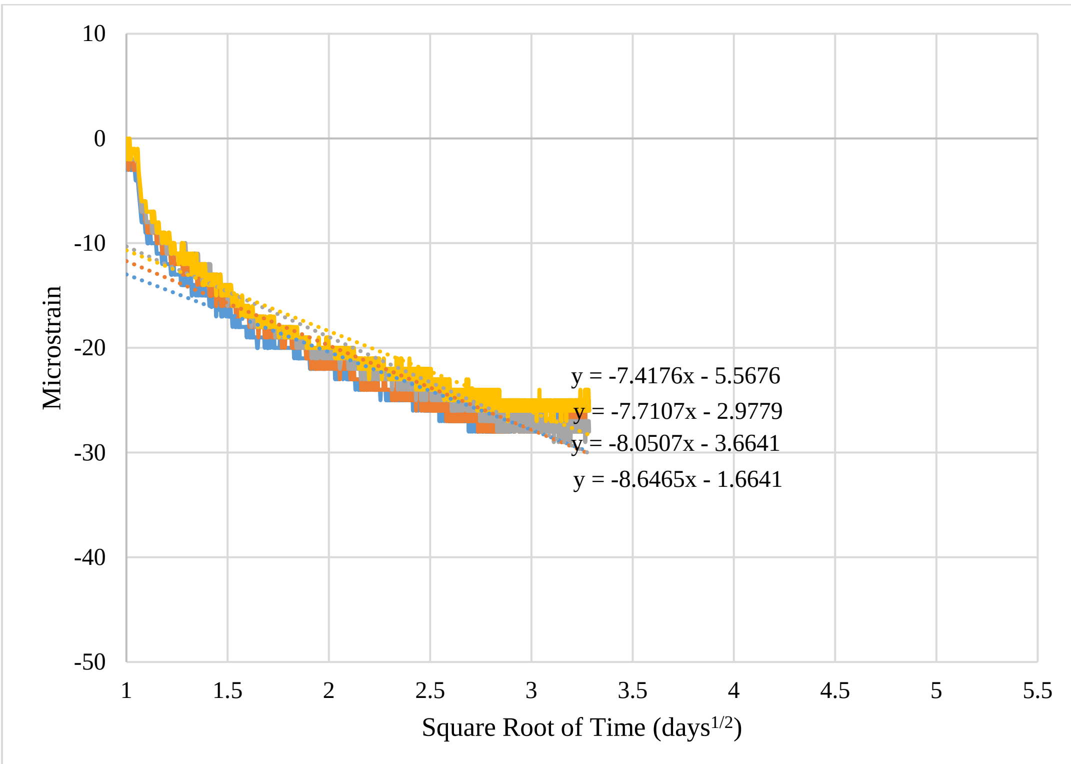


Figure 5.15: Example plot of strain vs. square root of time and linear regression

Per ASTM C1581, the slope of each linear regression is equal to the strain rate factor for each strain gage on the test specimen measured in (in./in.)/day$^{1/2}$. These four values are averaged to determine the average strain rate factor of each specimen. The stress rate of the specimen, q, is then determined by Equation 5.2, a function of average strain rate, a_{avg}, geometry of the concrete and steel ring, G, and time to cracking, t_r.

$$q = G\,|a_{avg}|\,/\,2\sqrt{t_r} \tag{5.2}$$

The three stress rates are averaged to determine an average stress rate for each concrete mixture, measured in psi/day. Plots of strain vs. time and strain vs. the square root of time and linear regression for each specimen are presented in the Appendix. A summary of these results are provided in Table 5.6 and Figure 5.16.

Table 5.6: Results of ASTM C1581 testing

Designation	**ASTM C1581: Average Stress Rate (psi/day)**			
	Ring 1	**Ring 2**	**Ring 3**	**Average**
CC	11.22	12.56	12.63	12.14
I1M	9.81	8.76	12.08	10.22
I2M	8.70	12.59	4.29	8.53
I1H	11.08	9.6	9.44	10.04
I2H	3.14	5.86	4.92	4.64
CF	12.34	4.12	7.31	7.92
I1MF	9.77	10.25	5.65	8.56
I2MF	5.66	10.04	6.80	7.50
I1HF	7.7	4.57	5.96	6.08

Early crack (≤ 14 days)
Late crack (14 - 28 days)
Did not crack w/in 28 days

***Testing of I2HF, IP, CL and IL was not complete at time of publication

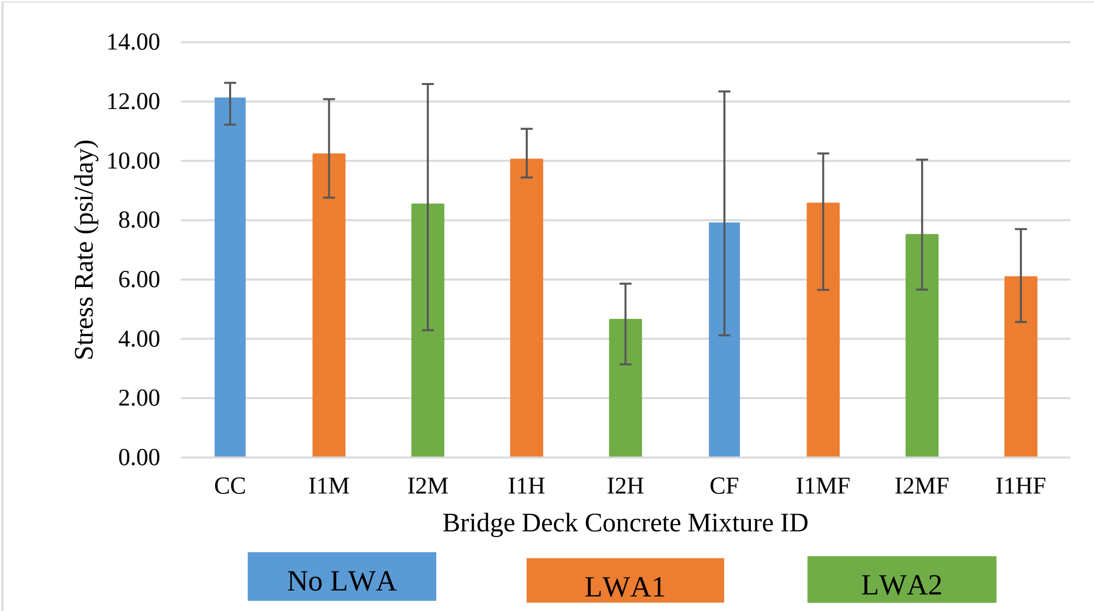


Figure 5.16: Restrained shrinkage average stress rate results

In general, the time to cracking of conventional mixes was higher than internally cured mixes. The time to cracking ranged from cracking at 10 days to no cracking at 28 days. Testing was concluded at 28 days per ASTM C1581. The results show that the conventional concrete mixture without fly ash developed the highest average stress rate. The conventional concrete mixture with fly ash shows mixed results with stress rates varying from 4.12 to 12.34 psi/day. The internal curing mixtures did tend to develop lower average stress rates than the conventional concrete mixtures with the exception of a few specimens. Specimens that did not crack tended to exhibit a significantly lower stress rate than those specimens that did crack. The relatively wide range in variability (shown in Figure 5.14 using range bars) makes further quantitative assessment of these results challenging. For example, the conventional concrete mixture with fly ash exhibited the widest range of performance between individual specimens. One specimen cracked within 14 days, another cracked between 14 and 28 days and another exhibited no cracking. This leads to a wide range of stress rates and corresponding conclusions. The results show that the addition of fly ash may provide a lower stress rate and a better resistance to cracking, similar to the use of internal curing. However, due to the variability in the fly ash control mixture (CF), further testing is recommended.

5.2.2.3.3 Autogenous Shrinkage

Autogenous shrinkage test results are summarized in Table 5.5, with an additional graphical analysis shown in Figure 5.17. Three specimens were prepared from each concrete mixture and the results were averaged. The results indicate favorable benefits of internal curing, with significantly less autogenous strain developed in the internal curing mixtures than in in the conventional mixtures. Compared to conventionally cured mortars, LWA1 at a moderate and high replacement reduced autogenous shrinkage by 36% and 30%, respectively. On the other hand, LWA2 reduced autogenous shrinkage by 39% and 56%, at moderate and high replacement rates respectively. Barrett, Miller et al. (2015) report up to 80% reduction in autogenous shrinkage for internal curing compared to non-internal curing in Indiana. Ardeshirilajimi et al. (2016) reported autogenous shrinkage reductions in Illinois concrete mixtures ranging from 30 - 50%. These reductions were measured in mixtures utilizing a variety of w/c ratios with multiple LWAs indicating autogenous shrinkage reductions correlate with amount of internal curing water available (Ardeshirilajimi et al. 2016). The results of this thesis corroborate with the findings of researchers in Indiana and Illinois (Barrett, Miller et al. 2015, Ardeshirilajimi et al. 2016).

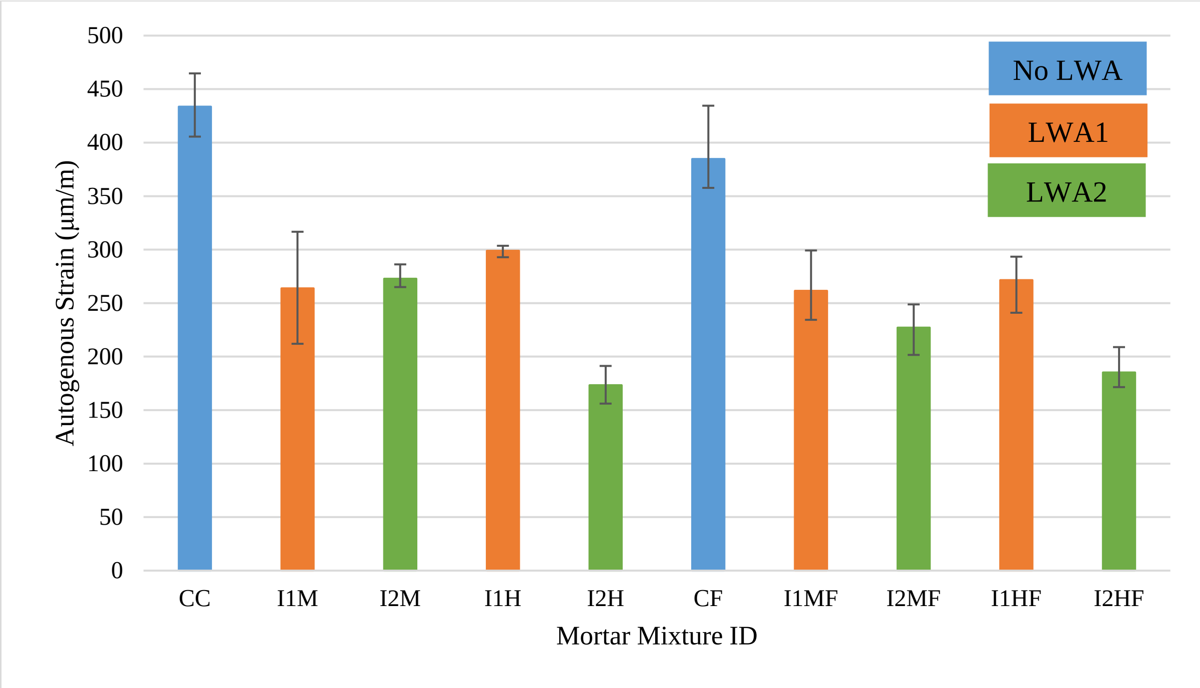


Figure 5.17: Average autogenous strain results

The results of shrinkage testing on concrete and mortar mixtures show no correlation between unrestrained drying shrinkage and restrained drying shrinkage or autogenous shrinkage. However, a correlation seems to exist between restrained drying shrinkage and autogenous shrinkage. Figure 5.18 presents a graphical illustration of this correlation. The stress rate of the restrained drying shrinkage specimens corresponds with the y-axis on the left (bar graph) while the autogenous strain developed corresponds with the y-axis on the right (markers).

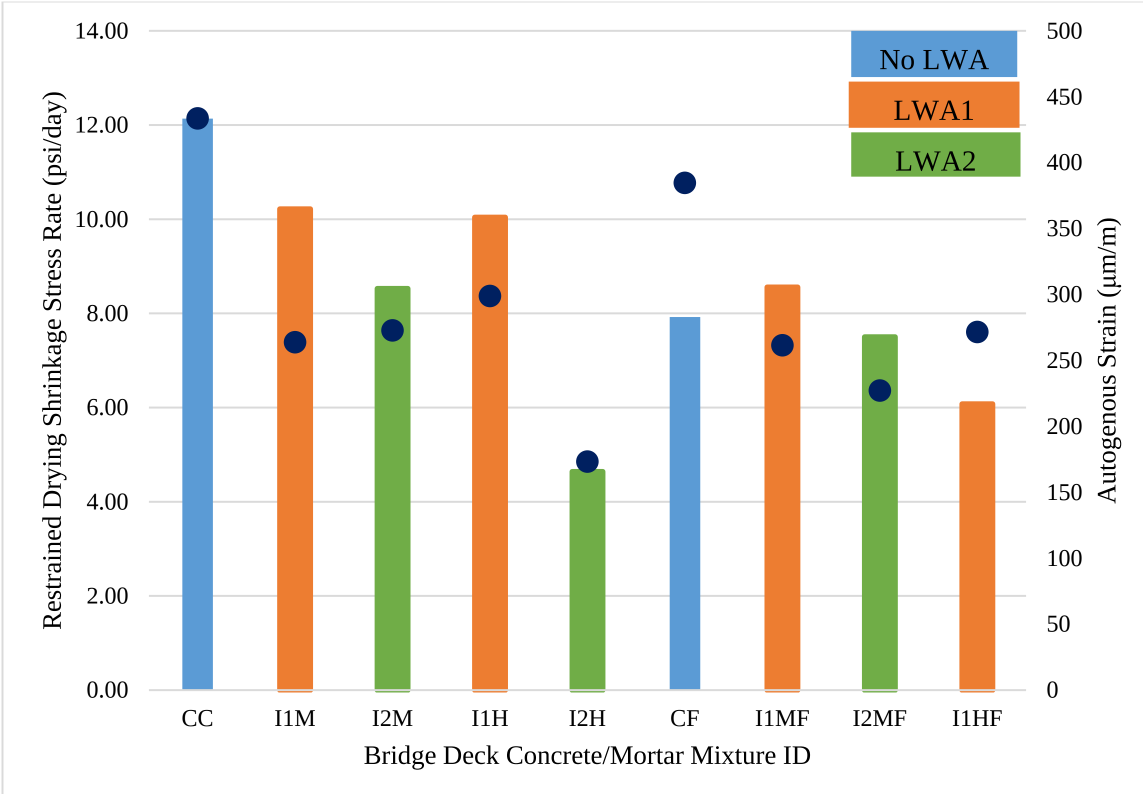


Figure 5.18: Restrained drying shrinkage compared with autogenous strain

5.3 Testing of Pavement Mixtures and Latex-Modified Concrete

In the previous section, results of testing of the ten bridge deck mixtures is presented. In this section, the results of testing of pavement mixtures and LMC mixtures is presented, since not directly comparable to the bridge deck concrete mixtures. The materials and mixture proportions of the internally cured pavement mixture IP (shown in Figure 4.1) correspond to a control pavement mixture from another study (NCDOT RP 2015-03, mixture P.B.N.N.), and has significantly different mixture proportions than the bridge deck mixtures discussed previously in this chapter. As a reminder to the reader, the pavement mixture contains a significantly lower cement content (573 pcy compared to 715 pcy) and a higher w/c ratio (0.48 compared to 0.35). Internally cured pavement mixture IP utilized the same coarse aggregate, normalweight fine aggregate and cement source as mixture P.B.N.N from NCDOT RP 2015-03 (Blanchard 2016, Chimmula 2016, Medlin 2016). Prewetted LWA1 was substituted at a 20% volumetric replacement for normalweight fine aggregate for the internal curing pavement mixture.

Similarly, the conventional LMC mixture and internally cured LMC mixture also differ from the concrete bridge deck mixtures. Latex-modified mixtures are typically used as concrete overlays with maximum thickness of 2 inches. Therefore #78 coarse aggregate (0.5” maximum size) was used in place of #67 (0.75” maximum size). This aggregate was also a granitic gneiss from the piedmont region of the state. The mixture design for the conventional mixture was taken from the NCDOT recommendation of frequent latex-modified concrete mixtures used in the state. Class I/II cement was used, (not the rapid set cement used in very early strength mixtures). Prewetted LWA1 was used in the internal curing latex-modified concrete mixture at a volumetric replacement of 35% for normalweight fine aggregate.

5.3.1 Experimental Procedures for Tests of Pavement Concrete Mixtures

Testing of the internal curing pavement mixture (referred to as IP) included similar testing performed on the bridge deck concrete mixtures and pavement mixtures from NCDOT RP 2015-03. Mechanical properties that were tested included compressive strength (3, 7, 28 and 90 days), Modulus of Elasticity, Poisson’s ratio and coefficient of thermal expansion all at 28 days of age. Durability performance testing included surface resistivity (3, 7, 28, and 90 days) and RCPT at both 28 and 90 days. Unrestrained drying shrinkage was also tested. These test results are compared to test results of P.B.N.N. from NCDOT RP 2015-03 in the subsequent section. Additional details on the control pavement mixture, along with all test results for this mixture, are presented in other publications (Blanchard 2016, Medlin 2016, Chimmula 2016).

5.3.2 Experimental Results for Tests of Pavement Concrete Mixtures

Results from testing of the internally cured pavement mixture are provided in Table 5.7 along with the results of the companion control mixture from NCDOT RP 2015-03 (Blanchard 2016). Data presented in this table are the averages of the multiple replicates of each specimen (same number of test replicates as described in Sections 5.1 and 5.2). As the desire of this study was to directly compare the two mixtures, the cement content and w/c ratio were kept constant. For both the internally cured and conventional pavement mixtures, an air entraining admixture was used to produce an entrained air content of 5.5 ± 0.5%. However, it is noted that even without the use of water-reducing admixtures, the slump for the internally cured mixture was 7.5 inches which exceeds specifications for paving mixtures. This indicates that if internal curing is utilized for paving mixtures, the w/c could be potentially be lowered, offering additional durability benefits.

The results of hardened concrete tests show slightly higher early age compressive strength for IP1 compared to the non-internally cured control mixture P.B.N.N. The results also show a significant reduction in MOE and a slight increase in Poisson’s ratio for the internally cured mixture. Lower MOE values have been shown to be associated with reduced cracking potential (Rao and Darter 2013).

Table 5.7: Test results for pavement concrete mixtures

Test	**Age (days)**	**P.B.N.N. (control mixture)**	**IP (internally cured mixture)**
Compressive Strength (psi)	3	3,010	3,360
	7	3,420	3,970
	28	4,390	4,650
	90	5,450	5,070
Surface Resistivity (kΩ-cm)	3	8.0	5.3
	7	8.7	6.4
	28	10.7	8.2
	90	11.0	9.1
MOE (psi)	28	3,515,000	2,310,000
Poisson's Ratio	28	0.19	0.21
CTE (in./in./°F)	28	5.309×10^{-6}	5.008×10^{-6}
RCPT (Coulombs)	28	4,390	6,190
	90	3,230	2,800
Unrestrained Drying shrinkage (% Change in Length)	14	0.0109	0.0315
	28	0.0182	0.0388
	56	0.0245	0.0500

The internally cured pavement mixture did tend to develop more compressive strength than P.B.N.N as well as a higher Poisson's ratio. The MOE was reduced as well, likely due to the addition of LWA. The durability performance results were similar to those seen in the bridge deck mixtures. The average surface resistivity was slightly lower in IP at all ages compared to the conventionally cured mixture. This is likely because of the extra water contained within the LWA which better conducts electricity. The average 28 day RCPT value was significantly higher in IP than P.B.N.N. at 28 days but was slightly lower at 90 days. The reason for this may be associated with additional hydration facilitated by internal curing, but this cannot be confirmed without additional study. However, the permeability classification of both concrete mixtures is high at 28 days and moderate at 90 days, likely influenced by the relatively high w/cm used for these mixtures. As shown in

Blanchard (2015) and Medlin (2015), these pavement mixtures would show additional durability benefits from the use of fly ash.

Thermal properties for concrete utilized as MEPDG inputs include CTE, thermal conductivity and heat capacity (Blanchard 2016). The average CTE value of IP was significantly lower than the CTE of P.B.N.N, which is favorable for pavement performance (Tanesi et al. 2007). This is likely due to the LWA which tends to resist thermal shrinkage better than natural aggregate (Rao and Darter 2013). The internally cured pavement mixture did not perform as well as the conventional pavement mixture in unrestrained drying shrinkage conditions, with the control concrete mixture P.B.N.N exhibiting roughly half the length reduction as the internal curing pavement mixture. Other researchers have found that internal curing is most beneficial in concrete mixtures with high cement contents and low w/c ratios (Bentz and Weiss 2011). The pavement mixtures utilized in this study possessed the opposite: a high w/c and low cement content, which may explain the findings presented above.

5.3.3 Latex-Modified Concrete Experimental Procedures

Testing of the latex-modified mixtures was performed using the test methods used on the bridge deck concrete mixtures and pavement mixtures described previously. Mechanical properties that were tested included compressive strength (3, 7, 28 and 90 days), MOE, Poisson’s ratio and CTE all at 28 days of age. Durability performance testing included surface resistivity (3, 7, 28, and 90 days), bulk conductivity and RCPT both at 28 and 90 days. Unrestrained drying shrinkage tests were also performed.

Of note, it was found that some durability performance tests, particularly the electrical methods of surface resistivity and bulk conductivity, resulted in unstable readings for LMC mixtures. This could be possibly be explained by the effect of the polymeric latex on the concrete binder matrix. Despite difficulties obtaining stable readings of the LMC with the surface resistivity meter and bulk conductivity test apparatus, the RCPT test results appear to be reliable.

5.3.4 Latex-Modified Concrete Experimental Results

A summary of test results for the control and internally cured LMC mixtures is provided below in Table 5.8. Based on project goals, the w/cm was maintained at 0.40 which is typically the value used in most LMC mixtures submitted to NCDOT for approval. However, it is noted that at this w/cm, the measured slump did not meet NCDOT specifications and the concrete would be rejected in field conditions. However, the results do show that regardless of cement type, LMC does gain strength rapidly compared to other concrete mixtures in this study and provided mechanical properties that would meet NCDOT specifications for LMC of 3,000 psi compressive strength at 7 days (NCDOT 2012).

Table 5.8: Average results of LMC testing

	Age (days)	CL1	IL1
Compressive Strength (psi)	3	4,050	3,610
	7	4,710	4,500
	28	7,230	5,220
	90	***	***
Surface Resistivity (kΩ-cm)	3	18.7	12.6
	7	22.0	16.7
	28	N/A	N/A
	90	***	***
MOE (psi)	28	3,651,000	3,801,000
Poisson's Ratio	28	0.22	0.24
CTE (in./in./°F)	28	6.726×10^{-6}	5.963×10^{-6}
RCPT (Coulombs)	28	2,170	3,250
	90	***	***
Unrestrained Drying shrinkage (% Change in Length)	14	0.0297	0.0155
	28	0.0321	0.0261
	56	***	***

***Testing ages will be reached after submission of this thesis

The MOE and Poisson's ratio for both the internally cured and control LMC are comparable with other conventional and internally cured mixtures tested during this study. RCPT results at 28 days reasonably correlate with other concrete mixtures tested in this study as both are categorized as moderate permeability per ASTM C1202. The CTE of the conventional latex-modified mixture is significantly higher than other CTE results in this study indicating less desirable thermal behavior. The reason for this test result is not immediately evident. The CTE of the internally cured mixture was in the range of all other CTE specimens studied in this research, 5.0×10^{-6} to 6.0×10^{-6} in./in./°F.

5.4 Summary of Laboratory Testing

Laboratory testing of the ten bridge deck concrete mixtures in this study provided valuable information pertaining to the future implementation of ICC in bridge decks.

Laboratory testing of an internally cured pavement mixture and latex-modified concrete mixtures also provided insight into the potential performance of ICC in concrete mixtures other than bridge decks. Some key findings from the laboratory testing are:

- The benefits of internal curing using locally available LWA and typical North Carolina mixtures were confirmed by laboratory test results which showed significantly reduced autogenous shrinkage for internally cured mortars.
- Reductions in autogenous strain ranged from 30% to 56% depending on the type of LWA used, as well as the percentage replacement of prewetted LWA for normalweight fine aggregate.
- The type of LWA utilized to facilitate internal curing did not appear to significantly impact the differences in reduction of autogenous shrinkage.
- Higher replacement percentages of one LWA did impact the reduction in autogenous shrinkage. However, the replacement percentage of a second LWA did not appear to reduce autogenous shrinkage. Additional testing of this LWA at a higher replacement rate (in line with the replacement rate suggested by the Bentz and Snyder approach) is suggested.
- Benefits of internal curing were not readily observed in unrestrained shrinkage. This is consistent with other research (Henkensiefken et al. 2009, Ardeshirilajimi et al. 2016).
- Internally cured concrete mixtures in this study tended to have lower stress rates in restrained drying shrinkage. The average stress rate of conventionally cured specimens was 10.03 psi/day compared to 8.72 psi/day in internally cured specimens, a 13% reduction.

- The unit weight of all bridge deck mixtures exceeded 135 pcf indicating that internally cured concrete can be considered normalweight concrete per AASHTO LRFD Bridge Design Specifications (2012) and would not require the lightweight concrete modification factor, lambda (ACI 2016).
- Internally cured concrete mixtures using locally available LWA exhibited:
  - Fresh concrete properties similar to control concrete that could be adjusted with admixtures to meet NCDOT specifications.
  - Adequate compressive strength to meet NCDOT specifications.
  - Lower MOE than control mixtures, which could further aid in preventing early age cracking.
- The presence of fly ash in bridge deck concrete mixtures has significantly more impact on permeability test results (RCPT, surface resistivity and bulk conductivity) than internal curing using prewetted LWA especially at later ages.
- The CTE was reduced in all internally cured concrete mixtures compared to control mixtures. Reductions ranged from 3% (fly ash mixtures) to 11% (LMC). This indicates potential for improved thermal performance.

The results of laboratory testing indicate that the benefits of internal curing can be achieved using locally available prewetted LWA in concrete mixtures typically used in North Carolina bridge decks and pavements. These benefits include the potential for reduced shrinkage and cracking potential in these internally cured concrete mixtures.

CHAPTER 6: DEVELOPMENT OF CONSTRUCTION SPECIFICATIONS AND FIELD TESTING PROGRAM FOR PILOT PROJECT

6.1 Introduction

To develop construction specifications for the use of ICC by the NCDOT, it was desired by both the NCDOT and the research team to conduct a pilot project to closely monitor the batching and placement process of ICC as well as its performance over time for a specific project. This chapter presents pre-construction information and discussions pertaining to the operations of both the concrete supplier and contractor as they utilize this new technology in a concrete bridge deck in North Carolina. Details regarding the specific materials, proportioning, pouring, finishing, testing (laboratory and field) and performance of the pilot project are presented in subsequent sections of the chapter.

6.2 Development of Project Special Provisions

To assist the three major parties (concrete supplier, contractor, and NCDOT) in construction of the bridge deck and the use of internally cured concrete, an initial specification was prepared by the research team. This document is referred to as the Project Special Provisions (PSP) and is provided in Appendix E. A summary of the contents of the PSP are presented in subsequent sections.

6.2.1 Materials, Mixture Proportioning and Batching

The concrete deck for the subject bridge is to be constructed using two concrete mixtures: one that is a conventional concrete mixture, and a second concrete mixture designed to provide internal curing. The deck in Stage I of the project (southbound lanes) is to be constructed to include one span of conventional normalweight concrete and one span of the internal curing concrete. Similarly, the deck in Stage II of the project (northbound lanes) will also be constructed to include one span of conventional concrete and one span of the internal curing concrete. Alternate spans will be selected for placement of each mixture, as shown in Figure 6.1, below. If desired, the contractor may elect to switch the spans for placement of each type of mixture. However, alternate spans are to be utilized for each mixture.

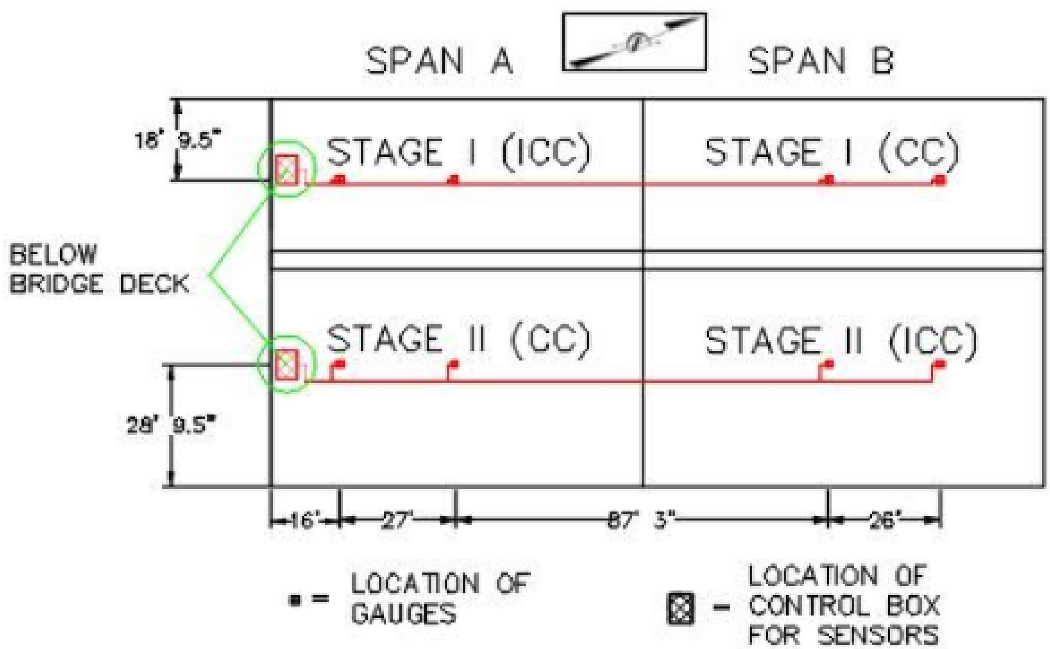


Figure 6.1: Schematic of bridge deck for pilot project

The concrete mixtures for this project are to be Class AA concrete meeting NCDOT Standard Specifications Section 1000-4. Cementitious materials utilized for the bridge deck will include fly ash using substitution rates allowable per NCDOT Specifications (NCDOT 2012). The internally cured concrete mixture will contain the same materials and mixture proportions utilized for the Class AA concrete mixture, but should be modified by substituting 30% prewetted lightweight fine aggregate (meeting AASHTO M195) by volume for the standard fine aggregate. Additional guidance for developing the internally cured concrete are provided in ACI (308-213) R-13. Both concrete mixtures, conventional and internally cured, will include air entraining admixture, water-reducing admixture, and other admixtures as required to meet NCDOT specifications for Class AA concrete and project specifications.

The proposed concrete mix designs, along with supporting test data, should be submitted to the Materials Engineer on Materials and Tests Form 312U for approval at least 14 days prior to placement. The following information should be provided for both the conventional and internally cured concrete mixtures:

- Cementitious materials content (pcy)
- Water content (pcy)
- Coarse aggregate content (pcy)
- Fine aggregate content (pcy) based on saturated surface dry (SSD) conditions
- Admixture dosages
- Compressive strength test results per NCDOT submittal requirements (psi)

The PSP further states that corrections must be made to account for additional free (surface) moisture contained on the prewetted LWA. The moisture content of the prewetted LWA should be determined using New York State DOT test method NY 703-19E "Moisture Content of Lightweight Fine Aggregate." The free moisture content of the prewetted LWA should be determined immediately prior to batching and adjust batch weights accordingly. The water should not be adjusted the mix water for absorbed water within the LWA, as it is retained within the LWA and does not affect the mix water. The entrained air content should be tested per ASTM C231 using a Type B pressure meter at the prewetted LWA is saturated prior to use.

6.2.2 Stockpile Management for Prewetted LWA

The PSP developed for this pilot project states that a stockpile area and moisture delivery system for the LWA should be established at the concrete production facility. Sprinklers are suggested for use, as they have been successfully utilized by other state agencies to pre-wet LWA stockpiles for internal curing purposes (Streeter et al. 2012). Stockpile management and maintenance is required to ensure the stockpile does not become contaminated with other materials, and the PSP states that contaminated aggregate must be discarded.

The appropriate stakeholder (likely the ready-mix concrete supplier) is required to provide drainage and turning/remixing provisions so that the moisture content of the material is uniform throughout the stockpile. Prior to batching and delivery, the lightweight aggregate must be prewetted for a minimum of 48 hours, or until the moisture content of the stockpile exceeds the absorption of the fine aggregate (free moisture available). If a steady rain of comparable intensity occurs, the soaking system may be turned off until the rain ceases. Prior to batching, the aggregate shall be allowed to drain for a period of 12 – 15 hours. Just prior to use of the material for production, the stockpile must be turned and remixed to obtain a homogeneous aggregate moisture content. The loose unit weight of the LWA should be measured following the procedures of ASTM C 29 (loose bulk density) periodically to ensure consistency and to aid in mix water adjustments.

6.2.3 Placement

The PSP states that placement of the conventional normalweight concrete and internally cured concrete mixtures shall be performed in accordance with NCDOT standard specifications and project special provisions. If placement of concrete is to be performed with a concrete pump, a minimum 5 inch diameter pump line is to be used to decrease the pressure that may prematurely draw the water out of the LWA pores. The finishing of internally cured concrete should be no different from the finishing of conventional concrete.

6.3 Pre-Construction Interviews

As part of preparation for construction of the pilot project as well as development of a specification for ICC, pre-construction interviews with each of the stakeholders were performed in order to evaluate opinions and perceptions regarding the PSP. The three stakeholders are the concrete supplier, contractor, and NCDOT personnel. The interviews consisted of the research team asking questions that obtained the desired feedback on the PSP document, perceptions going into the pilot project and thoughts on development of the PSP into a general specification for the future use of ICC. Several of the interview questions are presented below. The results of the three interviews are presented in subsequent sections.

- Have you found the PSP to be beneficial in preparation for construction of the bridge deck?
- What concerns do you have with the PSP and other pre-construction perceptions regarding the bridge deck?

- (Concrete Supplier) What challenges at the batch plant do you anticipate in regard to internally cured concrete?
- (Contractor) What construction challenges do you anticipate in regard to the bridge deck?
- (NCDOT) What challenges have you seen and anticipate with the use of internally cured concrete in both the pilot project and future projects?
- Are there any notes or inclusions you would like to see added to the PSP and future specifications?

6.3.1 Concrete Supplier Interview

The concrete supplier representative (a technical services specialist) provided insight into the production considerations and challenges of preparing the ICC for the pilot project. He stated that the production of ICC is not a significant challenge and that the supplier has batched concrete mixtures in the past that contained saturated lightweight fines. He does not think that this instance will be much different and ICC production should be fairly straightforward. He also commented on the benefits of being able to utilize the standard pressure meter rather than the volumetric method to determine air content. Furthermore, the concrete supplier’s representative did not foresee any issue with stockpile management provisions contained in the PSP, as their company traditionally practices similar non-contamination procedures.

The representative did, however, identify several challenges associated with the production of ICC for both the pilot project and potential future projects. In the previous instance where the supplier produced a concrete mixture with lightweight fines, staff members with technical expertise were available that are no longer with the company. The representative is working with new personnel to develop these mixtures.

The procedure for NCDOT approval of the ICC mixture was another concern raised by the representative. The ICC mixture is a modification of a new design, but replaces one approved material with another approved material. Typically with a new mixture design, three point curves plotting w/c ratio versus compressive strength are submitted to NCDOT. The supplier indicated that provisions guiding mixture submittal requirements for ICC mixtures would be an improvement for future ICC specifications.

Another challenge identified by the concrete supplier's representative was the need for an additional weigh bin for the prewetted LWA. He stated that for use of ICC in future projects, the batch plant capabilities (for storage and handling) will likely determine their ability to successfully produce ICC. Existing batch plants generally have a specific number of silos and bins that are already assigned to contain a specific material used to batch regularly produced concrete mixtures. Since LWA fines are not frequently used in current concrete mixtures, the supplier cannot dedicate a silo or bin for the LWA fines. This means that a stockpile and front-end loader will be used to sample and prepare the LWA for batching. The representative stressed that loader operator training will be critical, ensuring that the individual focus on avoiding contamination of the stockpile as well as stockpile mixing to ensure consistent moisture state. The supplier's final comments expressed concern that this is a lot of work to be performed to manage a small amount of material for the pilot project (roughly a truckload of LWA).

Overall, although the concrete supplier's technical service representative is still working through the challenges of implementing provisions to provide the ICC for the pilot project, he stated that the PSP was well written and includes the necessary information. He suggested no further provisions for the future specification other than guidance on submittal and approval procedures for the ICC mixture designs.

6.3.2 Contractor Interview

The contractor representative that was interviewed was the project manager with the construction company. The contractor's representative stated that in general he had no major concerns with the provisions outlined in the PSP. The contractor indicated that his concern was primarily that the concrete supplier learned the project was an ICC project later during the contract phase. The concrete supplier bid the work under the assumption that it was a standard sand lightweight mix. The contractor stated that in future projects, he plans to ensure additional communication with the concrete suppliers bidding the project, and ensuring the suppliers are well aware that the contract includes supplying ICC. To promote awareness of the use of ICC on future projects, the representative recommended that the drawings and specifications for the specific project highlight the fact that ICC is to be utilized in other ways in addition to a specification (which may become "buried" in the rest of the contract documents). He suggested that this could be done by indicating ICC clearly on the drawings or in the drawing notes section. Additionally, the representative suggested creating a separate pay item for bidders, which would clearly indicate that use of a less conventional material was required, and should be considered in the bidding phase.

When asked about challenges for future projects, the contractor's representative stated that producing ICC may be more problematic in rural areas or smaller markets. In these areas, concrete batch plants tend to be smaller, and may not have the capacity (weigh bins) or technical expertise to handle prewetted LWA fines and successfully batch ICC mixtures. The contractor's representative also indicated that ICC technology may be easily utilized in projects that utilize a large quantity of concrete such as pavements that are several miles in length. Since concrete batch plants are typically established on-site for these projects, the necessary provisions for handling and batching the prewetted LWA could be incorporated at the bidding/planning stages of the project.

6.3.3 NCDOT Interview

The NCDOT personnel interviewed were the Division 5 Resident Engineer and Senior Assistant Resident Engineer. These NCDOT personnel stated that it was early to provide a thorough review of the PSP and associated items, but in their opinion, they have no concerns at this point in the project. It was stated that the provisions included in the PSP appear to be readily transferrable to a more general specification for ICC, and no suggestions for modification are warranted at this point in time.

Potential issues foreseen by these representatives when considering the future use of ICC in NCDOT projects is a lack of resources potentially available for smaller concrete suppliers in rural areas of the state. These resources could include a lack of physical space/bins or personnel to accommodate an additional material at the batch plant, or a lack of technical expertise of personnel at these plants.

6.4 Proposed Field Testing and Monitoring Program

During placement of the bridge deck, the research team plans to collect data from the field-placed concrete as well as prepare specimens for laboratory testing. As part of the pilot project, the PSP outlines provisions to allow the research team to perform preconstruction testing. Many tests planned for the pilot project include those performed as part of the laboratory portion of this project. Tests that will be performed include mechanical properties, thermal expansion, durability performance and shrinkage characteristics. Procedures for preconstruction laboratory tests are described in Chapter 5. The pilot bridge deck is also scheduled to include embedded instrumentation as well as instrumentation installed after the concrete hardens. The subsequent sections describe the intended field testing methods and procedures, laboratory test specimens required, and additional information to be gathered from the pilot project.

6.4.1 Mechanical Property Tests

During the concrete placement for each section of bridge deck, the research team will acquire fresh concrete to prepare specimens for transport and testing of mechanical properties and durability performance in the laboratory. Tests will include compressive strength, MOE and Poisson’s ratio, and CTE.

6.4.2 Durability Performance Tests

To provide data to help assess the potential durability performance of the bridge deck, the surface resistivity meter will be used to obtain field measurements directly on the surface of the bridge deck at multiple ages. In the laboratory, cylinders will be tested for surface resistivity and bulk conductivity at ages of 1, 3, 7, 28 and 90 days after placement. RCPT will also be performed at 28 and 90 days of age. Unrestrained drying shrinkage tests will also be performed.

6.4.3 In-Situ Moisture Tests

In addition to standard laboratory tests on concrete detailed in Chapters 4 and 5, several test slabs were constructed and tested in the laboratory to develop a system of embedded sensors capable of monitoring internal strain and humidity at the pilot project. The embedded sensor system is not included in the scope of this thesis. However, the test slabs containing embedded instrumentation to monitor and document the behavior of ICC were utilized to develop a low-cost in-situ moisture monitoring protocol discussed subsequently in this section.

In general, there were two properties monitored within the test slabs, strain and internal relative humidity. Theoretically, ICC should develop less strain and possess more internal relative humidity than conventionally cured concrete. These two properties were measured over time for ICC laboratory test slabs. Strain was measured using vibrating wire strain gages embedded within the concrete. The internal relative humidity was measured with two different types of sensors, one embedded within the concrete and one installed into holes drilled after the concrete hardened. Further details, results and discussion of these three types of instrumentation are presented in Loflin (2017).

The low-cost method of monitoring in-situ moisture of the test slabs developed as part of this thesis involved measuring the change in weight over time of wooden dowels partially embedded within the concrete. Theoretically, ICC should possess a higher relative humidity than conventionally cured concrete due to the extra moisture stored within the LWA. Therefore, the weight of a wooden dowel embedded within ICC should be higher than the weight of a similar wooden dowel embedded in conventional concrete. Theoretically, a wooden dowel conditioned to a high moisture state should maintain its weight or potentially even gain weight in moisture due to the excess moisture present within the hydrating paste. A wooden dowel embedded in conventionally cured concrete should lose weight (moisture) over time as the cement paste requires more hydration than originally provided by the batch water. Utilizing testing of these embedded dowels as a field test could provide the contractor and concrete supplier a low-cost, rapid indication that internal curing is occurring and benefiting the cement paste.

To develop this field test method, wooden dowel specimens were prepared from one inch diameter poplar dowels (available commercially at common building supply stores). Each dowel bar was cut to 10 inch segments in length and 6 inches of each segment were coated with a high performance protective enamel. The 6 inches of enamel coating is essentially the “handle” of the dowel while the other 4 inches is embedded within the concrete and is susceptible to moisture transport. A picture of a completed wooden dowel specimen is provided in Figure 6.2.

Figure 6.2: Wooden dowel specimen

6.4.3.1 Development of Method by Other Researchers

This method is also currently under development by a researcher (Dr. Jason Weiss) at Oregon State University. Publications are currently not available on this method, but are in development by Dr. Weiss. As part of this research, the author worked concurrently on development of this method using materials available in North Carolina.

Through conversations with Dr. Weiss, it was recommended to develop calibration curves for each wooden dowel prior to embedding them within concrete. This is due to the fact that although wood is a homogenous material, the pore microstructure tends to vary especially in regard to absorption and desorption isotherms. Calibration curves are developed by placing wooden dowels in different environments with varying relative humidity and allowing the weight of the dowel to reach equilibrium within each atmosphere. Dr. Weiss (2017) stated that conditioning the dowels in enclosed containers over several types of salt solutions producing different relative humidity environments is a method that has been successful in their development of the method. For example, a supersaturated solution of NaCl generates a 75% relative humidity environment, a supersaturated solution of potassium chloride provides 85% relative humidity, and a supersaturated solution of potassium nitrate provides a 93% relative humidity – conditioning the dowels in these environments produces an appropriate calibration curve. Another option to develop calibration curves for the dowels is use of an environmental chamber at multiple humidity settings (e.g. 0%, 50% and 100%).

Once a calibration curve is developed for the wooden dowel, the exposed (non-sealed) end of the dowel can be embedded into the concrete. The weight of the wooden dowel can be monitored over time, and the calibration curve for each dowel can be used to provide an indication of the relative humidity of the concrete over time.

6.4.3.2 Calibration of Dowels

Wooden dowel calibration curves were developed at as part of this research in a similar manner recommended by Dr. Weiss (Weiss 2017). Four different relative humidity environments were utilized to develop the calibration curves. Each wooden dowel was oven dried to identify the oven dried mass (the 0% relative humidity environment). Two environmental chambers were utilized to provide the 50% and 100% relative humidity environments for calibration. Lastly, a supersaturated solution of potassium nitrate was utilized to develop the 93% relative humidity environment.

Five wooden dowels, similar to the one pictured in Figure 6.2, were placed in each of these environments and weighed frequently until the difference in weight over 24 hours was less than 0.01 grams. The equilibrium weight of each dowel at each relative humidity was determined as well as the amount of moisture present in each dowel. The amount of moisture of each dowel at each relative humidity is presented in Figure 6.3 with an average calibration curve of the specimens presented in Figure 6.4.

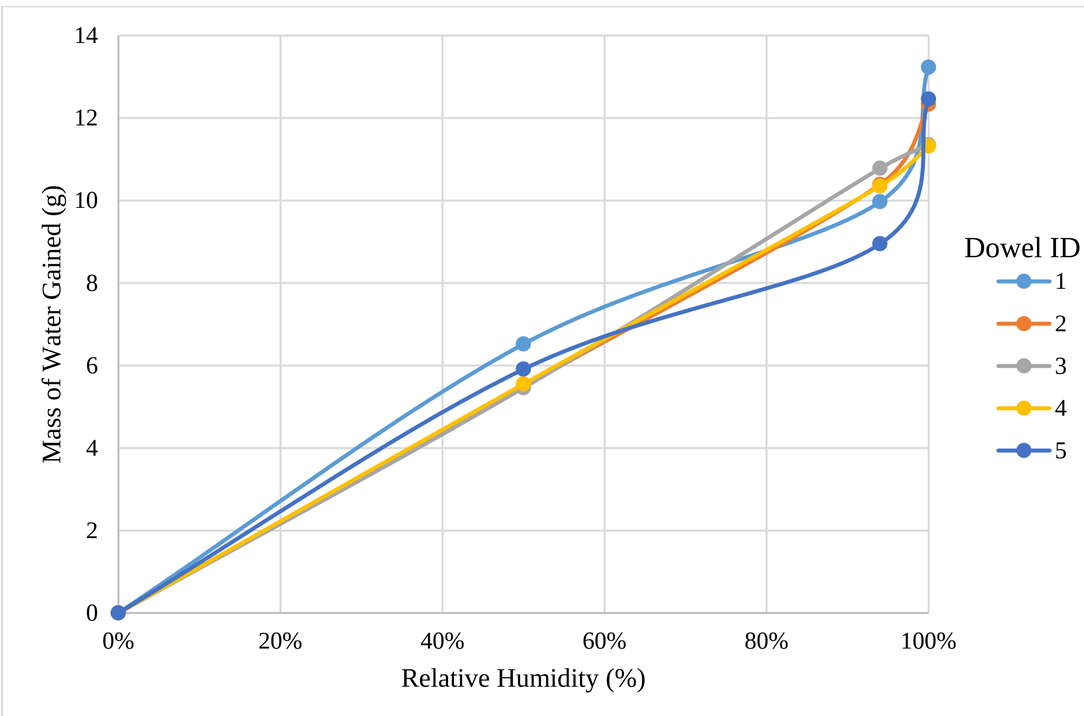


Figure 6.3: Wooden dowel calibration curves

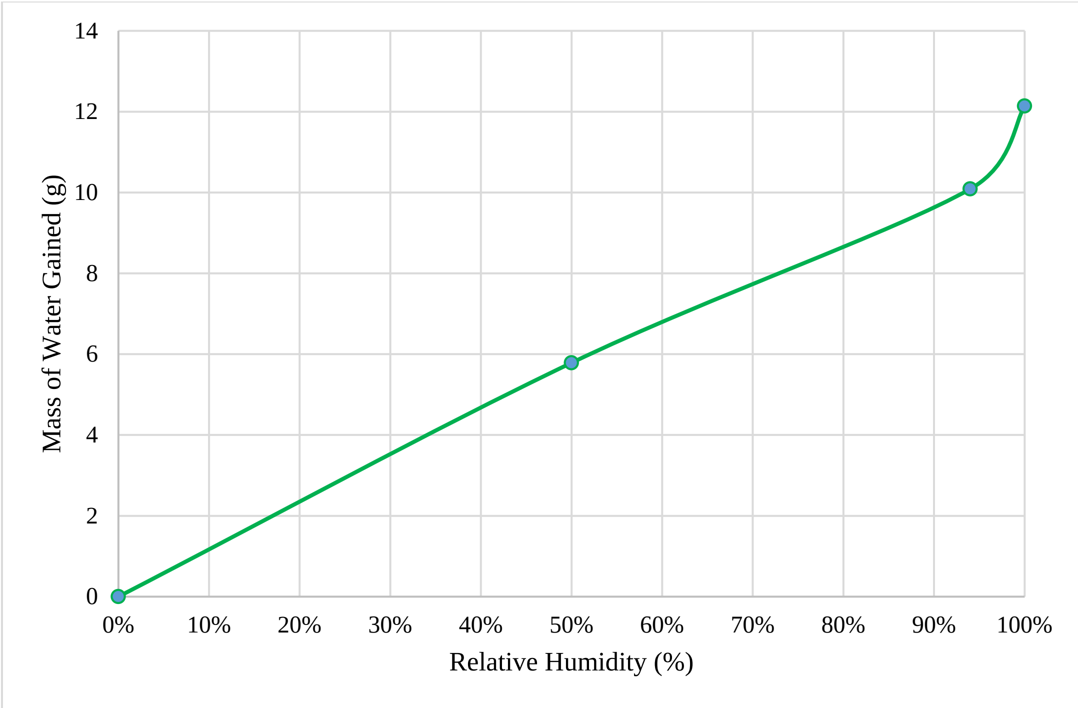


Figure 6.4: Average wooden dowel calibration curve

6.4.3.3 Validation of Method Using Laboratory Slab

The wooden dowel specimens that were calibrated in the previous section were placed in a small test slab cast in the laboratory at UNC Charlotte. During placement, one inch diameter PVC tubes were placed in the fresh concrete until first set. This was done so that the wooden dowels did not have a small layer of cement paste stick to them while the concrete was still in a fresh state. Photographs of the laboratory test slab before and after placement are presented in Figure 6.5 and 6.6. Documentation of the embedded instrumentation within the laboratory test slab as well as instrumentation installed retroactively is presented in Loflin (2017).

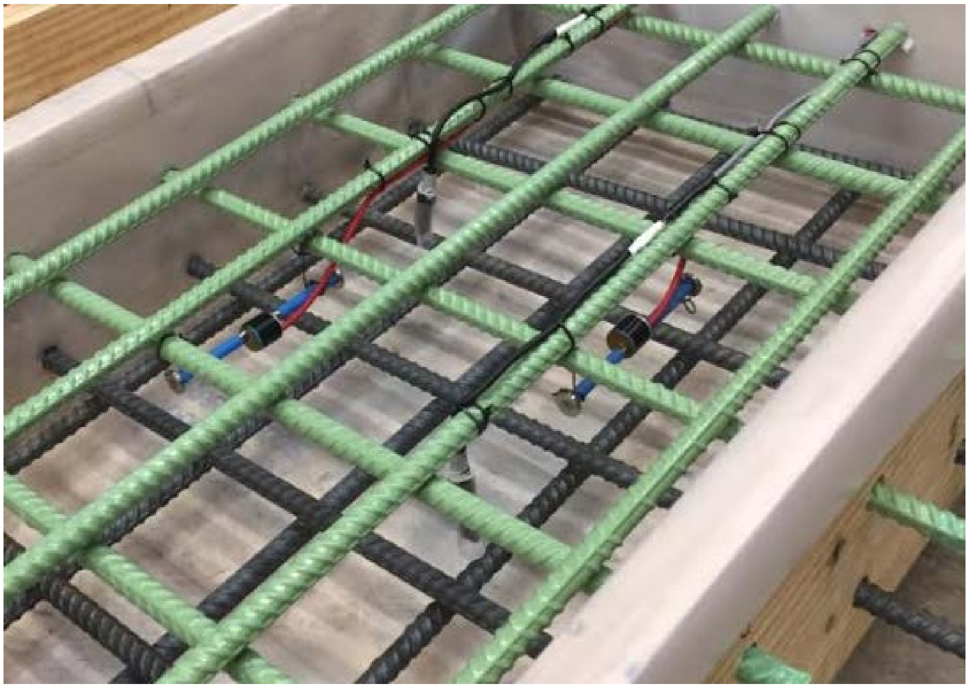


Figure 6.5: Formwork, rebar and embedded instrumentation for laboratory test slab

Figure 6.6: Laboratory test slab with PVC tubes after placement

Wooden dowels were soaked for approximately 24 hours prior to being placed within the laboratory test slab. Approximately 8 hours after placement, the PVC tubes were removed and the wooden dowels were inserted. The weight of each wooden dowel was measured frequently with no more than three days in between measurements. A photograph of the wooden dowels inserted into the test slab is presented in Figure 6.7.

Figure 6.7: Wooden Dowels Placed Within Laboratory Test Slab

Results from moisture monitoring of a test slab of internally cured concrete are presented in Figure 6.8. In this figure, the relative humidity obtained from an embedded humidity sensor is shown, along with the relative humidity results from four calibrated wooden dowels. At the end of four weeks, three of the four dowels (Dowels D, G, and X) possessed a mass that exceeded the mass observed at 100% relative humidity (from Figures 6.3 and 6.4). One wooden dowel's mass (Dowel A) had fallen slightly below the mass observed at 100% relative humidity, indicating that the relative humidity of the wooden dowel has fallen slightly beneath 100%. Although relative humidity values from the dowels do not directly correspond with relative humidity measured by the sensor, these early results indicate that the test slab had not obtained moisture from the dowels, and sufficient moisture to support internal curing was likely available.

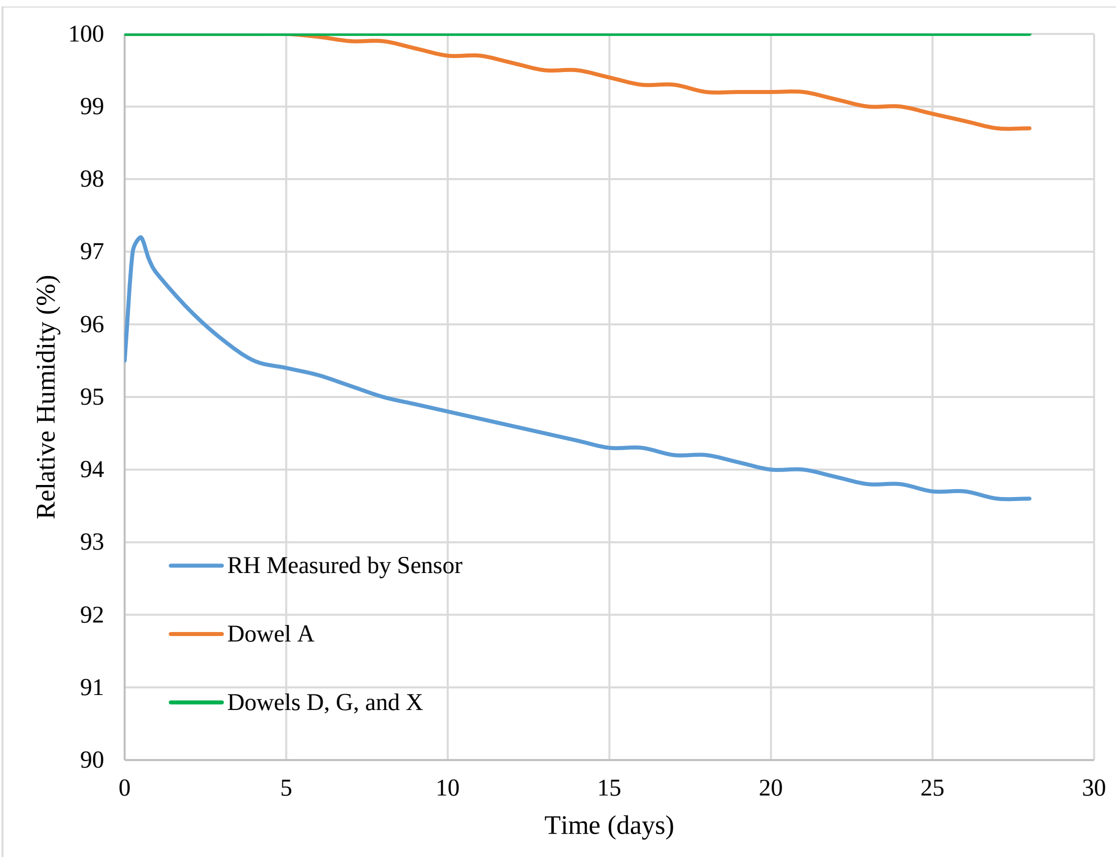


Figure 6.8: Results of in-situ moisture monitoring for ICC test slab

In Figure 6.9, the percent of moisture in each dowel, relative to the 28 day moisture content, is shown. It is readily apparent that at early ages, dowels D, G, and X possessed more moisture than the 28 day moisture content used to develop the calibration curves. It is suspected that the dowels obtained additional moisture from the external curing. However, since the mass of water in the dowels did not decrease below the quantity absorbed at 28-day saturation, it can be deduced that the concrete had enough moisture from the prewetted LWA to support internal curing, and did not take an excessive amount of moisture from the wooden dowels.

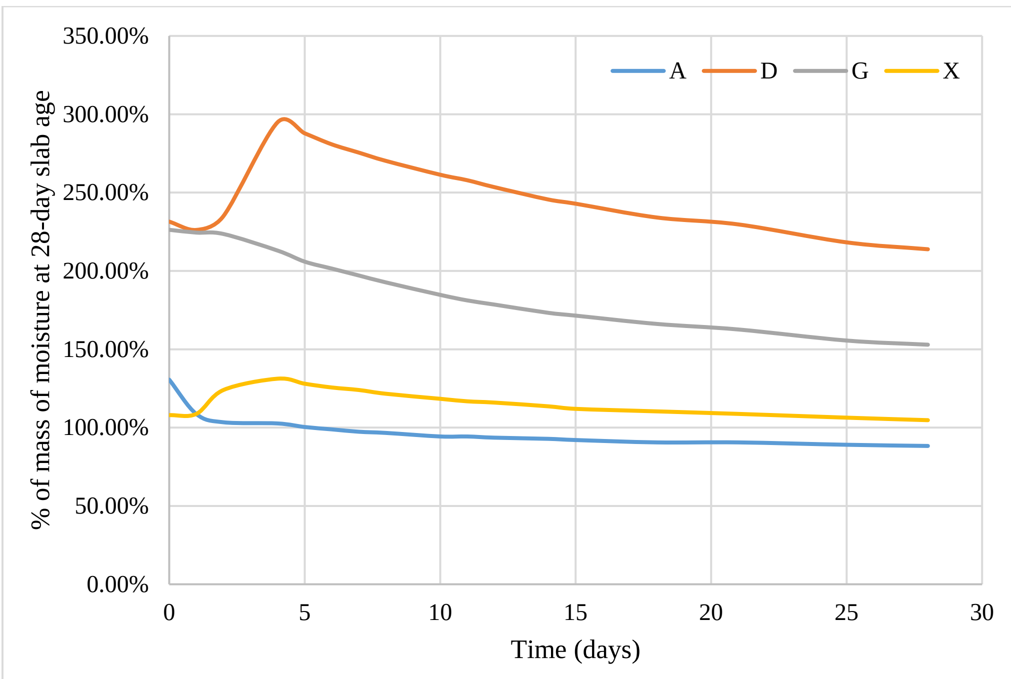


Figure 6.9: Percent of mass of moisture in dowels at 28 days

Although preliminary results with this low-cost in-situ field test to monitor ICC show promise, the calibration procedure for the dowels may need to be lengthened. Additionally, measures to prevent external curing water from interfering with the test should be taken. Work is needed to refine this procedure as part of the larger ongoing research effort.

6.4.3.4 Field Monitoring Plan

The procedure for in situ-moisture content using low cost wooden dowels in the laboratory was presented in the previous section. A similar procedure is to be utilized for in-situ moisture monitoring in the field of the pilot project bridge deck. It is planned to place the embedded dowels near in-situ moisture sensors to verify the validity of the method. Dowels will be prepared and calibrated prior to construction of the pilot project. Readings obtained from the dowels will be correlated to the calibration curves as well as to in-situ moisture readings obtained from the embedded and retrofit in-situ moisture sensors.

6.5 Results from Pilot Project Development

This section presents some of the results that were determined as part of preparation for the pilot project. These findings should be considered preliminary, as other findings regarding construction challenges and potential specification provisions will be identified during and after the construction of the pilot project. These results regarding pilot project development are separated into two categories, findings and recommendations for specifications. They are presented in subsequent sections.

6.5.1 Summary of Findings

From laboratory results and interviews with stakeholders, it has been found that the bulk of the effort to successfully implement ICC in a field project is placed on the concrete supplier. If the concrete supplier is able to successfully batch the ICC, the contractor should see very little difference with ICC construction compared to conventionally cured concrete. This is because the ICC concrete should have similar workability to conventional concrete (from laboratory testing as part of this study and others), and field tests such as slump and air content are performed in the same manner as conventional concrete. Placing and finishing operations for ICC are reportedly identical to that of conventional concrete (Bentz and Weiss 2011).

As the bulk of the responsibility is placed on the concrete supplier, it is important to highlight some of the key findings regarding operations at the batch plants. First and foremost is stockpile management of the LWA. The batch plant should have adequate space and resources to properly produce ICC. Typically, a relatively small stockpile with a sprinkling system is used. It is recommended by Barrett, Miller et al. (2015) that LWA be stored in a stockpile that is limited in height (five feet or less) and have as many free directions of drainage flow as possible available in an effort to minimize non-uniformity in surface moisture throughout the pile. As stated previously in this report, the stockpile is to be sprinkled with water for a minimum of 48 – 72 hours and drained for 12 – 15 hours prior to batching. Immediately prior to batching it is recommended that the stockpile be mixed by a loader to uniformly distribute the surface moisture. It is also recommended to take care to not sample from the bottom 4 inches of the stockpile as this area will possess more moisture due to gravity and to also avoid contamination. After the stockpile has been mixed, the LWA is ready for batching. In preparing trial batches of ICC for field use, management of the prewetted LWA could be performed in a manner similar to the soaking, draining, and pre-batching methods used in this work (described in Section 4.4).

As stated previously in this report, it is recommended by several state agencies that the mix design possess a 30% volumetric substitution of prewetted LWA for normalweight fines. The LWA must also be tested for surface moisture and accounted for by subtracting surface moisture from the batch water. Miller et al. (2014) recommend use of the centrifuge method to rapidly determine the quantity of surface moisture of the LWA stockpile. Furthermore, Barrett, Miller et al. (2015) state the following in regard to batching of prewetted LWA:

> "While batching systems may vary, it is common for the system inputs to be in terms of SSD design weight, absorption, total moisture, and specific gravity. The system then calculates free moisture (surface moisture) by subtracting absorption from total moisture. Many of these batching systems have built in limits for these values. It is common for the lightweight aggregate to exceed both the absorption and total moisture limits due to its high absorption capacity and the pre-wetting duration leaving the stockpile with large amounts of free moisture. If this is the case, it is often necessary to "trick" the computer system into batching the right amount of LWA. This can be accomplished by setting the absorption input to 0% and setting the total moisture to the value of free moisture. The batching system will then account for the free moisture correctly."

The above procedure will need to be tested and confirmed by individual batch plants when producing ICC. However, this is a useful finding for the batch plant as it may assist in better quality control of ICC.

6.5.2 Recommendations for Specifications

This section recommends provisions for inclusion in an NCDOT specification for use of internally cured concrete. Many of the specifications are directed towards the concrete supplier with several specifications directed towards the contractor.

- Internally cured concrete mixtures shall contain the same materials and mixture proportions as other approved NCDOT mixtures but should be modified by substituting lightweight fine aggregate (meeting AASHTO M195) by volume for the standard fine aggregate. (Note: testing to determine the percent replacement is still under investigation as part of this research, and will be identified in later stages of this project).

- Internally cured concrete mixtures should include air entraining admixture, water-reducing admixture and other admixtures as required to meet NCDOT specifications and project specifications.
- Submit proposed internally cured concrete mixture designs, along with supporting test data, to the Materials Engineer on Materials and Tests Form 312U for approval at least 14 days prior to placement.
- Lightweight fine aggregate shall be stored in a stockpile at the concrete production facility. The stockpile shall not exceed five feet in height and shall not be restrained at any point as to minimize moisture non-uniformity throughout the pile.
- Lightweight fine aggregate stockpiles shall be prewetted for a minimum of 48 – 72 hours or until the moisture content of the stockpile exceeds the absorption of the lightweight fine aggregate (free moisture available). This shall be done through the use of sprinklers or soaker hoses. If a steady rain of comparable intensity occurs, the soaking system may be turned off until the rain ceases.
- Prior to batching of the lightweight fine aggregate, the stockpile shall be allowed to drain for 12 – 15 hours. If rain occurs during this time and the stockpile is exposed, an impermeable tarp of sufficient size shall be used to cover the stockpile and allow it to drain.

- After the draining period and immediately prior to batching, the stockpile shall be turned and remixed to obtain a homogenous aggregate moisture content. The loose unit weight of the lightweight fine aggregate shall be measured following the procedures of ASTM C 29 (loose bulk density) periodically to ensure consistency and to aid in mix water adjustments.
- Immediately prior to batching, the surface moisture of the lightweight fine aggregate shall be determined. This may be determined by using the New York State DOT test method NY 703-19E “Moisture Content of Lightweight Fine Aggregate” or Indiana DOT Testing Method (ITM) 222-15T “Specific Gravity Factor and Absorption of Lightweight Fine Aggregate.”
- The surface moisture of the lightweight fine aggregate shall be subtracted from the batch water prior to mixing. Do not adjust the mix water for absorbed water in the prewetted lightweight fine aggregate, as it is retained within the aggregate and does not affect the mix water.
- During batching of internally cured concrete, the prewetted lightweight fine aggregate and a small portion of the batch water shall be placed in the mixer first and mixed several revolutions. This is to ensure the lightweight fine aggregate is prewetted. The remainder of the batching procedure shall follow NCDOT specifications.
- Fresh concrete shall be tested for air content per ASTM C 231 using a Type B pressure meter as the prewetted lightweight fine aggregate is saturated prior to use.

- Placement of internally cured concrete shall be performed in a similar manner as conventionally cured concrete per NCDOT standard specifications and project specifications.
- If placement of concrete is to be performed with a concrete pump, a minimum 5 inch diameter pump shall be used to decrease the pressure in the line that may prematurely draw the water out of the lightweight aggregate pores.
- Conventional curing techniques including wet burlap and polyethylene sheeting shall be performed on internally cured concrete in a similar manner to conventionally cured concrete per NCDOT specifications and project specifications.

6.6 Concluding Remarks

This chapter has presented the initial development of specifications and field testing protocols for the use of internally cured concrete in North Carolina. Documentation of field studies and other state highway agency specifications for ICC were reviewed and synthesized for development of a PSP for a North Carolina ICC pilot project. Stakeholders involved in the pilot project were interviewed for preliminary input on the specification provisions and construction challenges that may be faced in future use of ICC in North Carolina. Provisions for field and laboratory testing of concrete used to construct the pilot project have been outlined, and the pilot project is to be constructed after submission of this thesis. Additionally, preliminary testing to develop a low-cost field test for internal curing has been performed. This low-cost field test could be used to help stakeholders (the contractor, supplier, or state highway agency) verify that free moisture is present to facilitate internal curing at the field site. Provisions for future tests will be utilized to validate this method are presented.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

This work presents laboratory testing verifying the benefits of ICC for North Carolina concrete materials and mixtures, supporting the implementation of ICC in concrete infrastructure construction. The development of preliminary specifications and field testing protocols for the use of ICC were presented. Documentation of literature from universities and state agencies, field studies and other state highway specifications for ICC were reviewed and synthesized for development of a PSP for a North Carolina ICC pilot project. Stakeholders involved in the pilot project were interviewed for preliminary input on the specification provisions and construction challenges that may be faced in future use of ICC in North Carolina. Provisions for field and laboratory testing of concrete used to construct a pilot project have been outlined.

In this chapter, a summary of the research findings from laboratory testing is presented, along with key considerations utilized in development of a preliminary construction specification for a pilot project. Additionally, recommendations for future work are presented.

7.1 Findings and Conclusions

Laboratory testing of the thirteen concrete mixtures in this study provided useful information supporting the future implementation of ICC in bridge decks and pavements. Key findings from the laboratory testing are:

- The benefits of internal curing using locally available LWA and typical North Carolina mixtures were confirmed, since test results showed significantly reduced autogenous shrinkage for internally cured mortars.
- Reductions in autogenous shrinkage ranged from 30% to 56% depending on the type of LWA used, as well as the percentage replacement of prewetted LWA for normalweight fine aggregate.
- The type of LWA utilized to facilitate internal curing did not appear to significantly impact the differences in reduction of autogenous shrinkage.
- Higher replacement percentages of one LWA did impact the reduction in autogenous shrinkage. However, the replacement percentage of a second LWA did not appear to reduce autogenous shrinkage. Additional testing of this LWA at a higher replacement rate (in line with the replacement rate suggested by the Bentz and Snyder approach) is suggested.
- Benefits of internal curing were not readily observed in unrestrained shrinkage. This is consistent with other research (Henkensiefken et al. 2009, Ardeshirilajimi et al. 2016).
- Internally cured concrete mixtures in this study tended to have lower stress rates in restrained drying shrinkage. The average stress rate of conventionally cured specimens was 10.03 psi/day compared to 8.72 psi/day in internally cured specimens, a 13% reduction.
- The unit weight of all bridge deck mixtures exceeded 135 pcf indicating that internally cured concrete can be considered normalweight concrete per AASHTO

LRFD Bridge Design Specifications (2012) and would not require the ACI 318-16 lightweight concrete modification factor, lambda (ACI 2016).

- Internally cured concrete mixtures using locally available LWA exhibited:
  - Fresh concrete properties similar to control concrete that could be adjusted with admixtures to meet NCDOT specifications.
  - Adequate compressive strength to meet NCDOT specifications.
  - Lower MOE than control mixtures, which could further aid in preventing early age cracking.
- The presence of fly ash in bridge deck concrete mixtures has significantly more impact on permeability test results (RCPT, surface resistivity and bulk conductivity) than internal curing using prewetted LWA especially at later ages.
- The CTE was reduced in all internally cured concrete mixtures compared to control mixtures. Reductions ranged from 3% (fly ash mixtures) to 11% (LMC). This indicates potential for improved thermal performance of ICC.

The results of laboratory testing indicate that the benefits of internal curing can be achieved using locally available prewetted LWA in concrete mixtures typically used in North Carolina bridge decks and pavements. These benefits include the potential for reduced shrinkage and cracking potential in these internally cured concrete mixtures.

In addition to laboratory testing, an ICC pilot project was planned and developed and results and findings are provided. These findings should be considered preliminary, as other findings regarding construction challenges and potential specification provisions will be identified during and after the construction of the pilot project.

From laboratory results and interviews with stakeholders, it has been found that the bulk of the effort to successfully implement ICC in a field project is placed on the concrete supplier. If the concrete supplier is able to successfully batch the ICC, the contractor should see very little difference with ICC construction compared to conventionally cured concrete. This is because the ICC concrete should have similar workability to conventional concrete (verified in laboratory testing as part of this study and others), and field tests such as slump and air content are performed in the same manner as conventional concrete. Placing and finishing operations for ICC are reportedly identical to that of conventional concrete (Bentz and Weiss 2011).

As the bulk of the responsibility is placed on the concrete supplier, it is important to highlight some of the key findings regarding operations at the batch plants. The batch plant should have adequate space and resources to store, handle, condition, and batch the prewetted LWA in order to properly produce ICC. Guidance on stockpile management and batching is presented in Chapter 6, and was included in the project special provisions in Appendix E. In preparing trial batches of ICC for field use, management of the prewetted LWA could be performed in a manner similar to the soaking, draining, and pre-batching methods used in this work (described in Section 4.4).

7.2 Recommendations for Future Work

This section provides recommendations for future work with ICC. Based on the findings of this research, there are four areas of future work: placement of ICC bridge deck (pilot project) to confirm laboratory findings regarding ICC with materials locally available to North Carolina, evaluation of field performance of the pilot project, development of a general ICC specification for NCDOT and lastly, a cost-benefit analysis.

It is anticipated that the pilot project bridge deck will be constructed in mid to late June of 2017. This document along with the PSP detail the necessary research needs during construction. Following construction, the pilot project performance (along with companion laboratory testing data) will be evaluated by the research team. This evaluation should provide specific information to use in development of an ICC specification as well as the cost-benefit analysis.

During the development of ICC specifications, it may be beneficial to evaluate the costs associated with ICC from both the concrete supplier and the contractor. These costs can be utilized in a cost-benefit analysis of the pilot project. It is anticipated that the initial cost of ICC bridge deck concrete will increase slightly due to the increased cost of manufactured LWA as well as increased batch plant operations. Jones et al. (2014) predict that ICC will increase the price of concrete by approximately \$3-10/cy of concrete. However, increased service life and reduced maintenance costs are anticipated which may justify the increased initial costs.

REFERENCES

AASTHO. (2006). “Standard Specification for Lightweight Aggregates for Structural Concrete.” M 195-06, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. (2009). “Standard Test Method for the Coefficient of Thermal Expansion of Hydraulic Cement Concrete.” T 336-09, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. (2011). “Surface Resistivity Indication of Concrete’s Ability to Resist Chloride Ion Penetration.” TP 95-11, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. (2012). “LRFD Bridge Design Specifications.” Washington, DC: American Association of State Highway Transportation Officials.

AASHTO. (2017). “Standard Specification with Commentary for Performance Engineered Concrete Pavement Mixtures.” MP XX-17, American Association of State Highway and Transportation Officials, Washington, DC.

ACI (308-213) R-13. 2013. Report on Internally Cured Concrete Using Prewetted Absorptive Lightweight Aggregate. ACI committees 308, 213.

American Society for Testing and Materials (ASTM). (2006). ASTM Standard C136. “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates.” ASTM International, West Conshohocken, PA. DOI: 10.1520/C0136-06. www.astm.org

American Society for Testing and Materials (ASTM). (2009). ASTM Standard C29. “Standard Test Method for Bulk Density (“Unit Weight”) and Voids in Aggregate.” ASTM International, West Conshohocken, PA. DOI: 10.1520/C0029_C0029M-09. www.astm.org

American Society for Testing and Materials (ASTM). (2009). ASTM Standard C1581. “Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage.” ASTM International, West Conshohocken, PA. DOI: 10.1520/C1581_C1581M-09A. www.astm.org

American Society for Testing and Materials (ASTM). (2011). ASTM Standard C685. “Standard Test Method for Concrete Made by Volumetric Batching and Continuous Mixing.” ASTM International, West Conshohocken, PA. DOI: 10.1520/C0685_C0685M-11. www.astm.org

American Society for Testing and Materials (ASTM). (2011). ASTM Standard C496. “Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete

Specimens." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0496_C0496M-11. www.astm.org

American Society for Testing and Materials (ASTM). (2012). ASTM Standard C127. "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0127-12. www.astm.org

American Society for Testing and Materials (ASTM). (2012). ASTM Standard C143. "Standard Test Method for Slump of Hydraulic-Cement Concrete." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0143_C0143M-12. www.astm.org

American Society for Testing and Materials (ASTM). (2012). ASTM Standard C1202. "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." ASTM International, West Conshohocken, PA. DOI: 10.1520/C1202-12. www.astm.org

American Society for Testing and Materials (ASTM). (2012). ASTM Standard C91. "Standard Specification for Masonry Cement." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0091-12. www.astm.org

American Society for Testing and Materials (ASTM). (2012). ASTM Standard C1760. "Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete." ASTM International, West Conshohocken, PA. DOI: 10.1520/C1760-12. www.astm.org

American Society for Testing and Materials (ASTM). (2013). ASTM Standard C33. "Standard Test Method for Concrete Aggregates." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0033_C0033M-13. www.astm.org

American Society for Testing and Materials (ASTM). (2013). ASTM Standard C1437. "Standard Test Method for Flow of Hydraulic Cement Mortar." ASTM International, West Conshohocken, PA. DOI: 10.1520/C1437-13. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C39. "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0039_C0039M-14A. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C469. "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0469_C0469M-14. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C157. "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and

Concrete." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0157_C0157M-08R14E01. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C231. "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure method." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0231_C0231M-14. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C138. "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0138_C0138M-14. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C192. "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0192_C0192M-14. www.astm.org

American Society for Testing and Materials (ASTM). (2014). ASTM Standard C1698. "Standard Test Method for Autogenous Strain of Cement Paste and Mortar." ASTM International, West Conshohocken, PA. DOI: 10.1520/C1698-09. www.astm.org

American Society for Testing and Materials (ASTM). (2015). ASTM Standard C595. "Standard Test Method for Blended Hydraulic Cements." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0595M-15E01. www.astm.org

American Society for Testing and Materials (ASTM). (2015). ASTM Standard C128. "Standard Test Method for Relative Density (Specific Gravity) and Absorption of Fine Aggregate." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0128-15. www.astm.org

American Society for Testing and Materials (ASTM). (2015). ASTM Standard C150. "Standard Specification for Portland Cement." ASTM International, West Conshohocken, PA. DOI: 10.1520/C0150_C0150M-15. www.astm.org

Ardeshirilajimi, A., Wu, D., Chaunsali, P., Mondal, P. Chen, Y.T., Rahman, M.M., Ibrahim, A., Lindquist, W., Hindi, R. (2016). Bridge Decks: Mitigation of Cracking and Increased Durability. Report No. FHWA-ICT-16-016, ISSN: 0197-9191.

Barrett, T. J., Miller, A. E., & Weiss, W. J. (2015). Documentation of the INDOT Experience and Construction of the Bridge Cecks Containing Internal Curing in 2013 (Joint Transportation Research Program Publication No. FHWA/IN/JTRP-2015/10). West Lafayette, IN: Purdue University.

Bentur, A., Igarashi, S.-I., and Kovler, K. (2001). Prevention of Autogenous Shrinkage in High Strength Concrete by Internal Curing Using Wet Lightweight Aggregates. Cement and Concrete Research, 31(11), 1587-1591.

Bentz, D.P., Lura, P., and Roberts, J.W. (2005). Mixture Proportioning for Internal Curing, Concrete International, 27 (2), 35-40.

Bentz, D.P. (2007). Internal Curing of High Performance Blended Cement Mortars, ACI Materials Journal, 104 (4), 408-414.

Bentz, D.P. (2009). Influence of Internal Curing Using Lightweight Aggregates on Interfacial Transition Zone Percolation and Chloride Ingress in Mortars, Cement and Concrete Composites, 31, 285-289.

Bentz, D. P., Weiss, J. W. (2010). Internal Curing: A 2010 State-of-the-Art Review. NISTIR 7765.

Bentz, D. P., Weiss, J. W. (2013). Internal Curing: Research Findings and Observations from Implementation. PowerPoint PDF.

Bentz, D. P., Lura, P., Roberts, J. W. (2005). Mixture Proportioning for Internal Curing. Concrete International, 27 (2), 35-40.

Bentz, D.P., and Snyder, K.A. (1999). Protected Paste Volume in Concrete. Extension to Internal Curing Using Saturated Lightweight Fine Aggregate, Cement and Concrete Research, **29**, 1863-1867.

Blanchard, E. (2016). Determination and Evaluation of Inputs for Portland Cement Concrete Pavement to Support Local Calibration of MEPDG for North Carolina. Master’s Thesis. Department of Civil Engineering Technology and Construction Management. The University of North Carolina at Charlotte.

Castro, J., Keiser, L., Golias, M., and Weiss, J. (2011). Absorption and Desorption Properties of Fine Lightweight Aggregate for Application to Internally Cured Concrete Mixtures, Cement and Concrete Composites, 33 (10), 1001-1008.

Castrodale, R.W. and Cavalline, T.L. (2017). Thermal Properties of Lightweight Concrete and Their Impact on Bridge Design. 2017 Precast Concrete Institute (PCI) Convention and National Bridge Conference. February/March 2017. Cleveland, Ohio.

Chimmula, Rohit R. (2016). Quantitative Assessment off the Impact of use of Portland Limestone Cements in North Carolina Concrete Pavements. Master’s Thesis. The Department of Engineering Technology and Construction Management. University of North Carolina at Charlotte.

Choi, P. and Yun, K. (2014). Experimental Analysis of Latex-Solid Content Effect on Early-Age and Autogenous Shrinkage of Very-Early Strength Latex-Modified Concrete. Construction and Building Materials, Elsevier Publications, 65, 396-404.

Cusson, D. and Hoogeveen, T. (2008). Internal curing of high-performance concrete with pre-soaked fine lightweight aggregate for prevention of autogenous shrinkage cracking. Cement and Concrete Research, 38(6), 757-765.

Cusson, D., Lounis, Z., Daigle, L. (2010). Benefits of Internal Curing on Service Life and Life-Cycle Cost of High-Performance Concrete Bridge Decks – A Case Study. 339-350. Cement and Concrete Composites. National Research Council, Canada.

Delatte, N., Mack. E., Cleary, J. (2007). Evaluation of High Absorptive Materials to Improve Internal Curing of Low Permeability Concrete. State Job Number 134227, Final Report.

Friggle, T. and Reeves, D. (2008). Internal Curing of Concrete Paving Laboratory and Field Experiences. ACI SP-256, Bentz and Mohr, 71-80.

Geiker, M.R., Bentz, D.P., and Jensen, O.M. (2004). Mitigating Autogenous Shrinkage by Internal Curing. High-Performance Structural Lightweight Concrete. Proceedings of American Concrete Institute Fall Convention, 2004.

Gudimettla, J.M., Parvini, M., and Crawford, G.L. (2012). Concrete Coefficient of Thermal Expansion Test for Quality Assurance. Proceedings, International Conference on Long-Life Concrete Pavements.

Guthrie, W.S. and Yaede, J.M. (2013). Internal Curing of Concrete Bridge Decks in Utah: Preliminary Evaluation. Paper No. 13-5374, TRB 92, Jan. 13-17.

Henkensiefken, R. (2008). Internal Curing in Cementitious Systems Made Using Saturated Lightweight Aggregate. Master's Thesis. The Department of Civil Engineering. Purdue University.

Henkenseifken, R., Bentz, D., Nantung, T., and Weiss, J. (2009). Volume Change and Cracking in Internally Cured Mixtures Made with Saturated Lightweight Aggregate under Sealed and Unsealed Conditions, Cement and Concrete Composites, 31, 427-437.

Hoff, G.C. (2003). Internal Curing of Concrete Using Lightweight Aggregates, in Theodore Bremner Symposium on High-Performance Lightweight Concrete, Eds. J.P. Ries and T.A. Holm, Proceedings of the 6th CANMET/ACI International Conference on Durability of Concrete, Thessalonika, Greece, 185-204.

Holm, T.A., Ooi, O.S., and Bremner, T.W. (2003). Moisture Dynamics in Lightweight Aggregate and Concrete, in Theodore Bremner Symposium on High-Performance Lightweight Concrete, Eds. J.P. Ries and T.A. Holm, Proceedings of the 6th

CANMET/ACI International Conference on Durability of Concrete, Thessalonika, Greece, 167-184.

Holm, T.A., and Ries, J. (2007). "Chapter 2: Manufacturing of ESCS Lightweight Aggregate." Reference Manual for the Properties and Applications of Expanded Shale, Clay and Slate Lightweight Aggregate, Expanded Shale, Clay and Slate Institute, Chicago, IL.

Jones, W., House, M., and Weiss, J. (2014). Internal Curing of High Performance Concrete Using Lightweight Aggregates and Other Techniques, Research Report 2014-3, Colorado Department of Transportation, 129 pages.

Kovler, K., Souslikov, A., and Bentur, A. (2003). Pre-soaked lightweight aggregates as additives for internal curing of high-strength concretes. Cement, Concrete, and Aggregates, 26(2).

Loflin, G.D. (2017). Title unavailable. Master's Thesis. The Department of Civil Engineering. University of North Carolina at Charlotte. (draft)

Lopez, M., Kahn, L.F., and Kurtis, K.E. (2006). High-Strength Self-Curing Low-Shrinkage Concrete for Pavement Applications, submitted to International Journal of Pavement Engineering.

Lopez, M., Kahn, L.F., and Kurtis, K.E. (2008). Effect of Internally Stored Water on Creep of High Performance Concrete, ACI Materials Journal, 105 (3), 265-273.

Maruyama, I., Teramoto, A. (2012). Effect of Water-retaining Lightweight Aggregate on the Reduction of Thermal Expansion Coefficient in Mortar Subject to Temperature Histories. Cement & Concrete Composites, 34, Elsevier Publications.

Medlin, C. (2016). "Durability of Pavement Concrete with Replacement of Cement by Fly Ash and Portland Limestone Cement." Master's Thesis. The Department of Civil Engineering. University of North Carolina at Charlotte.

Memphis University. (2007). Curing Concrete. Design and Control of Concrete Mixtures, EB001. Chapter 12.

Miller, A.E., Barrett, T.J., Zander, A.R., Weiss, W.J. (2014). Using a Centrifuge to Determine Moisture Properties of Lightweight Fine Aggregate for Use in Internal Curing. Vol.3, No. 1, ACEM20130111.

Mokarem, D.W., Weyers, R.E., Lane, D.S. (2004). Development of a Shrinkage Performance Specifications and Prediction Model Analysis for Supplemental Cementitious Material Concrete Mixtures. Cement and Concrete Research 35, 918-925, Elsevier Publications.

Nantung, T. (2016). Monitoring Indiana High Performance Concrete Pavement. TRB, Concrete Pavement Instrumentation Workshop. PowerPoint Presentation.

NRMCA. (2000). CIP 11 – Curing In-Place Concrete. National Ready Mixed Concrete Association. 900 Spring St., Silver Spring, MD.

Neville, A. M. (2011). Properties of Concrete Fifth Edition.

Philleo, R.E. (1991). Concrete Science and Reality, in Materials Science of Concrete II, Eds. J. Skalny and S. Mindess, American Ceramic Society, Westerville, OH, 1-8.

Peled, A., Castro, J., and Weiss, J. (2010). Atomic Force Microscopy (AFM) Examinations of Mortar Made Using Water-Filled Lightweight Aggregate, Transportation Research Record: Journal of the Transportation Research Board, Vol. 2141/2010, 92-101.

Rao, C. Darter, M.I. (2013). Evaluation of Internally Cured Concrete for Paving Applications. Final Report, Illinois, ESCSI.

Rupnow, T.D (2017). Personal Communication with Cavalline, T. and Leach, J.

Rupnow, T.D., Icenogle, P.J. (2011). Evaluation of Surface Resistivity Measurments as an Alternative to the Rapid Chloride Permeability Test for Quality Assurance and Acceptance.

Schlitter, J., Henkensiefken, R., Castro, J., Raoufi, K., and Weiss, J. (2010). Development of Internally Cured Concrete for Increased Service Life, Final Report, FHWA/IN/JTRP-2010/10, Joint Transportation Research Program.

Speck, J. (2013). Internal Curing Using Prewetted Lightweight Aggregates: Improving Concrete Durability and Sustainability Using Internal Curing. Concrete Airport Pavement Worshop. Oct-29. PowerPoint Presentation.

Streeter, D.A. (2017). Personal Communication with Cavalline, T. and Leach, J.

Streeter, D.A., Wolfe, W.H., Vaughn, R.E. (2012). Field Performance of Internally Cured Concrete Bridge Decks In New York State. ACI SP-290, Edited by Schindler, Grygar and Weiss.

Tanesi, J., Kutay, M.E., Abbas, A., and Meininger R. (2007). “Effect of Coefficient of Thermal Expansion Test Variability on Concrete Pavement Performance as Predicted by Mechanistic-Empirical Pavement Design Guide.” Journal of the Transportation Research Board: 40-44

Transportation Research Board (TRB), Basic Research and Emerging Technologies Related to Concrete Committee (2006). “Control of Cracking in Concrete: State of the Art.: Transportation Research Circular E-C107.

Villarreal, V.H., and Crocker, D.A. (2007). Better Pavements through Internal Hydration, Concrete International, 29(2), 32-36.

Whiting, D. (1981). Rapid Measurement of the Chloride Permeability of Concrete. Public Roads, 45(3), 101-112.

Weber, S., and Reinhardt, H.W. (2003) Modeling the Internal Curing of High Strength Concrete Using Lightweight Aggregates, Theodore Bremner Symposium on High Performance Lightweight Concrete, Eds. J.P. Ries and T.A. Holm, 6th CANMET/ACI International Conference on Durability, Thessalonika, Greece, 45-64.

Wei, Y. Hansen, W. (2008). Pre-soaked Lightweight Fine Aggregates as Additives for Internal Curing in Concrete. SP-256-3. 34-43.

Weiss, J. (2017). Personal Communication with Cavalline, T and Leach, J.

Weiss, Jason. Bentz, Dale P. Schindler, Anton. Lura, Pietro. (2012). Construction Issues: Internal Curing, Constructing More Robust Concrete. Structure Magazine. 10-14. January.

APPENDIX A: SUPPLEMENTAL INFORMATION FOR CHAPTER 2

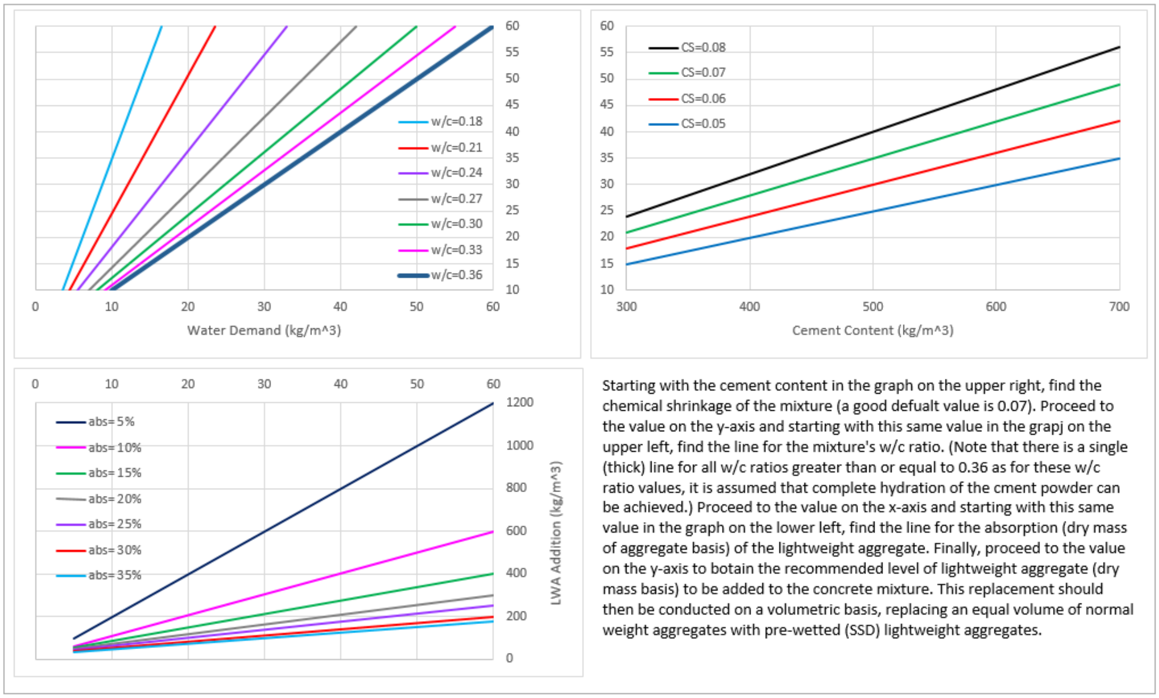


Figure A.1: Nomograph in SI units (Bentz 2009)

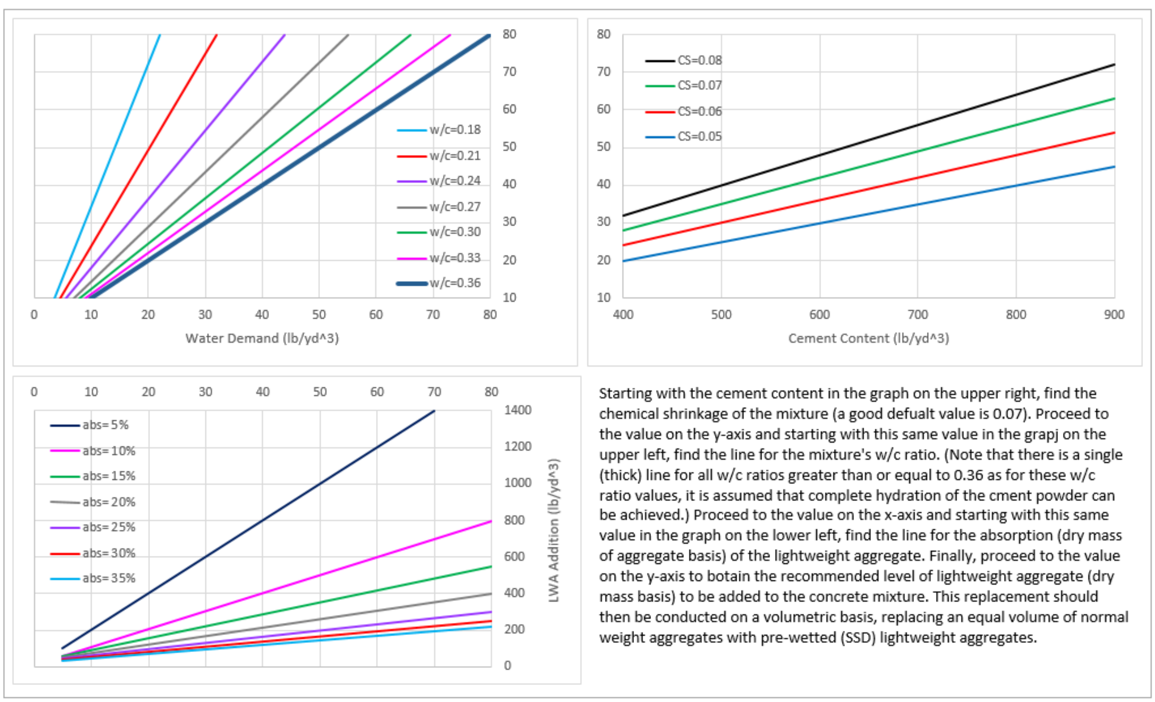


Figure A.2: Nomograph in English units (Bentz 2009)

APPENDIX B: SUPPLEMENTAL INFORMATION FOR CHAPTER 3

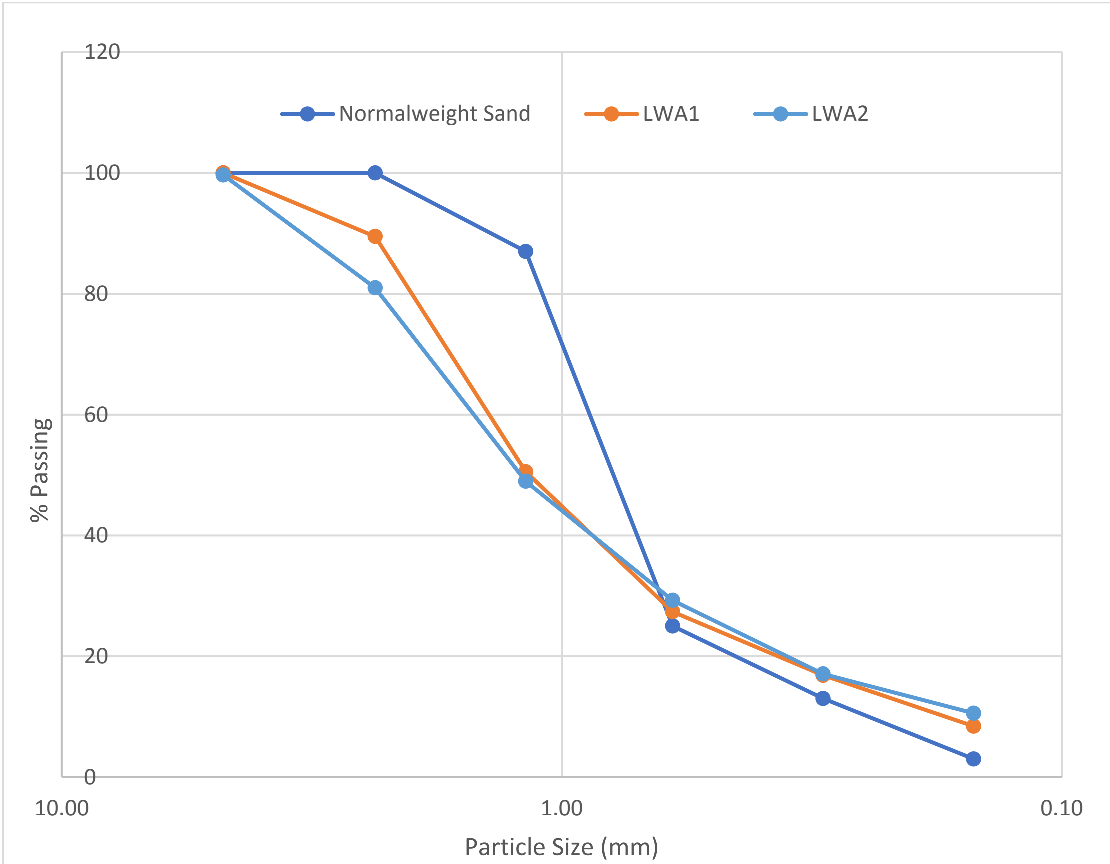


Figure B.1: Gradation curves for fine aggregates

Version 6.42

Material Certification Report

Portland Cement
I-II(MH)

Test Period: 14-Sep-2015
To: 15-Sep-2015

Certification

:ts the specifications of ASTM C150 for Type I-II(MH) cement,
ı AASHTO M85 specifications for Type I-II(MH) cement.

General Information

Supplier:
Address:

Telephone:
Date Issued:

Source Location:

Contact:

The following information is based on average test data during the test period.
The data is typical of cement shipped by Holcim; individual shipments may vary.

Tests Data on ASTM Standard Requirements

Chemical

Item	Limit[A]	Result
SiO_2 (%)	-	20.4
Al_2O_3 (%)	6.0 max	4.8
Fe_2O_3 (%)	6.0 max	3.3
CaO (%)	-	63.8
MgO (%)	6.0 max	1.6
SO_3 (%)	3.0 max[B]	3.1
Loss on Ignition (%)	3.0 max	1.6
Insoluble Residue (%)	0.75 max	0.23
CO_2 (%)	-	1.1
Limestone (%)	5.0 max	2.6
$CaCO_3$ in Limestone (%)	70 min	92
Inorganic Processing Addition (%)	5.0 max	0.0
Potential Phase Compositions[C]:		
C_3S (%)	-	54
C_2S (%)	-	17
C_3A (%)	8 max	7
C_4AF (%)	-	10
$C_3S + 4.75C_3A$ (%)	100 max	87.3

Physical

Item	Limit[A]	Result
Air Content (%)	12 max	6
Blaine Fineness (m^2/kg)	260-430	393
Autoclave Expansion (%) (C151)	0.80 max	0.05
Compressive Strength MPa (psi):		
3 days	10.0 (1450) min	29.0 (4210)
7 days	17.0 (2470) min	34.8 (5040)
Initial Vicat (minutes)	45-375	117
Mortar Bar Expansion (%) (C1038)		0.006
Heat of Hydration: kJ/kg (cal/g)[D] 7 Days (for informational purposes)	-	305 (73)

Tests Data on ASTM Optional Requirements

Chemical

Item	Limit[A]	Result
Equivalent Alkalies (%)	0.60 max	0.53

Physical

Item	Limit[A]	Result

Notes

[A] Dashes in the limit / result columns mean Not Applicable.
[B] It is permissible to exceed the specification limit provided that ASTM C1038 Mortar Bar Expansion does not exceed 0.020 % at 14 days.
[C] Adjusted per Annex A1.6 of ASTM C150 and AASHTO M85.
[D] Test result represents most recent value and is provided for information only. Analysis of Heat of Hydration has been carried out by CTLGroup, Skokie, IL.
This data may have been reported on previous mill certificates.

Silo 18
9/14/2015
Grind 257-259

Additional Data

Inorganic Processing Addition Data

Item	Result[A]
Type	-
Amount (%)	-
SiO_2 (%)	-
Al_2O_3 (%)	-
Fe_2O3 (%)	-
CaO (%)	-
SO_3 (%)	-

Base Cement Phase Composition

Item	Result
C_3S (%)	56
C_2S (%)	18
C_3A (%)	7
C_4AF (%)	10

By

Figure B.2: Cement mill report

Date: February 10, 2016
I.D.: ______
Lab No.: ______

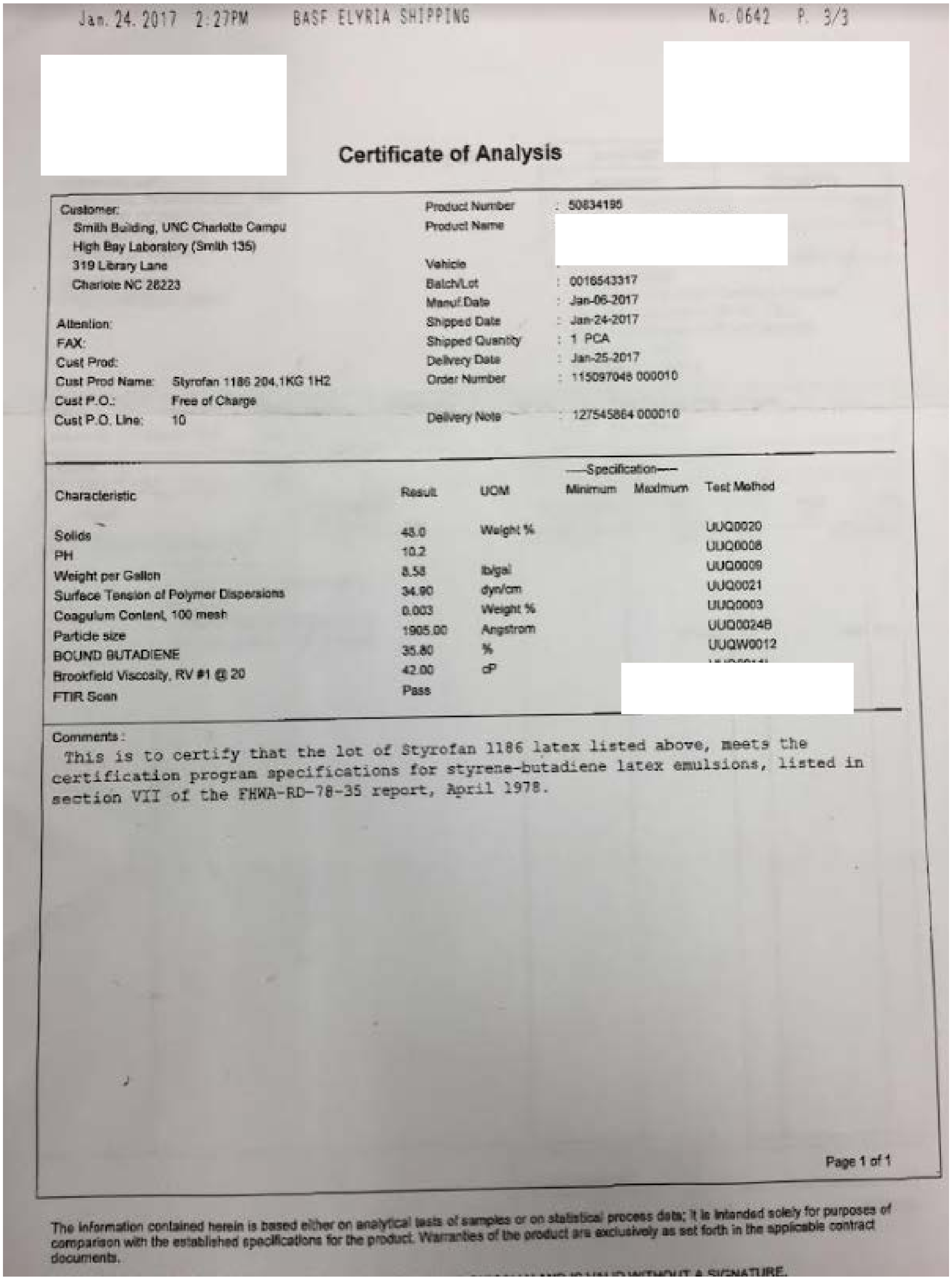
REPORT OF FLY ASH TESTS

Date Sampled:	DS 11/23-12/11	Start Date:	November 23, 2015
Manufacturer:	Roxboro	End Date:	December 11, 2015
		Date Received:	December 16, 2015

Chemical Analysis**	Results (wt%)	Specification (Class F) ASTM C618-15	AASHTO M295-11
Silicon Dioxide (SiO_2)	**53.8**	----	----
Aluminum Oxide (Al_2O_3)	**27.5**	---	----
Iron Oxide (Fe_2O_3)	**8.05**	----	----
Sum of Silicon Dioxide, Iron Oxide & Aluminum Oxide (SiO_2+Al_2O_3+Fe_2O_3)	**89.3**	70 % min.	70 % min.
Calcium Oxide (CaO)	**2.3**	----	----
Magnesium Oxide (MgO)	**1.0**	----	----
Sodium Oxide (Na_2O)	**0.45**	----	----
Potassium Oxide (K_2O)	**2.44**	---	----
"Sodium Oxide Equivalent (Na_2O+0.658K_2O)"	**2.05**	----	----
Sulfur Trioxide (SO_3)	**0.62**	5 % max.	5 % max.
Loss on Ignition	**2.1**	6 % max.	5 % max.
Moisture Content	**0.18**	3 % max.	3 % max.
Available Alkalies**			
Sodium Oxide (Na_2O) as Available Alkalies	**0.16**	----	----
Potassium Oxide (K_2O) as Available Alkalies	**0.71**	----	----
Available Alkalies as "Sodium Oxide Equivalent (Na_2O+0.658K_2O)"	**0.63**	----	1.5 % max.*
Physical Analysis			
Fineness (Amount Retained on #325 Sieve)	**21.9%**	34 % max.	34 % max.
Strength Activity Index with Portland Cement			
At 7 Days: Control Average, psi: 4820; Test Average, psi: 3780	**78%**	75 % min.† (of control)	75 % min.† (of control)
At 28 Days: Control Average, psi: 6100; Test Average, psi: 5190	**85%**	75 % min.† (of control)	75 % min.† (of control)
Water Requirements (Test H_2O/Control H_2O) Control, mls: 242; Test, mls: 236	**98%**	105 % max. (of control)	105 % max. (of control)
Autoclave Expansion:	**-0.03%**	± 0.8 % max.	± 0.8 % max.
Specific Gravity:	**2.21**	----	----

† Meeting the 7 day or 28 day strength activity index will indicate specification compliance
* Optional
**Chemical Analysis performed by

Figure B.3: Fly ash report



Jan. 24. 2017 2:27PM BASF ELYRIA SHIPPING No. 0642 P. 3/3

Certificate of Analysis

Customer:
Smith Building, UNC Charlotte Campu
High Bay Laboratory (Smith 135)
319 Library Lane
Charlote NC 28223

Attention:
FAX:
Cust Prod:
Cust Prod Name: Styrofan 1186 204,1KG 1H2
Cust P.O.: Free of Charge
Cust P.O. Line: 10

Product Number: 50834195
Product Name:
Vehicle:
Batch/Lot: 0016543317
Manuf Date: Jan-06-2017
Shipped Date: Jan-24-2017
Shipped Quantity: 1 PCA
Delivery Date: Jan-25-2017
Order Number: 115097048 000010
Delivery Note: 127545864 000010

Characteristic	Result	UOM	Specification Minimum	Specification Maximum	Test Method
Solids	48.0	Weight %			UUQ0020
PH	10.2				UUQ0008
Weight per Gallon	8.58	lb/gal			UUQ0009
Surface Tension of Polymer Dispersions	34.90	dyn/cm			UUQ0021
Coagulum Content, 100 mesh	0.003	Weight %			UUQ0003
Particle size	1905.00	Angstrom			UUQ0024B
BOUND BUTADIENE	35.80	%			UUQW0012
Brookfield Viscosity, RV #1 @ 20	42.00	cP			
FTIR Scan	Pass				

Comments:
This is to certify that the lot of Styrofan 1186 latex listed above, meets the certification program specifications for styrene-butadiene latex emulsions, listed in section VII of the FHWA-RD-78-35 report, April 1978.

Page 1 of 1

The information contained herein is based either on analytical tests of samples or on statistical process data; it is intended solely for purposes of comparison with the established specifications for the product. Warranties of the product are exclusively as set forth in the applicable contract documents.

Figure B.4: Latex polymer report

APPENDIX D: SUPPLEMENTAL INFORMATION FOR CHAPTER 5

Table D.1: Compiled 28 day compressive strength results

Designation	28 day Compressive Strength (psi)			Average Compressive Strength (psi)	Standard Deviation
	1	2	3		
CC	7,842	7,604	7,913	7,786	162
I1M	5,356	5,290	5,107	5,251	129
I2M	7,139	7,003	6,650	6,931	252
I1H	5,036	5,667	5,613	5,439	350
I2H	5,309	4,657	4,847	4,938	335
CF	4,956	4,625	5,248	4,943	312
I1MF	5,111	5,217	5,413	5,247	153
I2MF	5,391	5,382	5,904	5,559	299
I1HF	5,518	5,007	5,148	5,224	264
I2HF	5,545	5,543	5,415	5,501	74
IP	4,784	4,310	4,867	4,654	301
P.B.N.N	4,220	4,458	4,484	4,387	145
CLMC	7,023	7,436	N/A	7,230	292
ILMC	5,008	5,426	N/A	5,217	296

Table D.2: Compiled 28 day modulus of elasticity results

Designation	28 day Modulus of Elasticity (psi)		Average Modulus of Elasticity (psi)	Standard Deviation
	1	2		
CC	4,512,810	4,265,648	4,389,229	174,770
I1M	3,185,763	3,198,571	3,192,167	9,057
I2M	3,172,954	3,075,877	3,124,416	68,644
I1H	2,963,253	3,180,091	3,071,672	153,328
I2H	2,926,487	3,309,295	3,117,891	270,686
CF	3,241,809	3,616,904	3,429,357	265,232
I1MF	4,000,631	3,364,158	3,682,395	450,054
I2MF	3,144,204	2,947,625	3,045,915	139,002
I1HF	3,515,275	3,059,639	3,287,457	322,183
I2HF	3,414,691	3,315,429	3,365,060	70,189
IP	2,218,806	2,398,764	2,308,785	127,250
P.B.N.N	2,919,808	4,109,804	3,514,806	841,454
CLMC	3,528,130	3,773,139	3,650,635	173,248
ILMC	3,940,835	3,670,950	3,805,893	190,838

Table D.3: Compiled 28 day Poisson's ratio results

Designation	28 day Poisson's Ratio (psi)		Average Poisson's Ratio (psi)	Standard Deviation
	1	2		
CC	0.19	0.21	0.20	0.01
I1M	0.26	0.26	0.26	0.00
I2M	0.22	0.22	0.22	0.00
I1H	0.19	0.24	0.22	0.04
I2H	0.21	0.22	0.22	0.01
CF	0.23	0.23	0.23	0.00
I1MF	0.23	0.24	0.24	0.01
I2MF	0.22	0.26	0.24	0.03
I1HF	0.23	0.21	0.22	0.01
I2HF	0.21	0.22	0.22	0.01
IP	0.21	0.20	0.21	0.01
P.B.N.N	0.18	0.20	0.19	0.01
CLMC	0.23	0.22	0.23	0.01
ILMC	0.25	0.24	0.25	0.01

Table D.4: Compiled 28 day CTE results

Designation	28 day CTE (x 10^{-6} in/in°F)			Average CTE (x 10^{-6} in/in°F)	Standard Deviation
	1	2	3		
CC	5.83	5.81	6.06	5.90	0.14
I1M	5.50	5.45	5.68	5.54	0.12
I2M	5.41	5.41	5.55	5.46	0.08
I1H	5.16	5.24	5.27	5.22	0.06
I2H	5.32	5.35	5.55	5.41	0.13
CF	5.40	5.40	5.47	5.42	0.04
I1MF	5.12	5.22	5.44	5.26	0.16
I2MF	5.18	5.23	5.45	5.29	0.14
I1HF	5.01	5.09	5.16	5.09	0.08
I2HF	4.96	5.00	5.21	5.06	0.13
IP	4.95	5.07	5.00	5.01	0.06
P.B.N.N	5.31	5.29	5.33	5.31	0.02
CLMC	6.65	6.71	6.82	6.73	0.09
ILMC	5.86	5.97	6.06	5.96	0.10

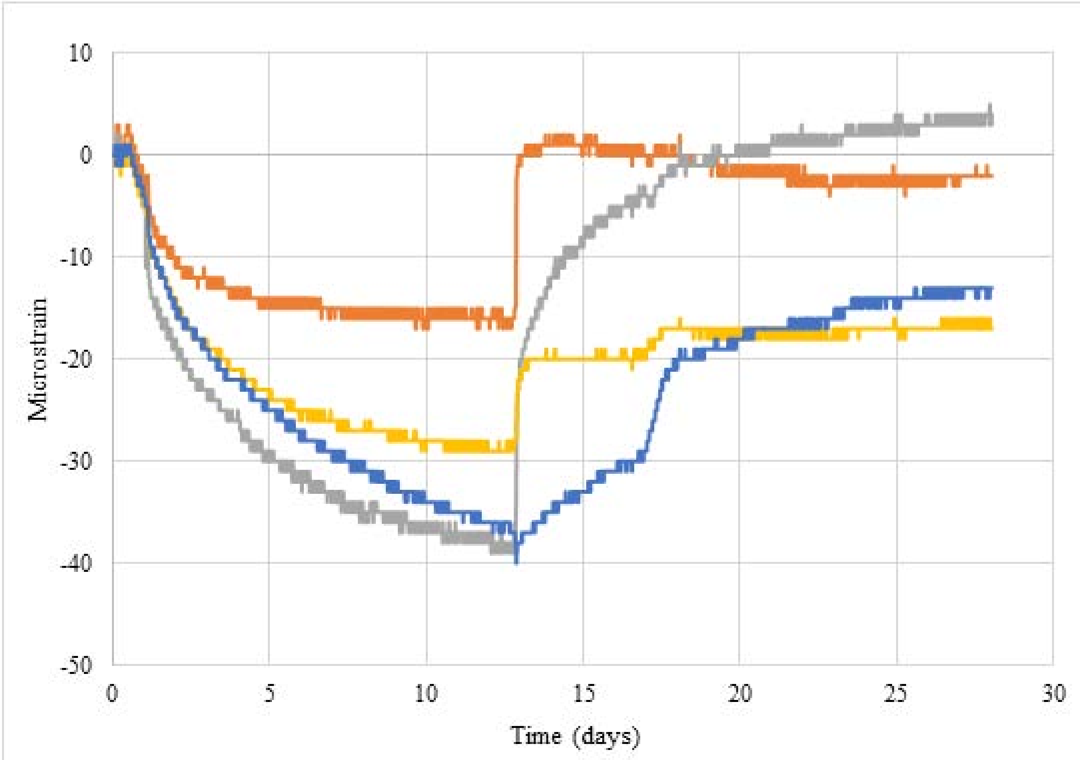


Figure D.1: Microstrain vs. time CC3 Specimen 1

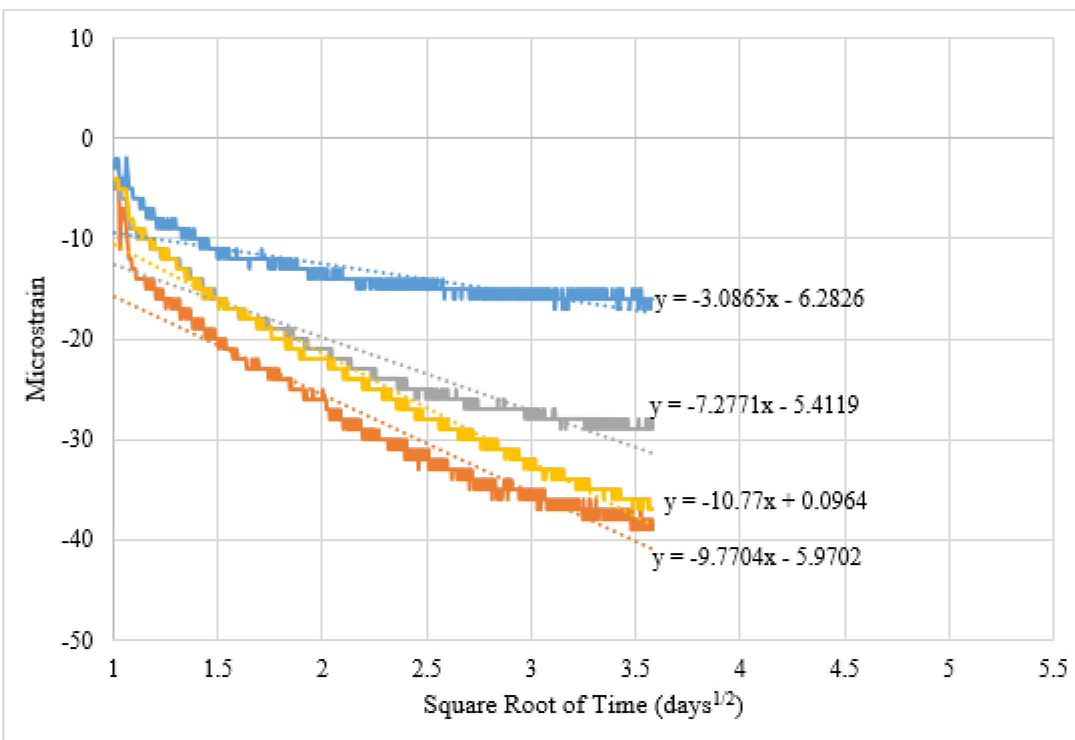


Figure D.2: Microstrain vs. square root of time CC3 Specimen 1

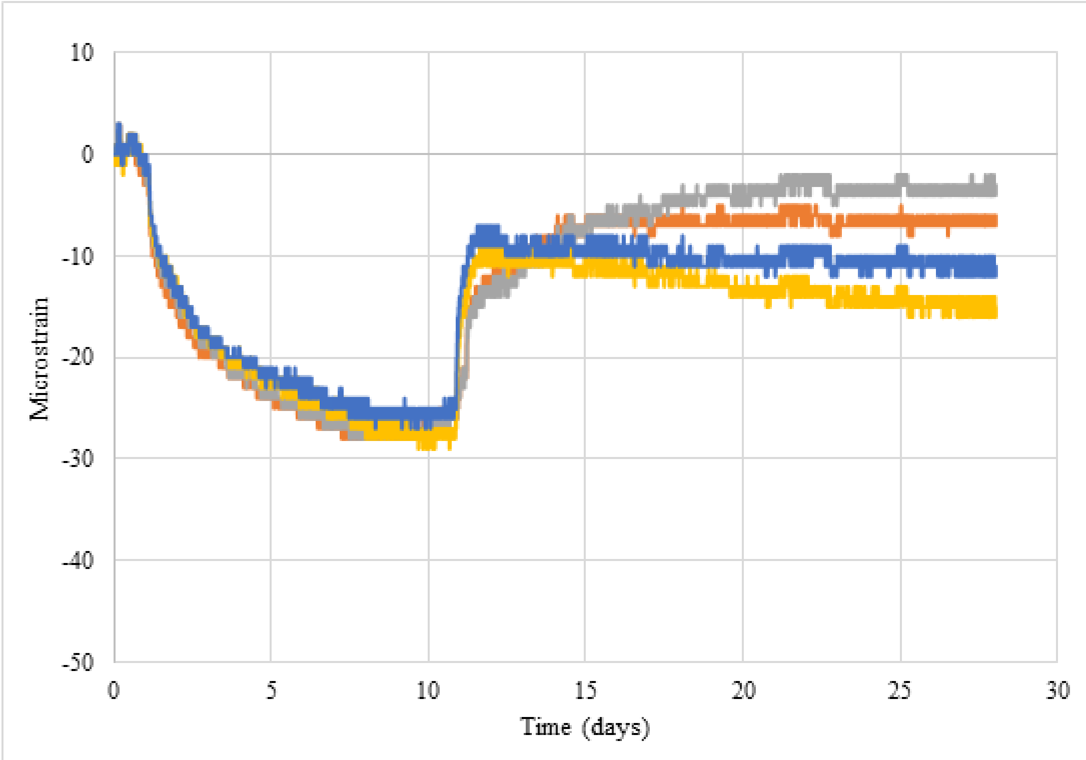


Figure D.3: Microstrain vs. time CC3 Specimen 2

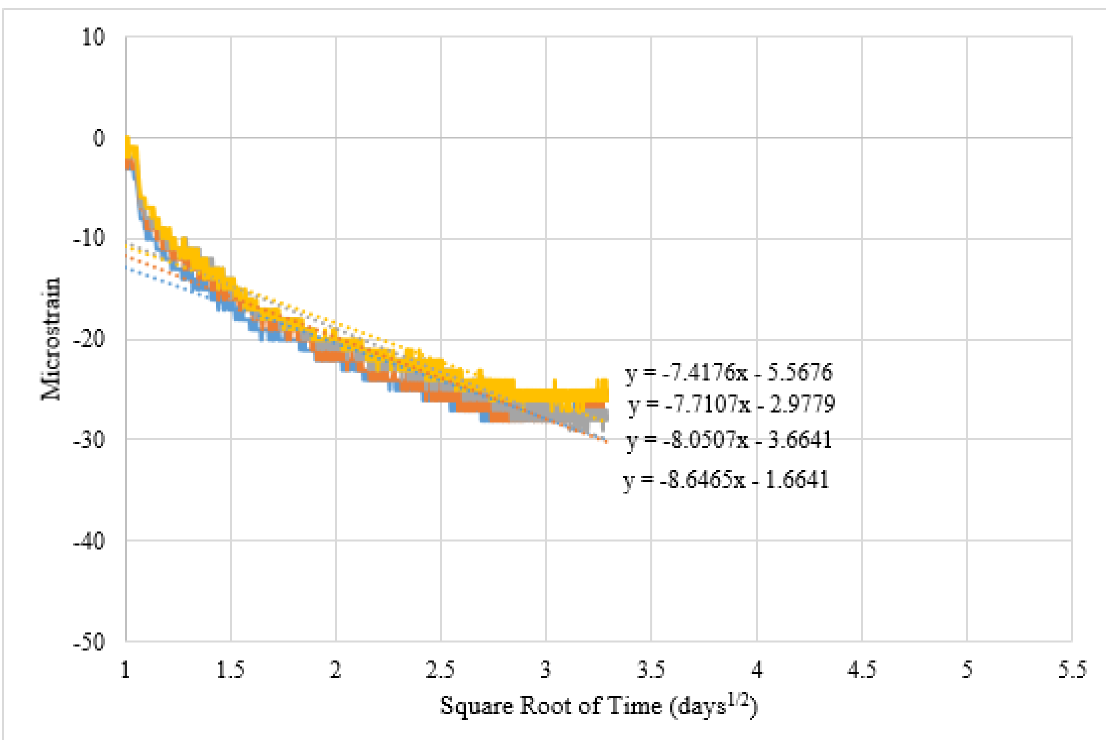


Figure D.4: Microstrain vs. square root of time CC3 Specimen 2

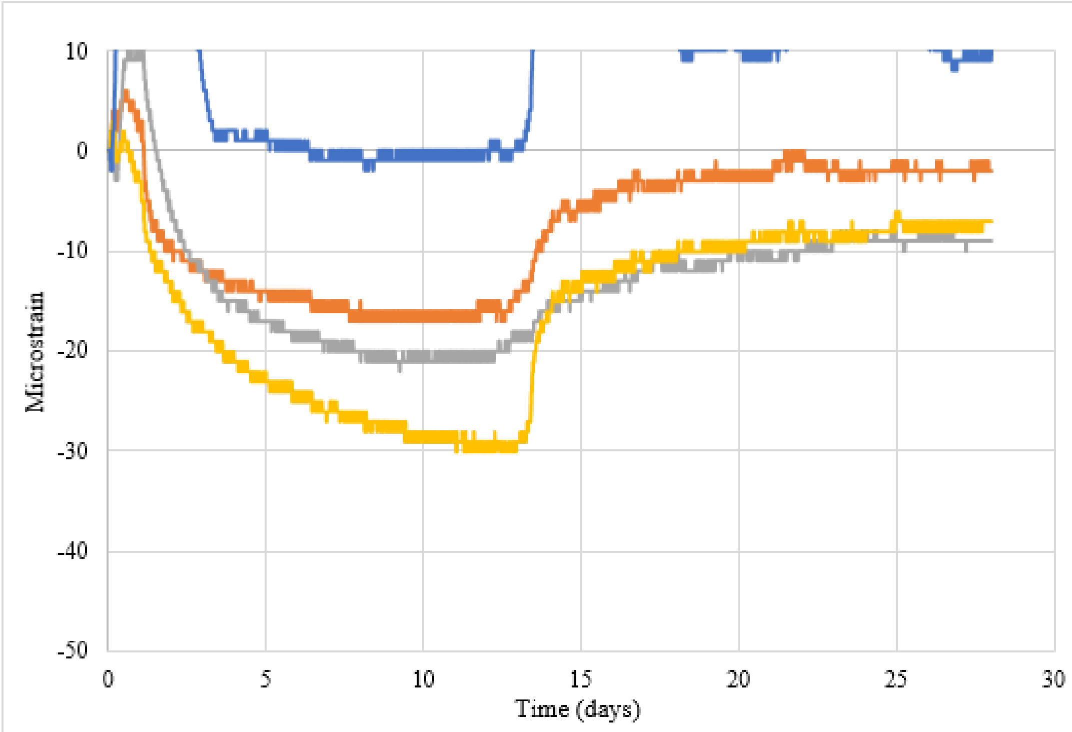


Figure D.5: Microstrain vs. time CC3 Specimen 3

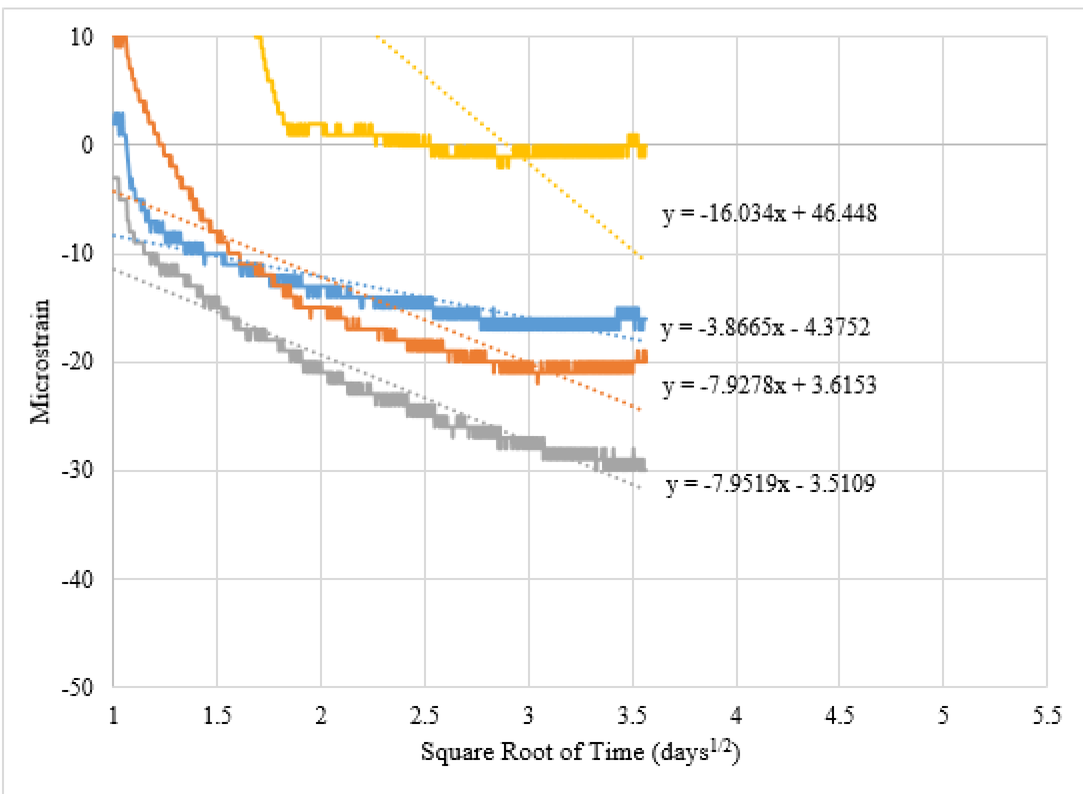


Figure D.6: Microstrain vs. square root of time CC3 Specimen 3

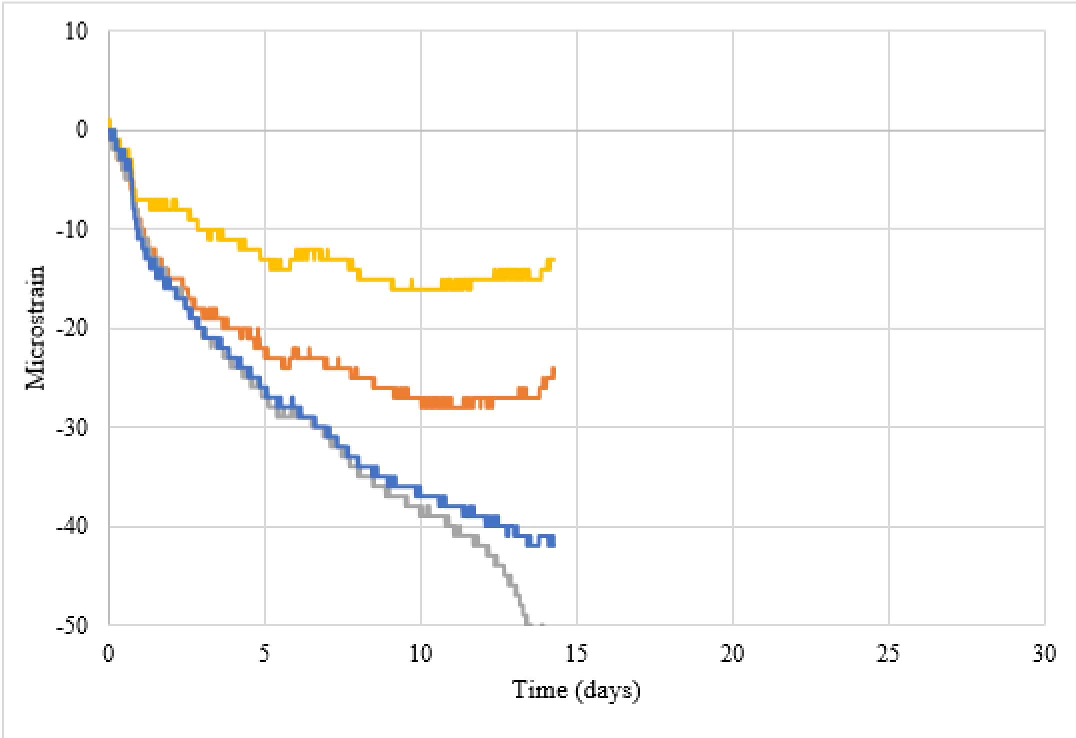


Figure D.7: Microstrain vs. time I1M3 Specimen 1

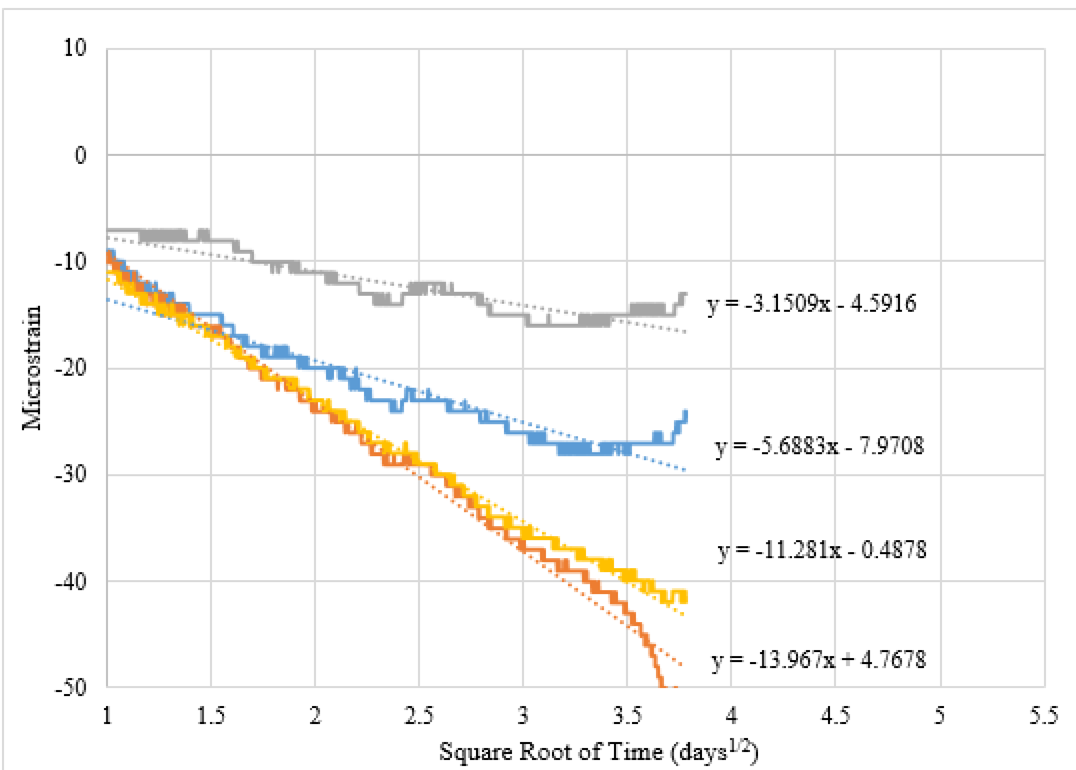


Figure D.8: Microstrain vs. square root of time I1M3 Specimen 1

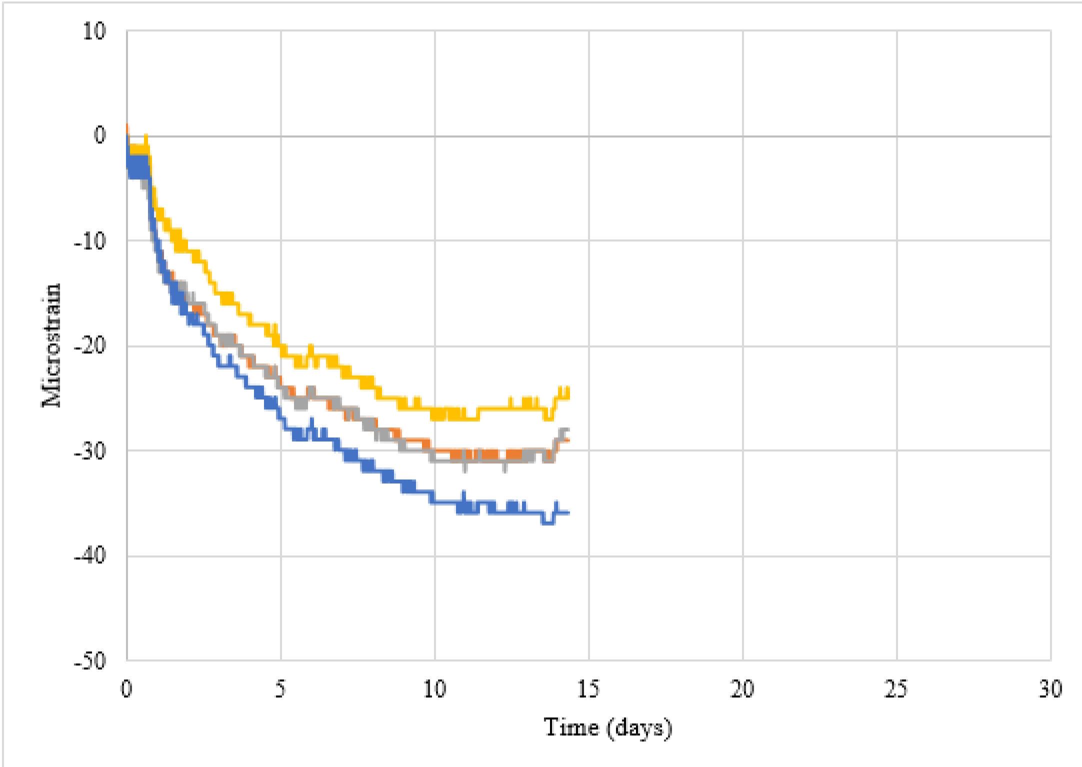


Figure D.9: Microstrain vs. time I1M3 Specimen 2

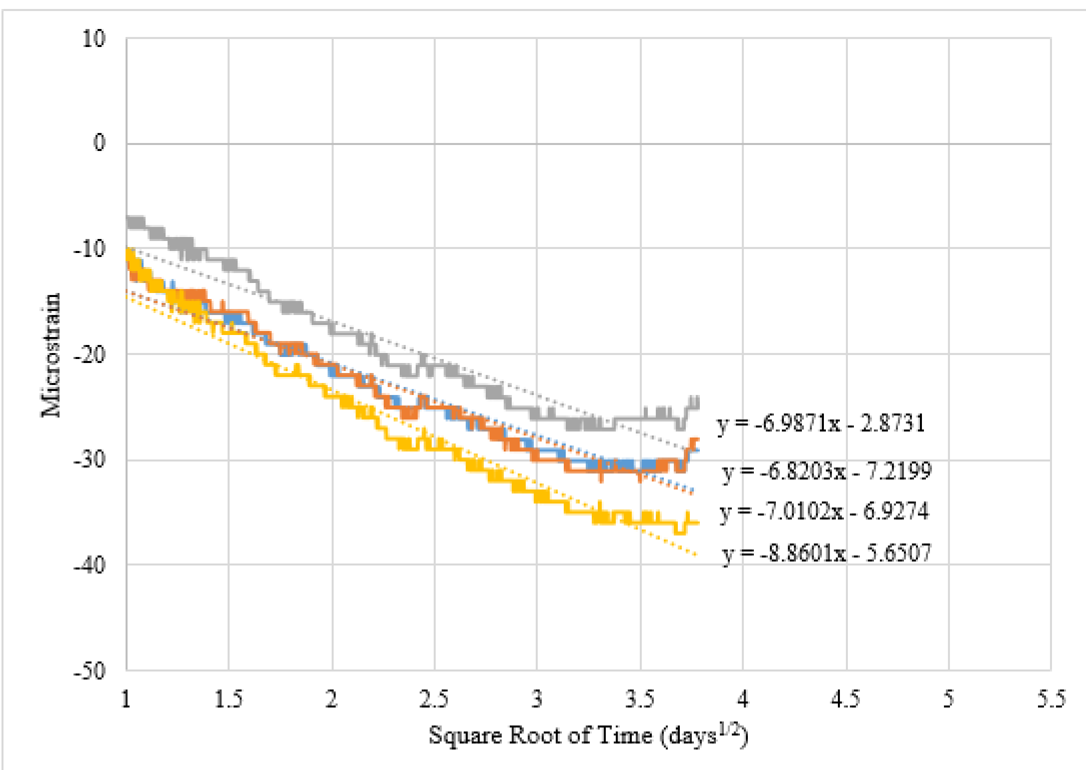


Figure D.10: Microstrain vs. square root of time I1M3 Specimen 2

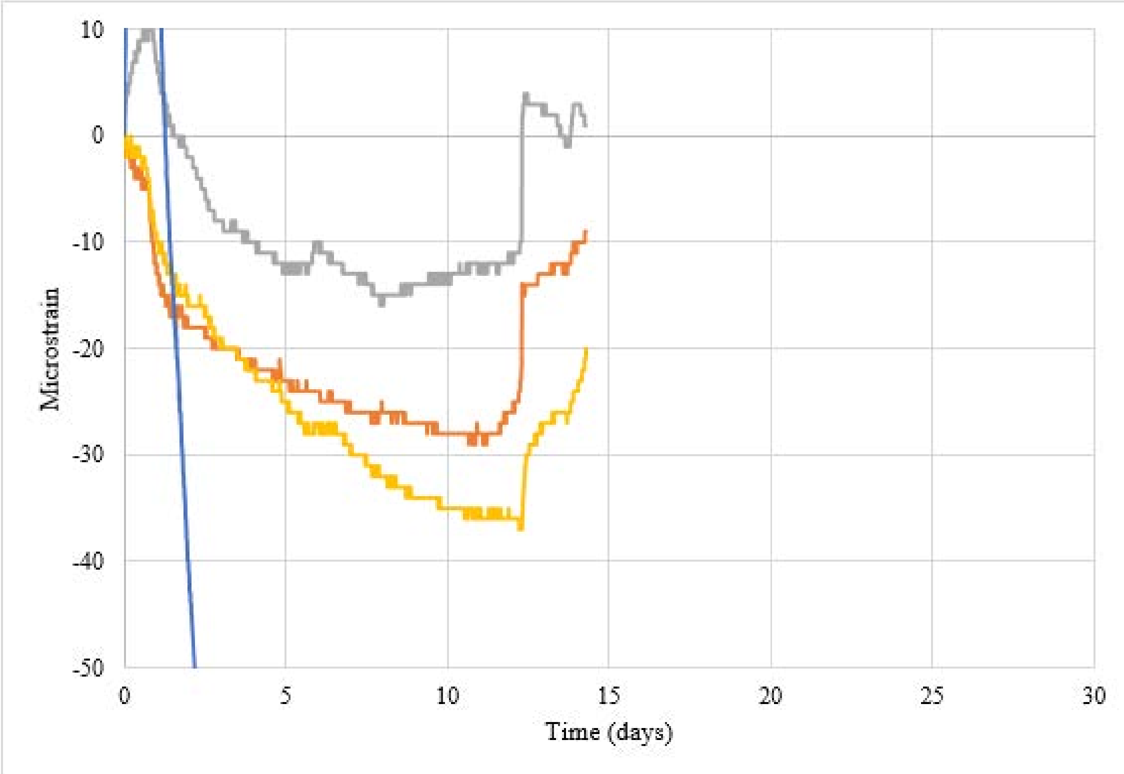


Figure D.11: Microstrain vs. time I1M3 Specimen 3

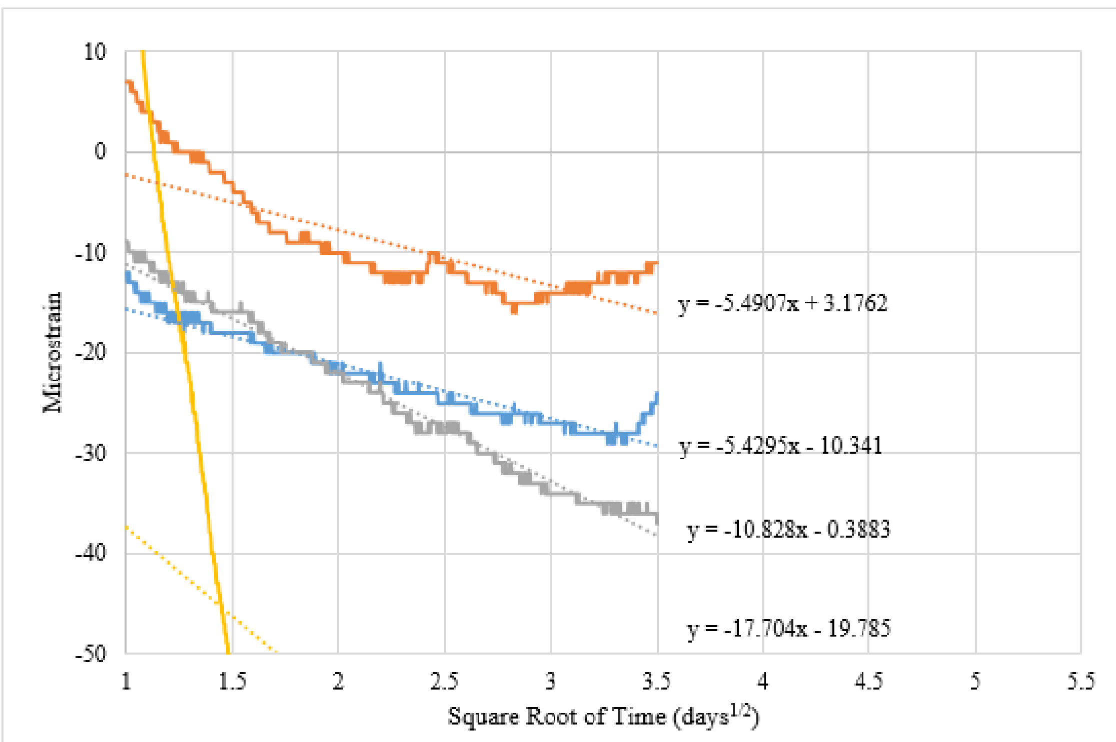


Figure D.12: Microstrain vs. square root of time I1M3 Specimen 3

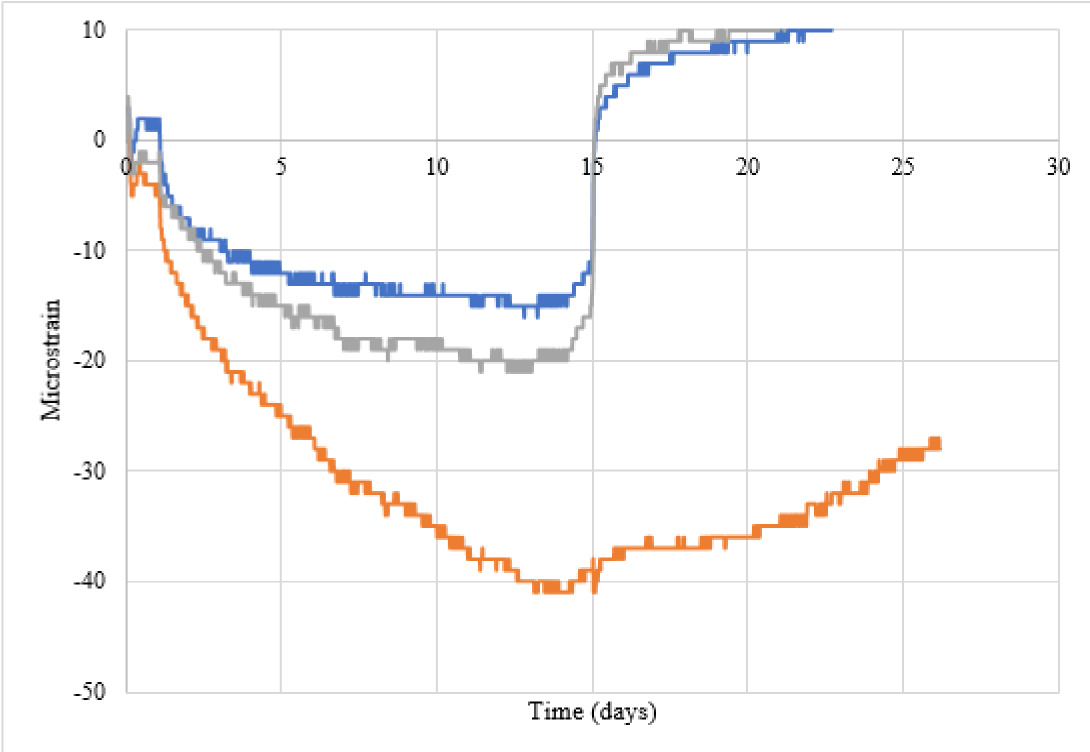


Figure D.13: Microstrain vs. time I2M3 Specimen 1

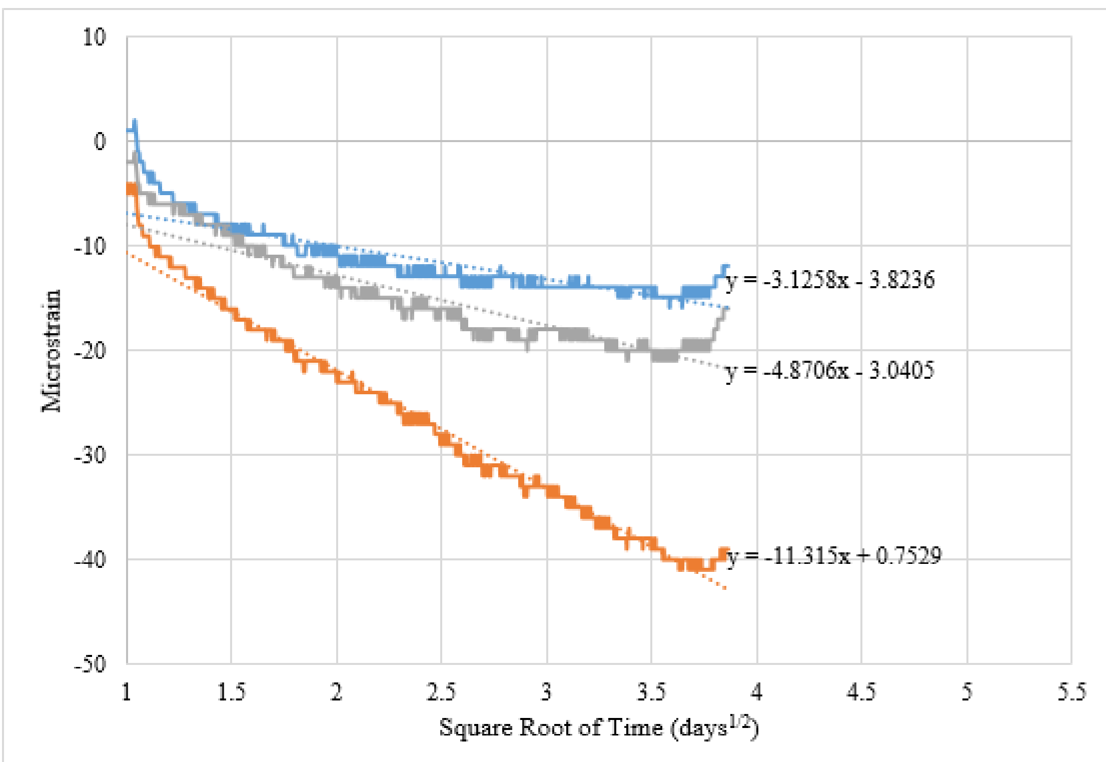


Figure D.14: Microstrain vs. square root of time I2M3 Specimen 1

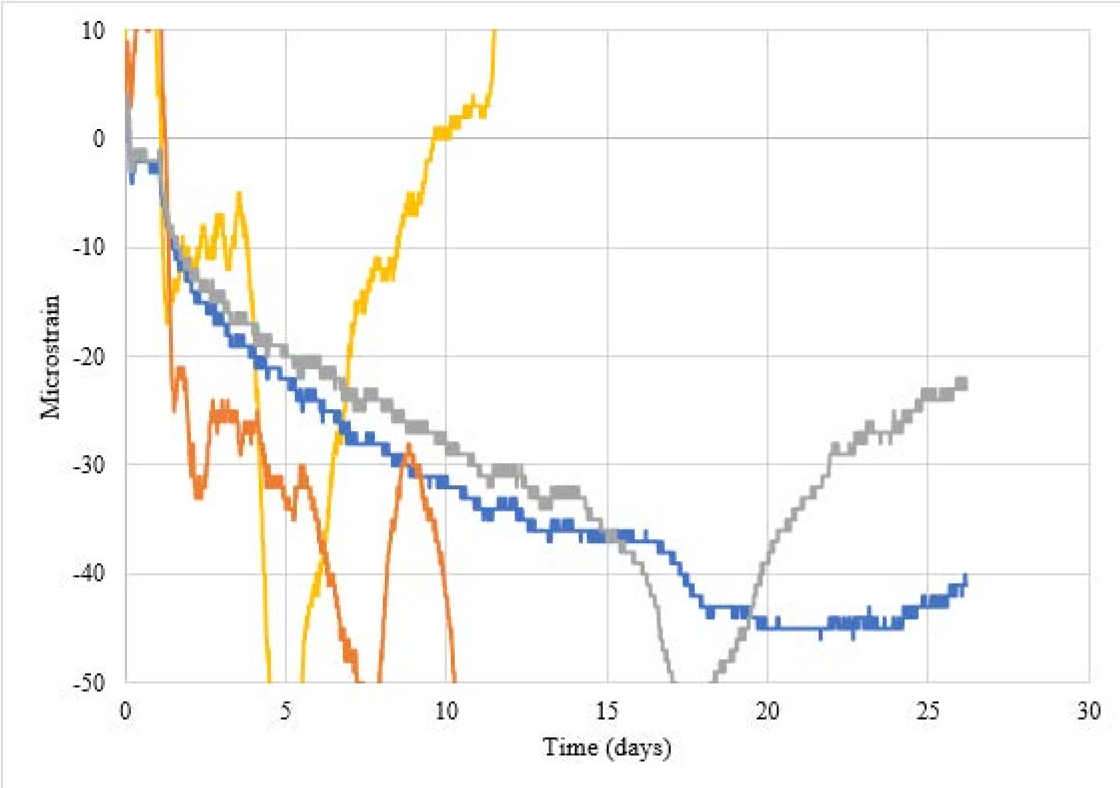


Figure D.15: Microstrain vs. time I2M3 Specimen 2

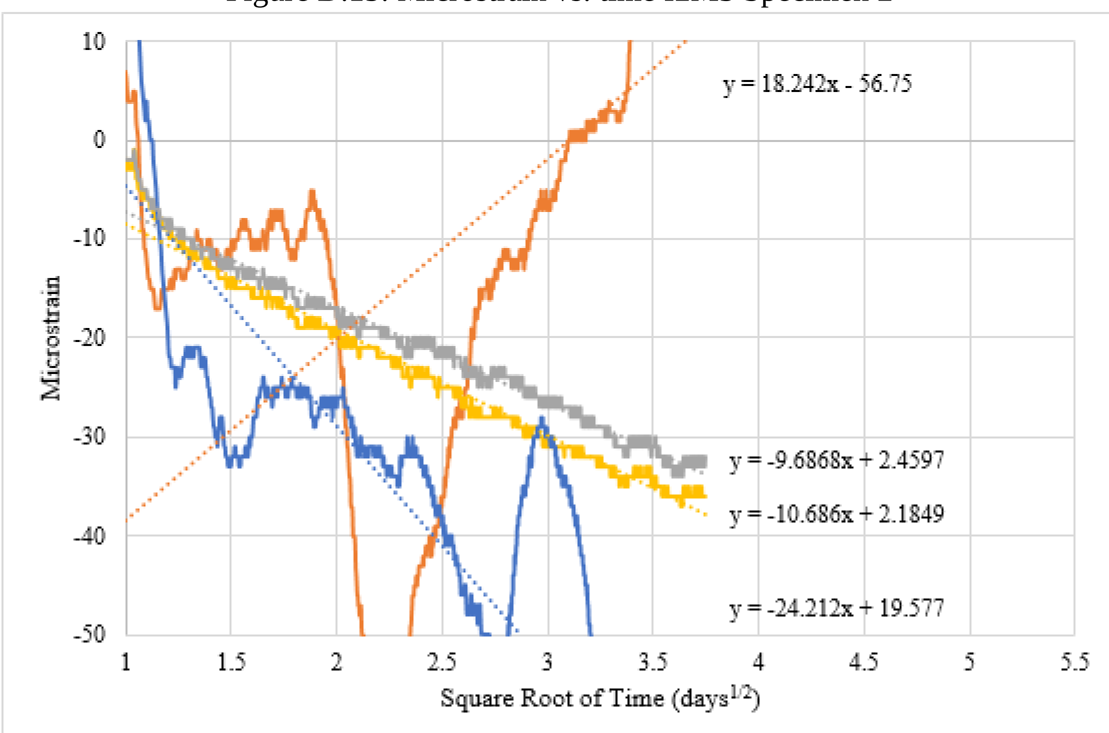


Figure D.16: Microstrain vs. square root of time I2M3 Specimen 2

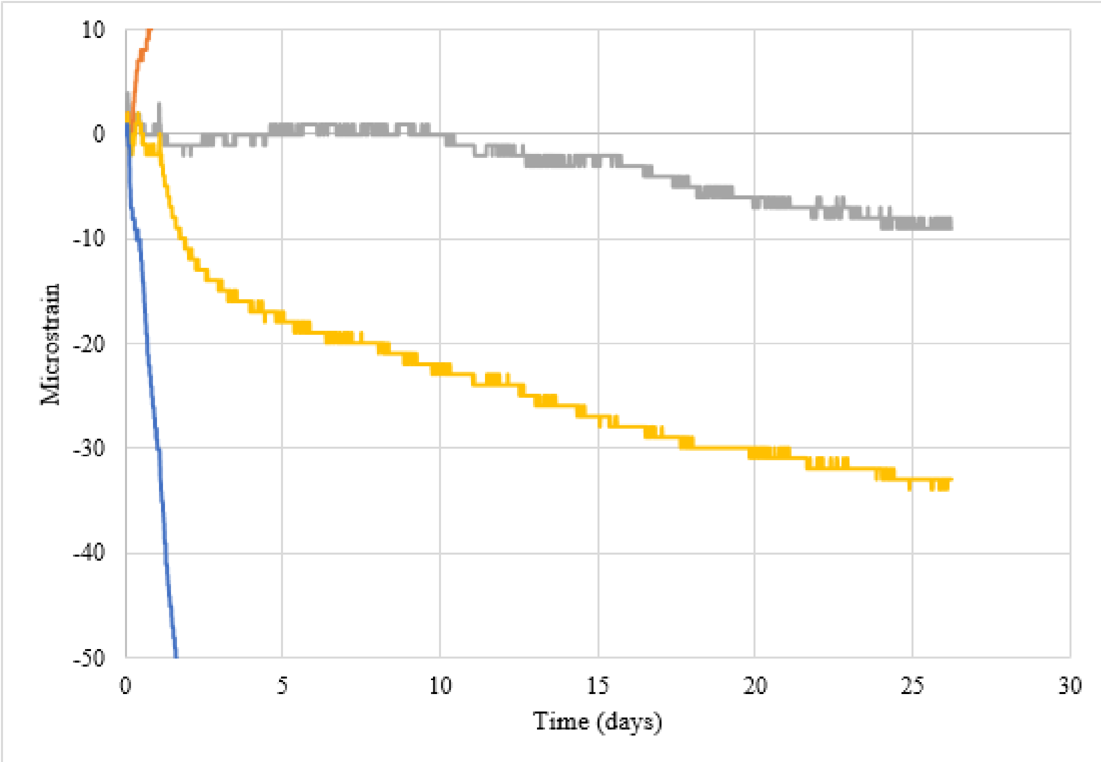


Figure D.17: Microstrain vs. time I2M3 Specimen 3

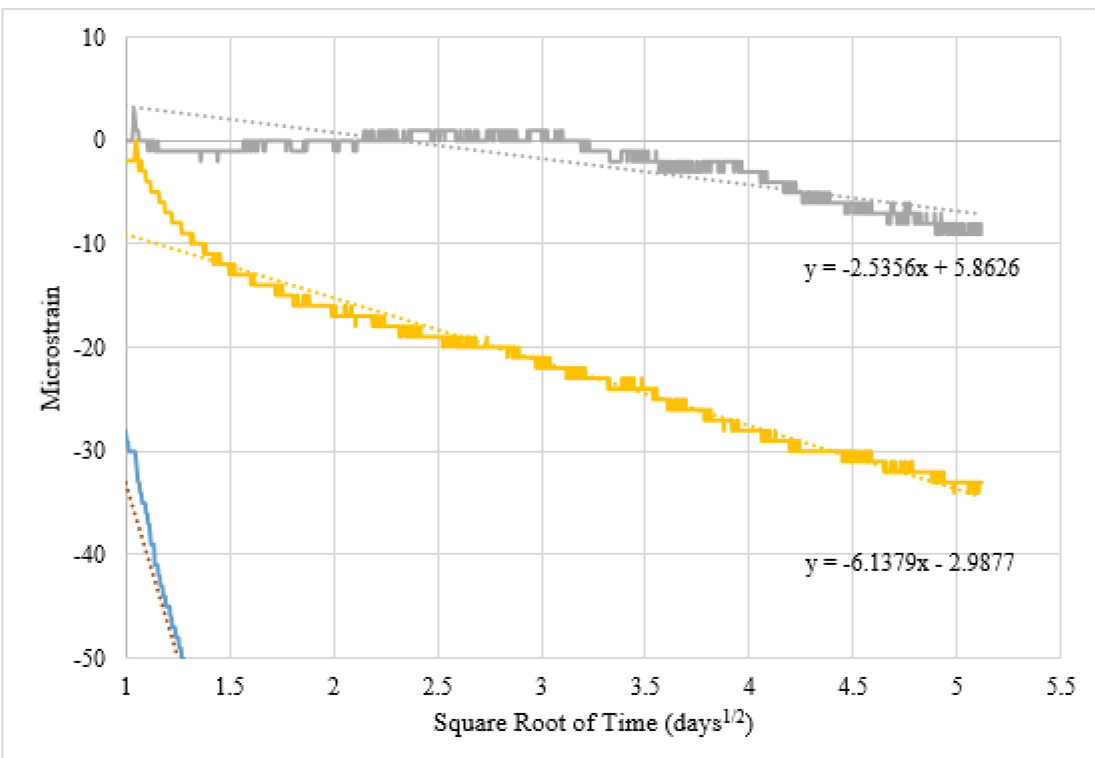


Figure D.18: Microstrain vs. square root of time I2M3 Specimen 3

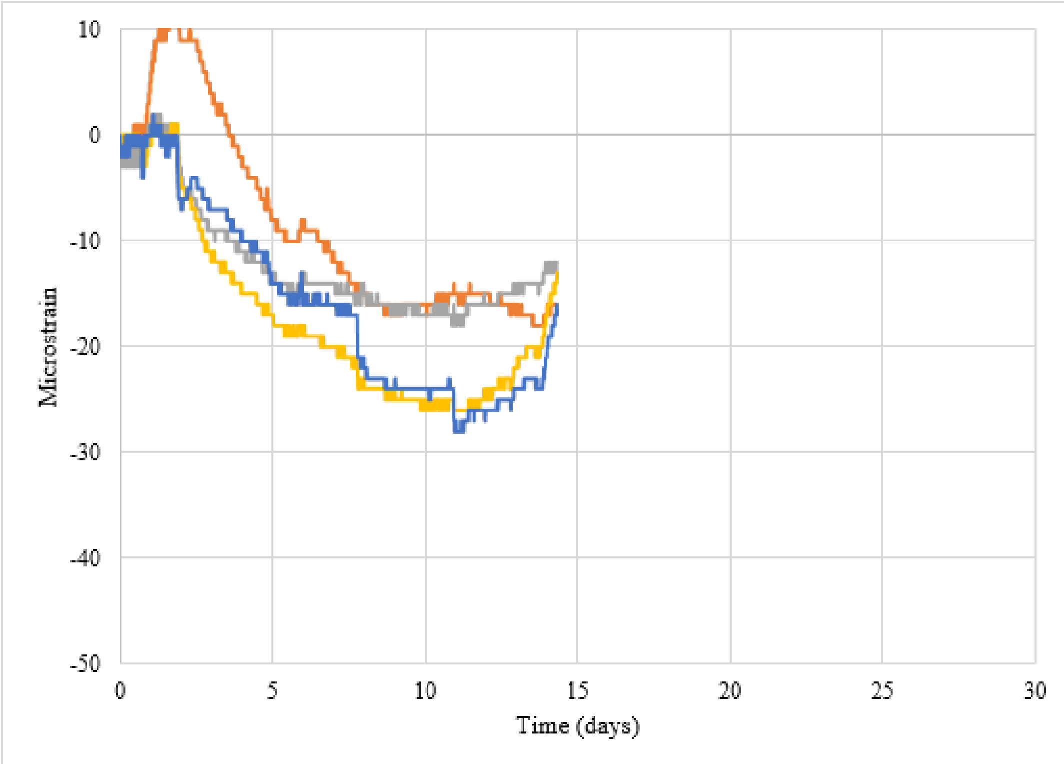


Figure D.19: Microstrain vs. time I1H3 Specimen 1

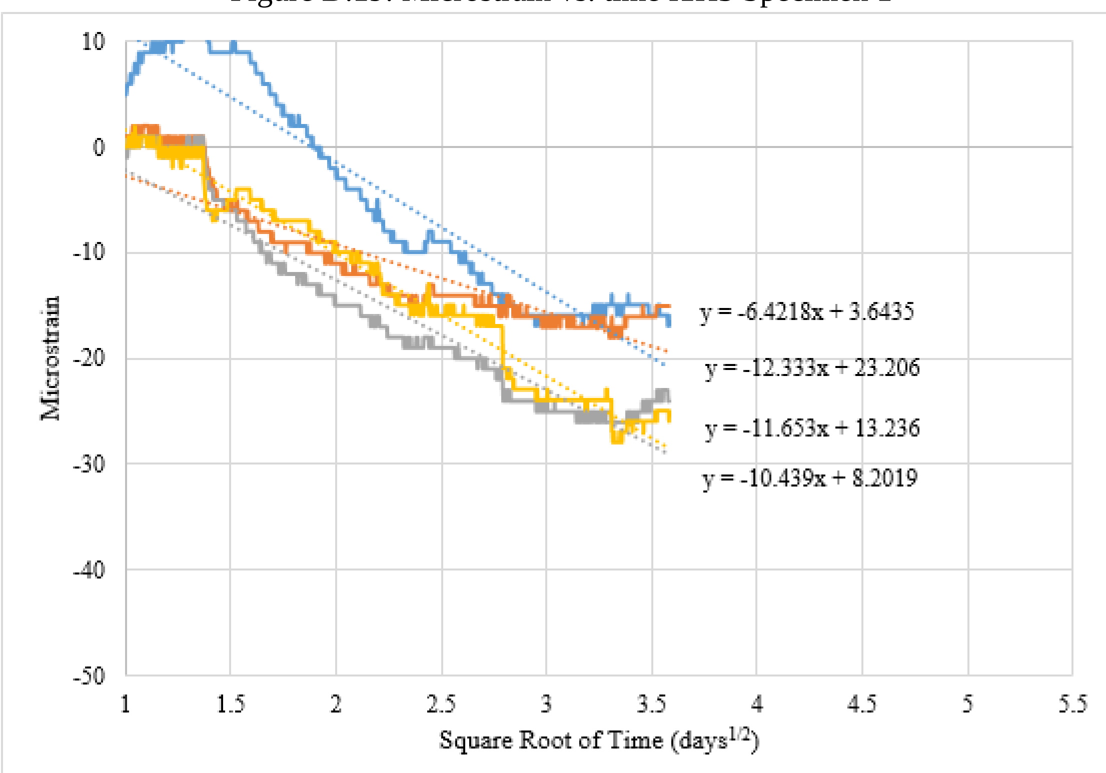


Figure D.20: Microstrain vs. square root of time I1H3 Specimen 1

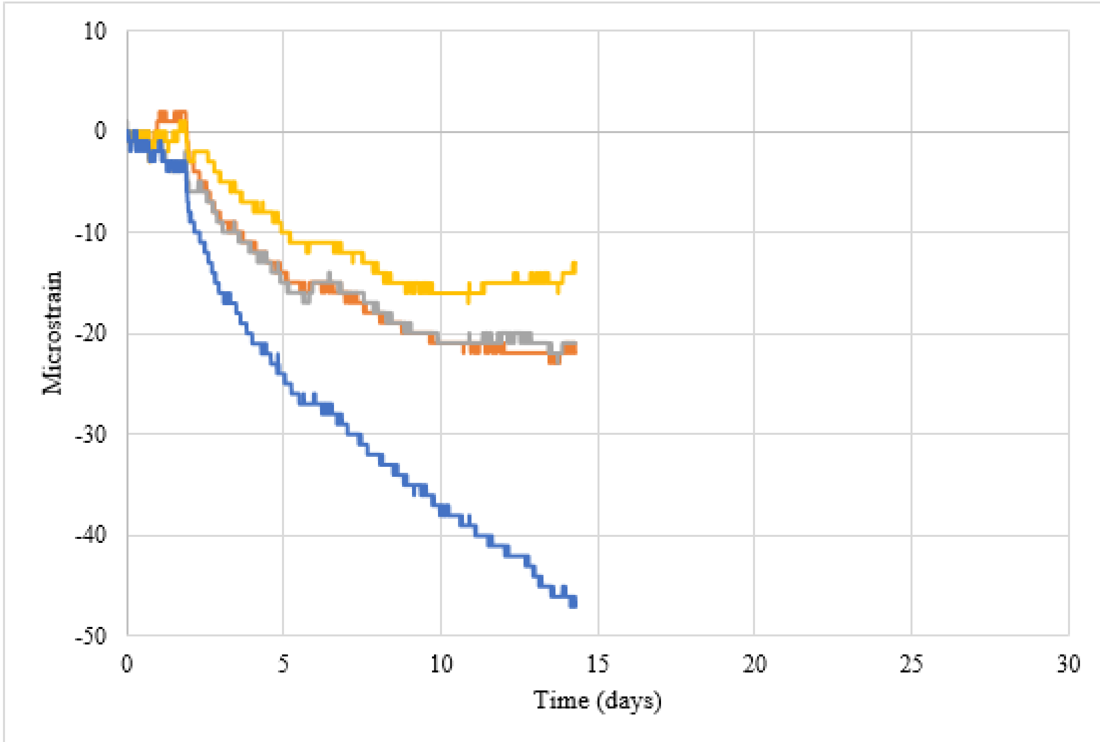


Figure D.21: Microstrain vs. time I1H3 Specimen 2

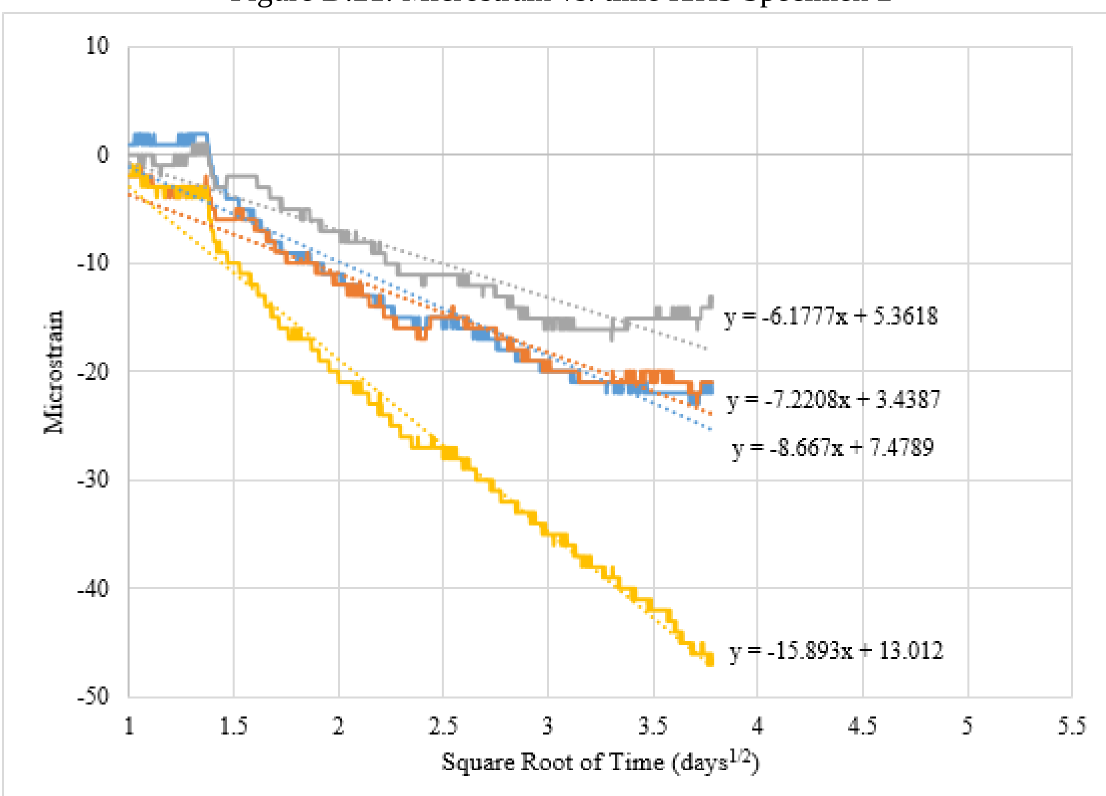


Figure D.22: Microstrain vs. square root of time I1H3 Specimen 2

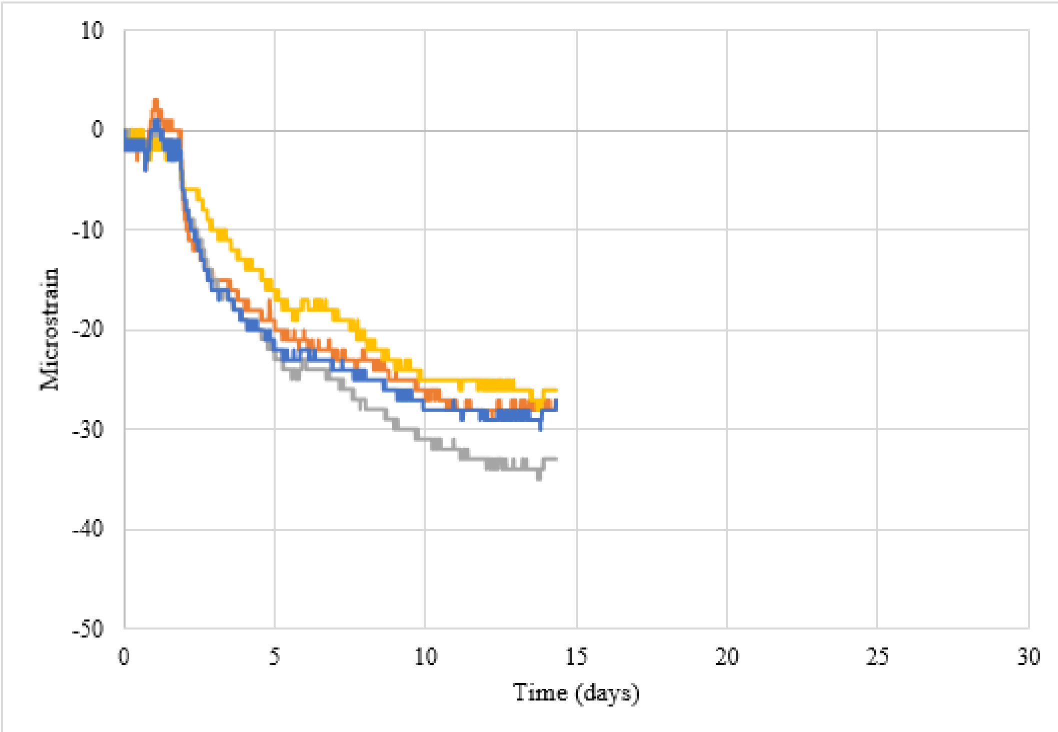


Figure D.23: Microstrain vs. time I1H3 Specimen 3

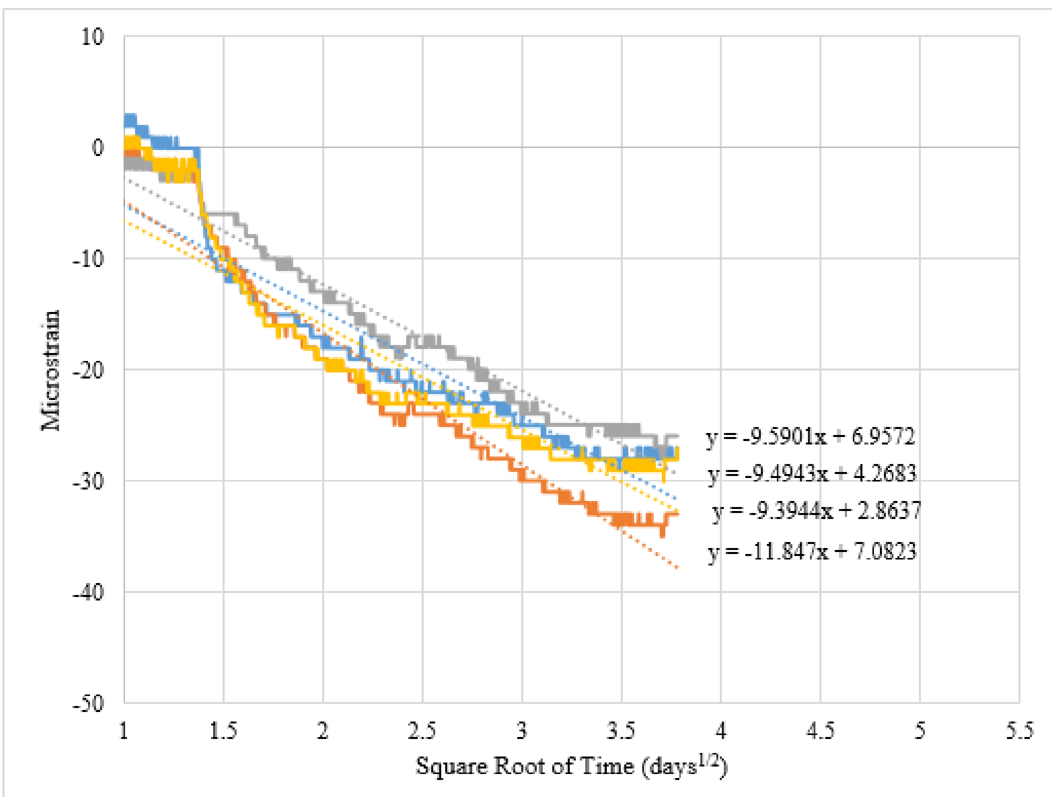


Figure D.24: Microstrain vs. square root of time I1H3 Specimen 3

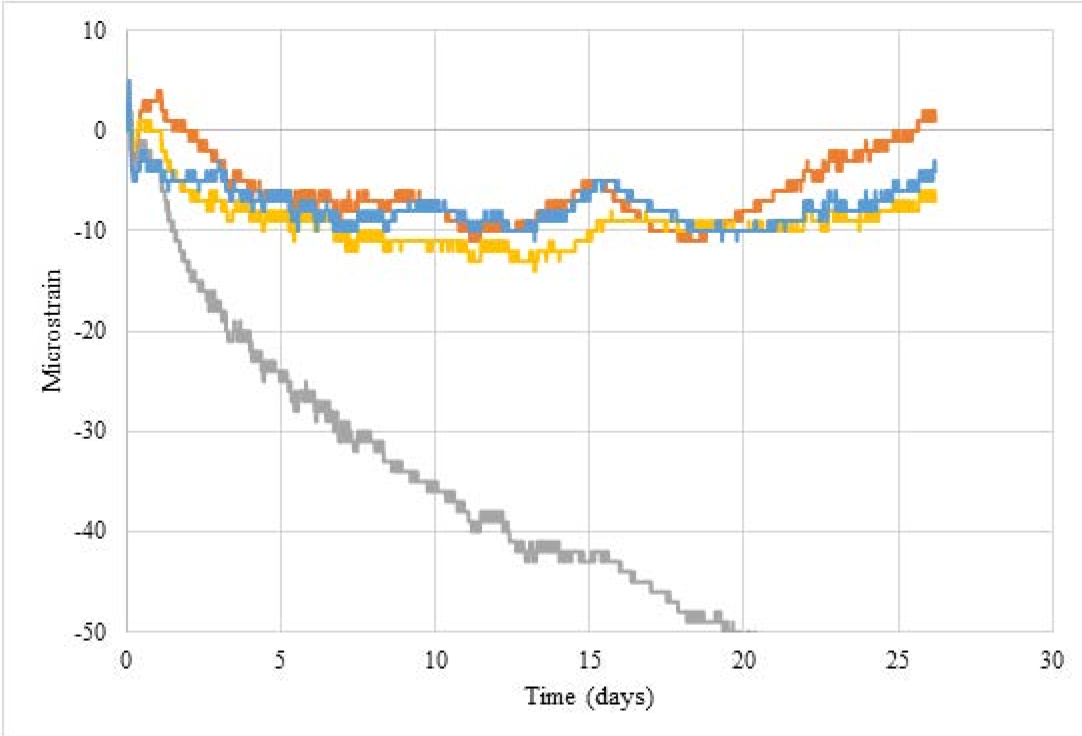


Figure D.25: Microstrain vs. time I2H3 Specimen 1

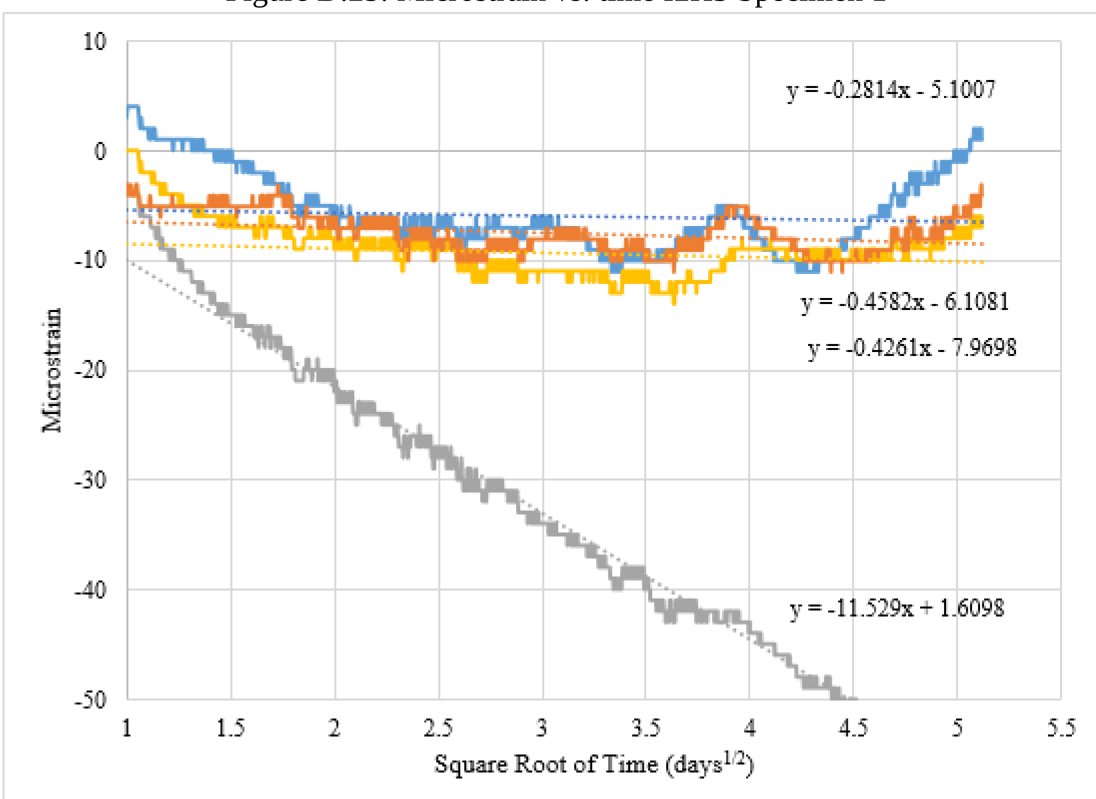


Figure D.26: Microstrain vs. square root of time I2H3 Specimen 1

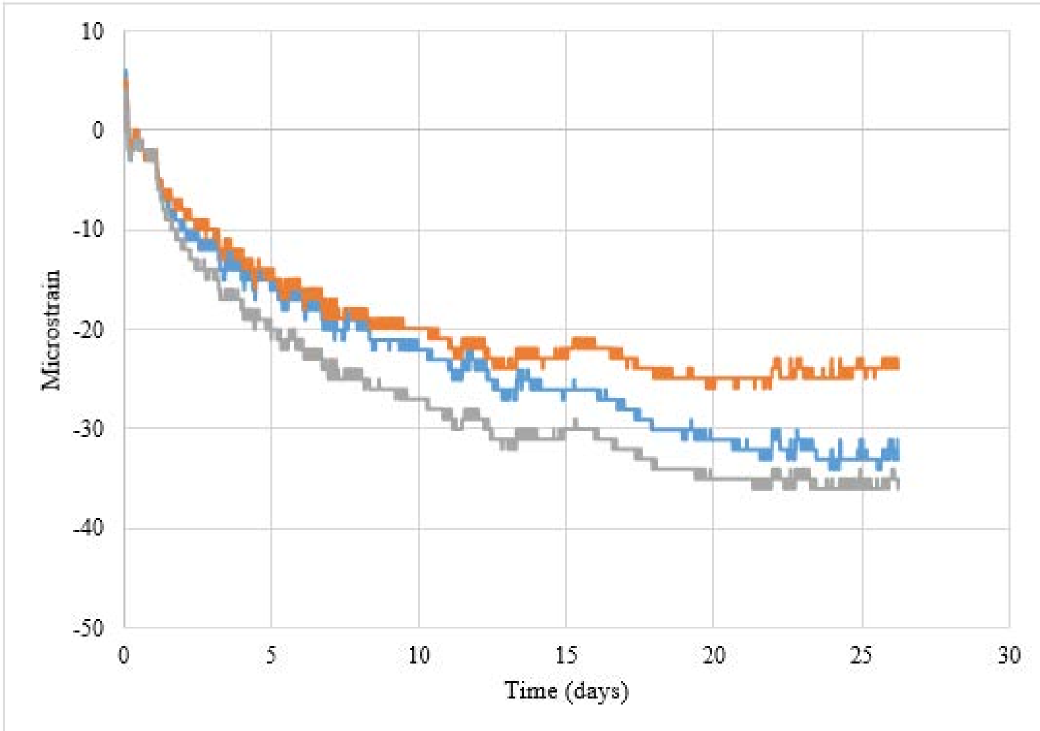


Figure D.27: Microstrain vs. time I2H3 Specimen 2

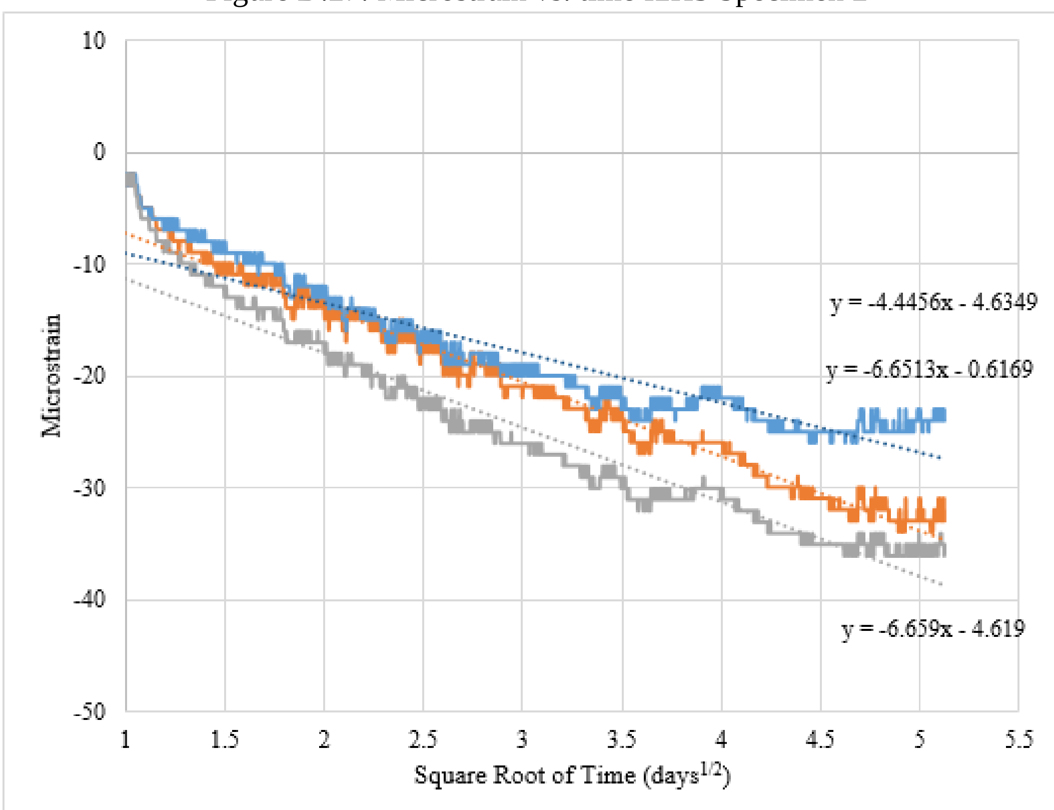


Figure D.28: Microstrain vs. square root of time I2H3 Specimen 2

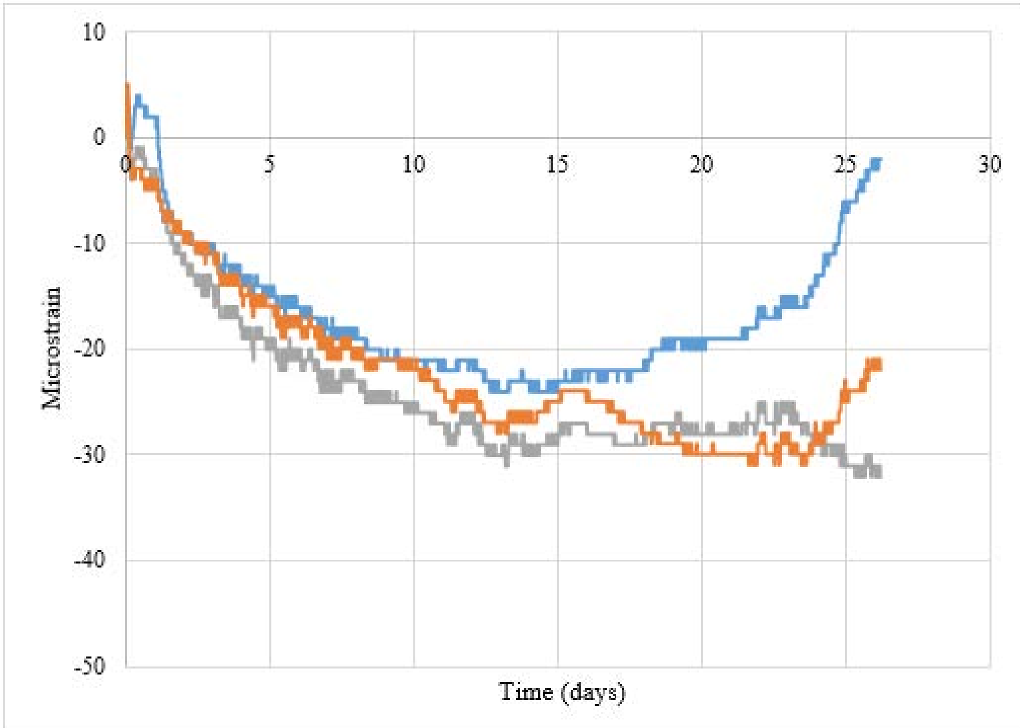


Figure D.29: Microstrain vs. time I2H3 Specimen 3

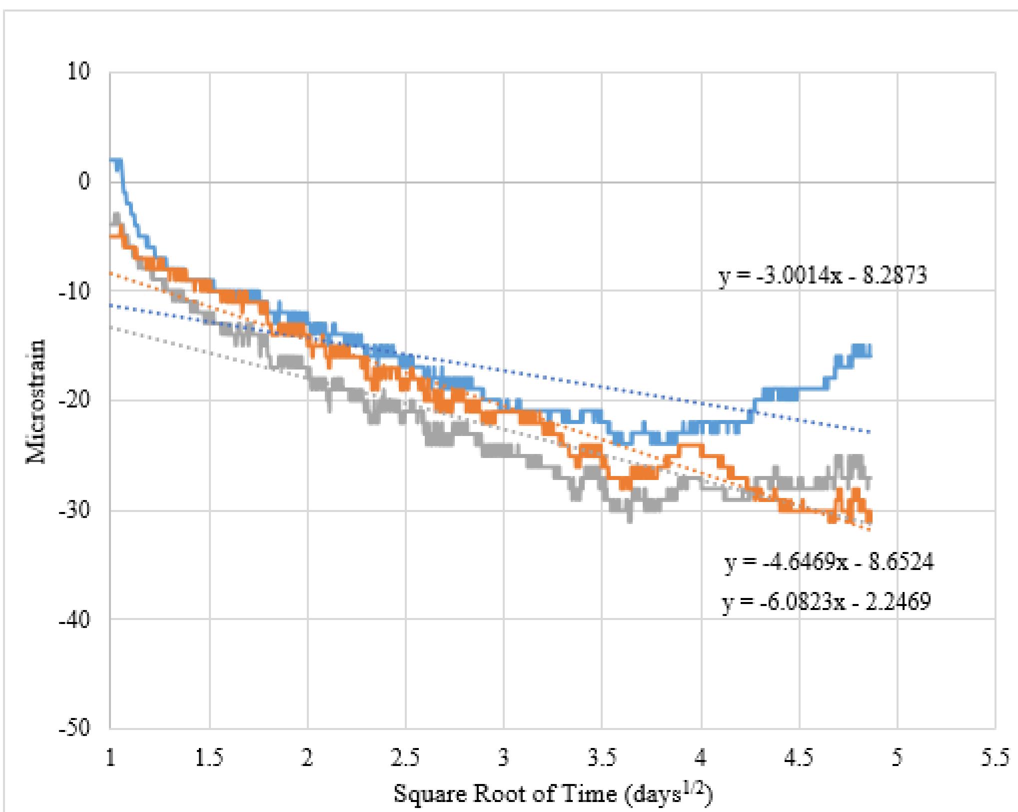


Figure D.30: Microstrain vs. square root of time I2H3 Specimen 3

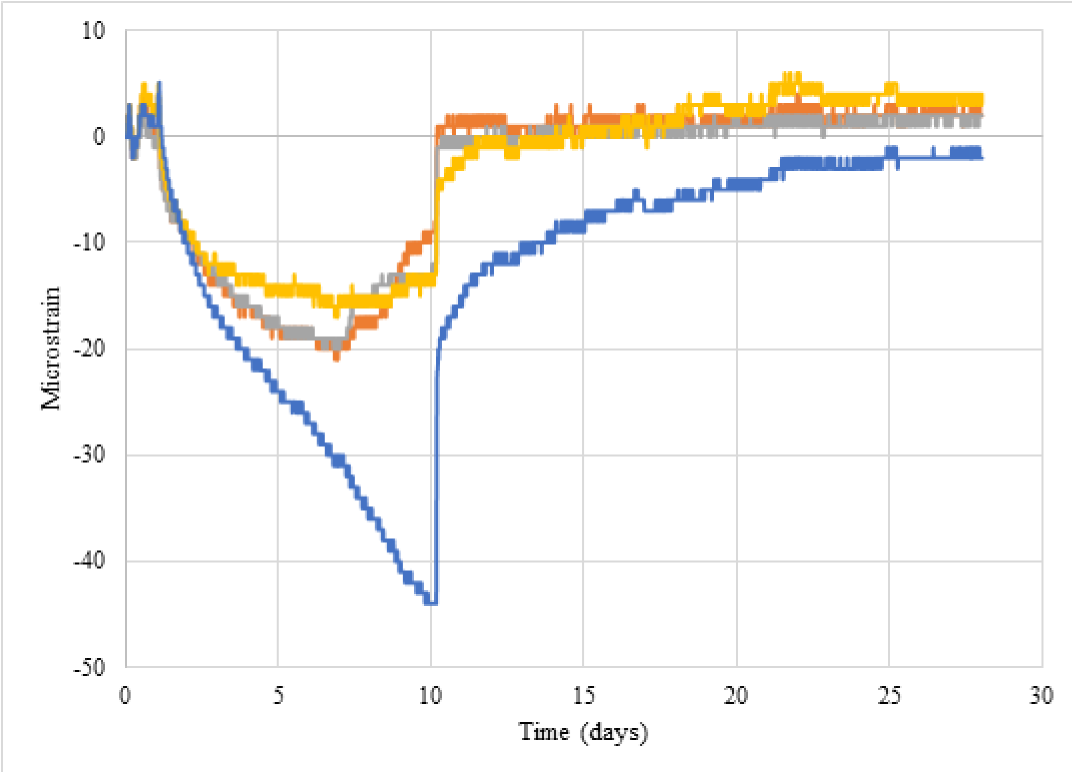


Figure D.31: Microstrain vs. time CF3 Specimen 1

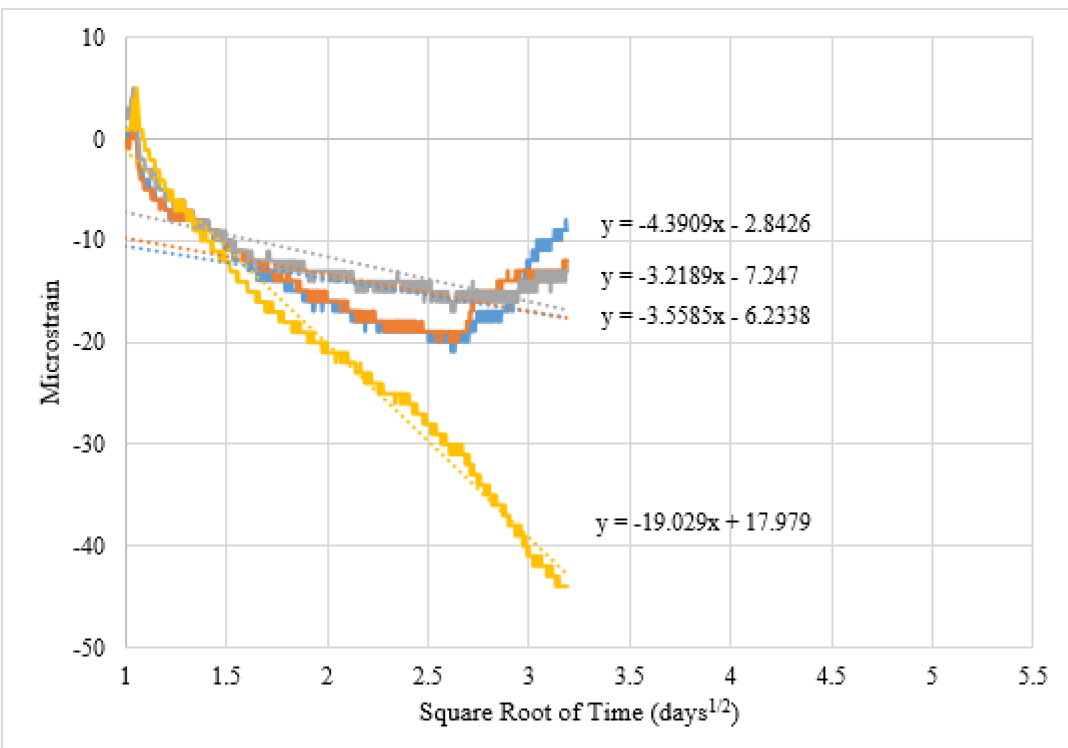


Figure D.32: Microstrain vs. square root of time CF3 Specimen 1

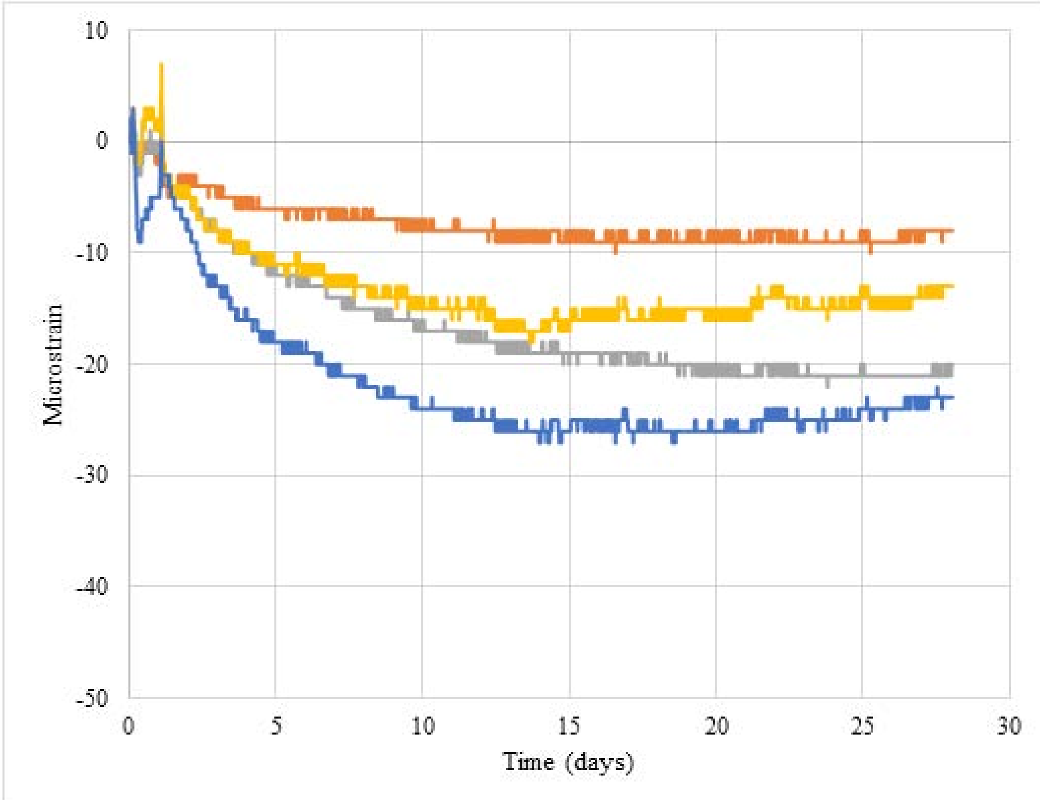


Figure D.33: Microstrain vs. time CF3 Specimen 2

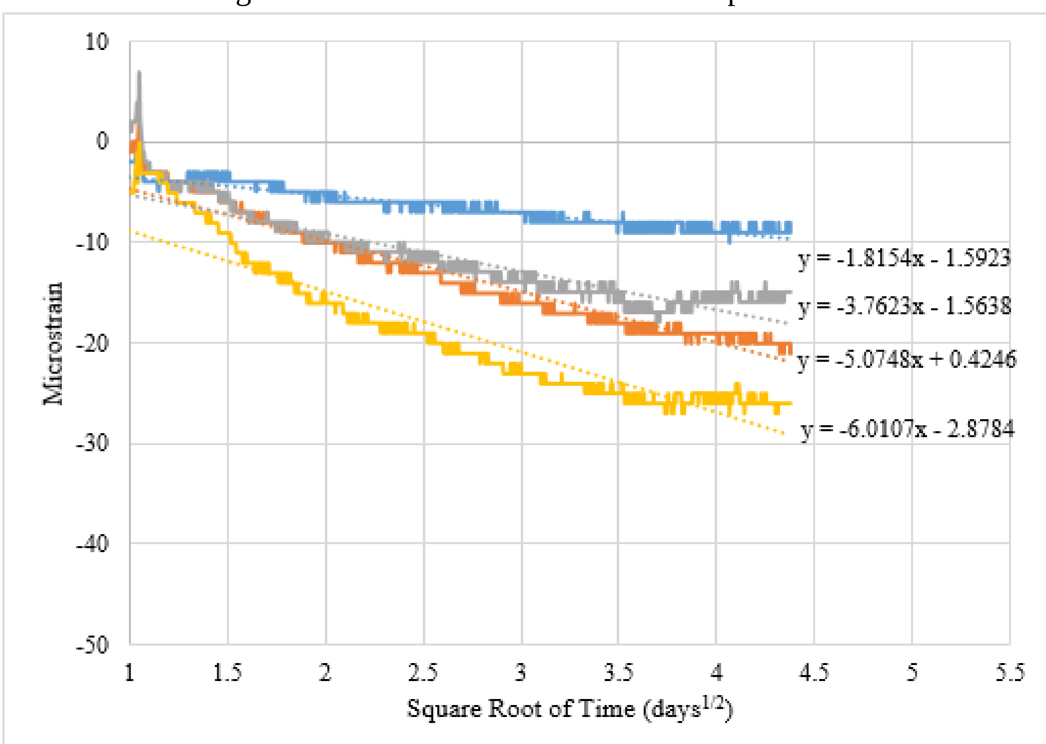


Figure D.34: Microstrain vs. square root of time CF3 Specimen 2

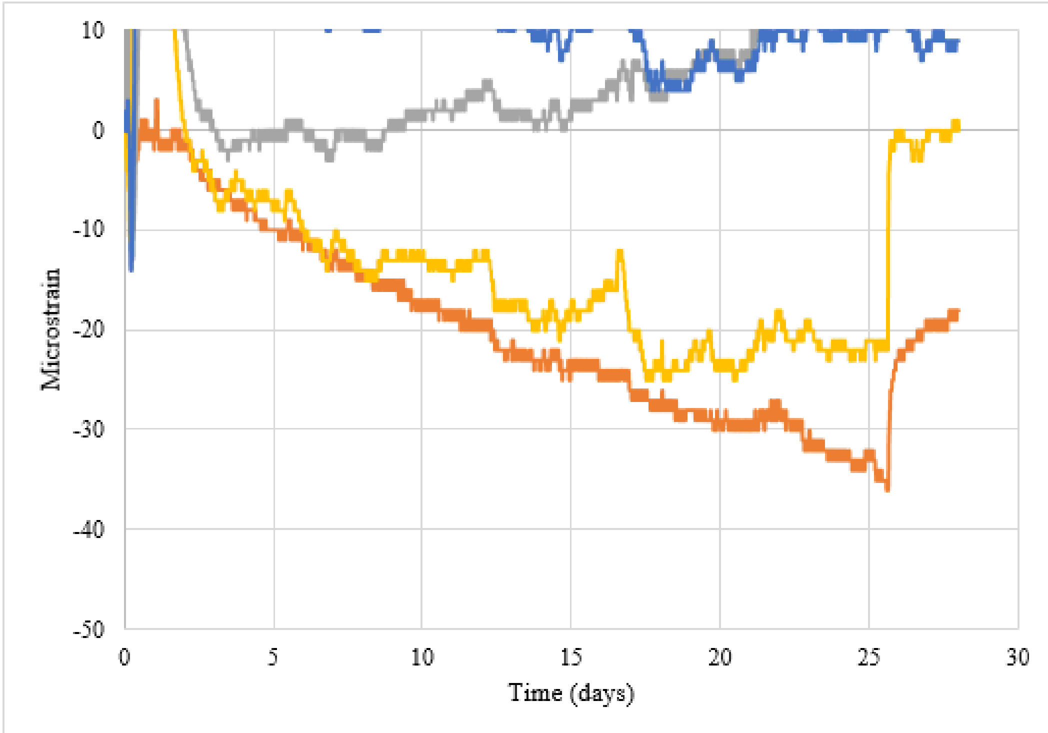


Figure D.35: Microstrain vs. time CF3 Specimen 3

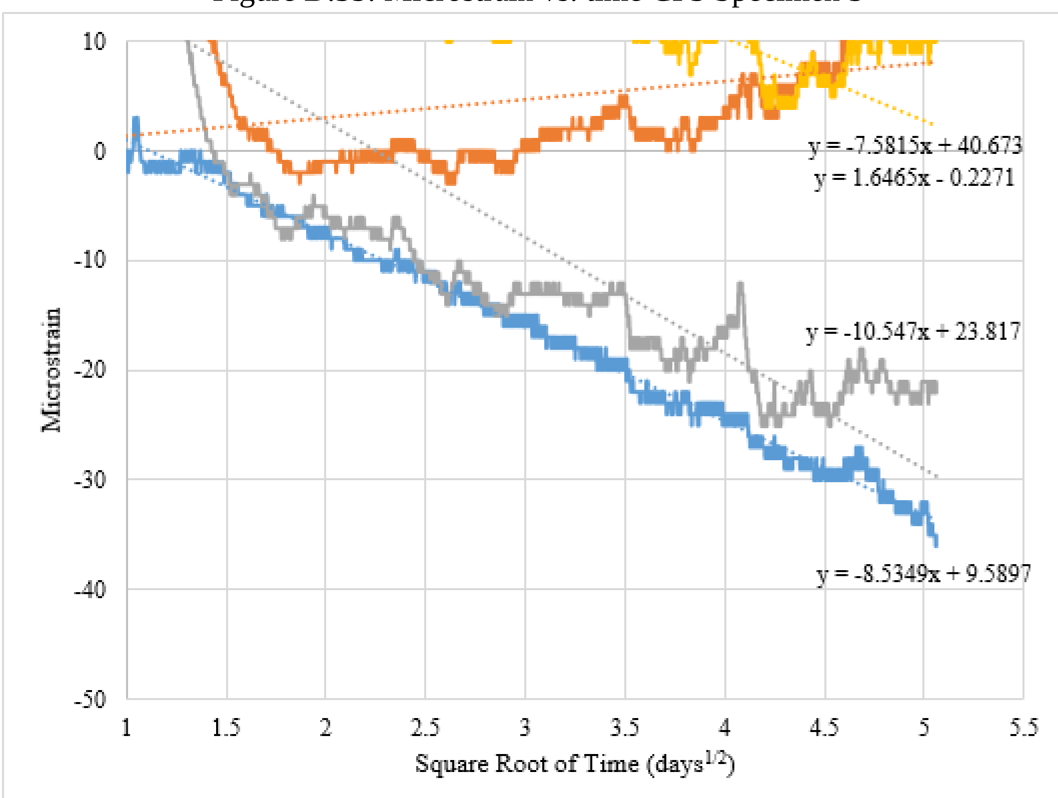


Figure D.36: Microstrain vs. square root of time CF3 Specimen 3

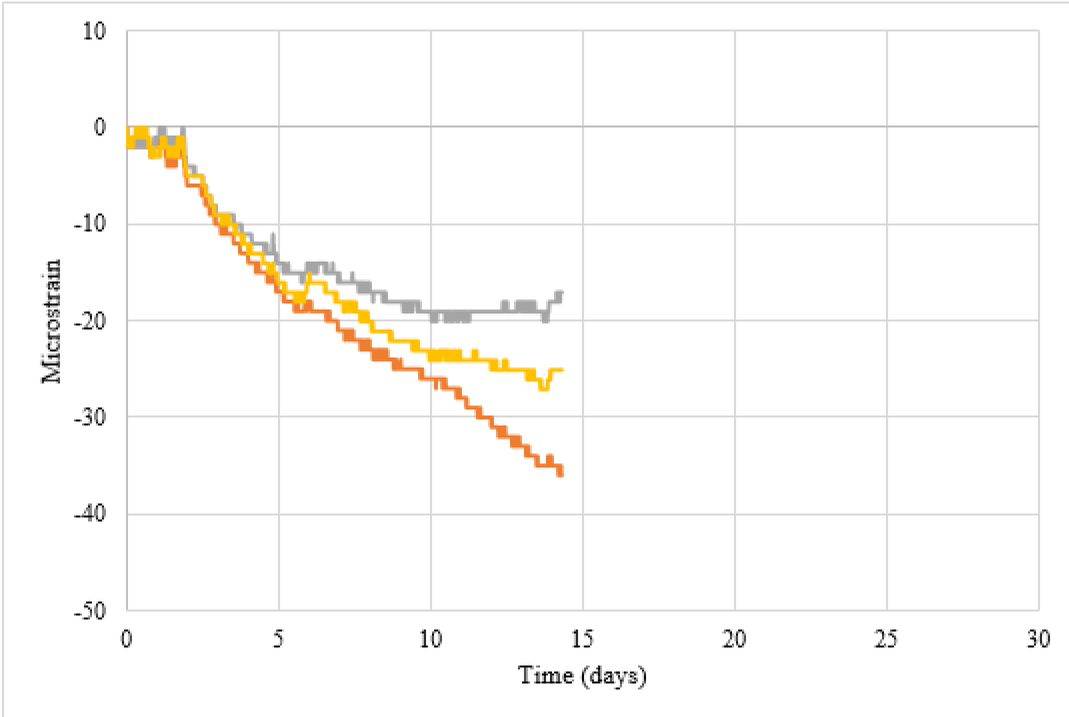


Figure D.37: Microstrain vs. time I1MF3 Specimen 1

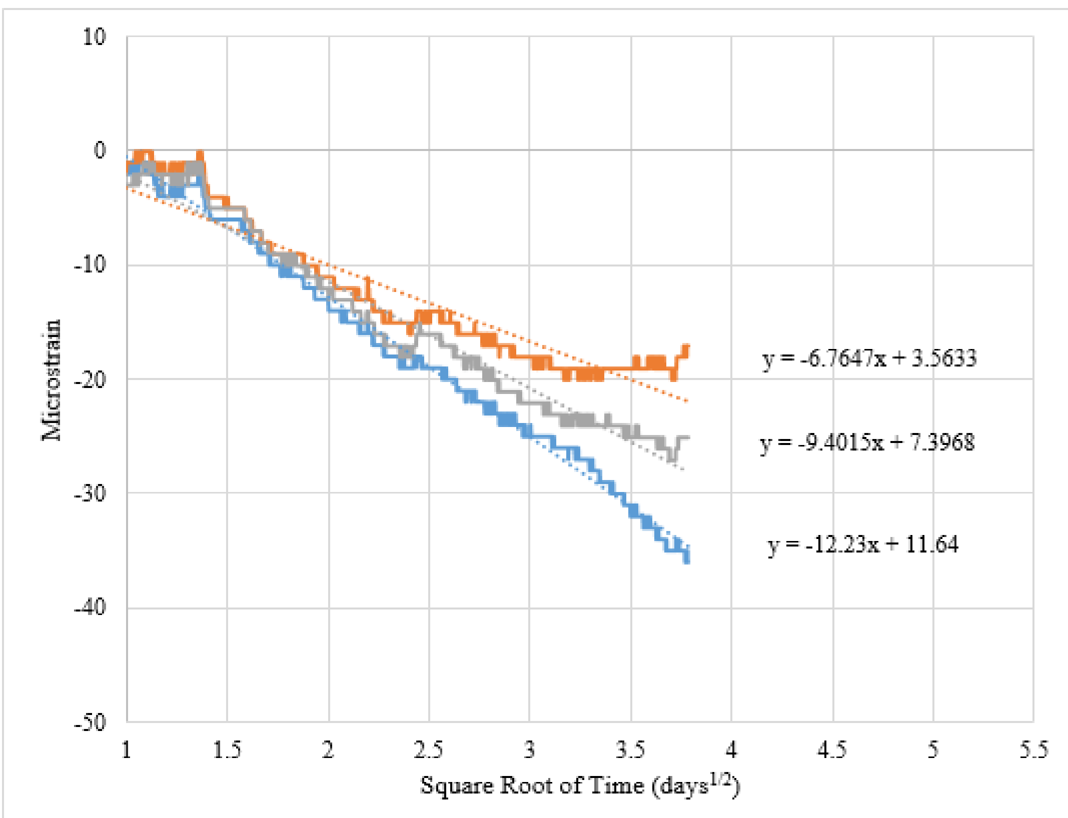


Figure D.38: Microstrain vs. square root of time I1MF3 Specimen 1

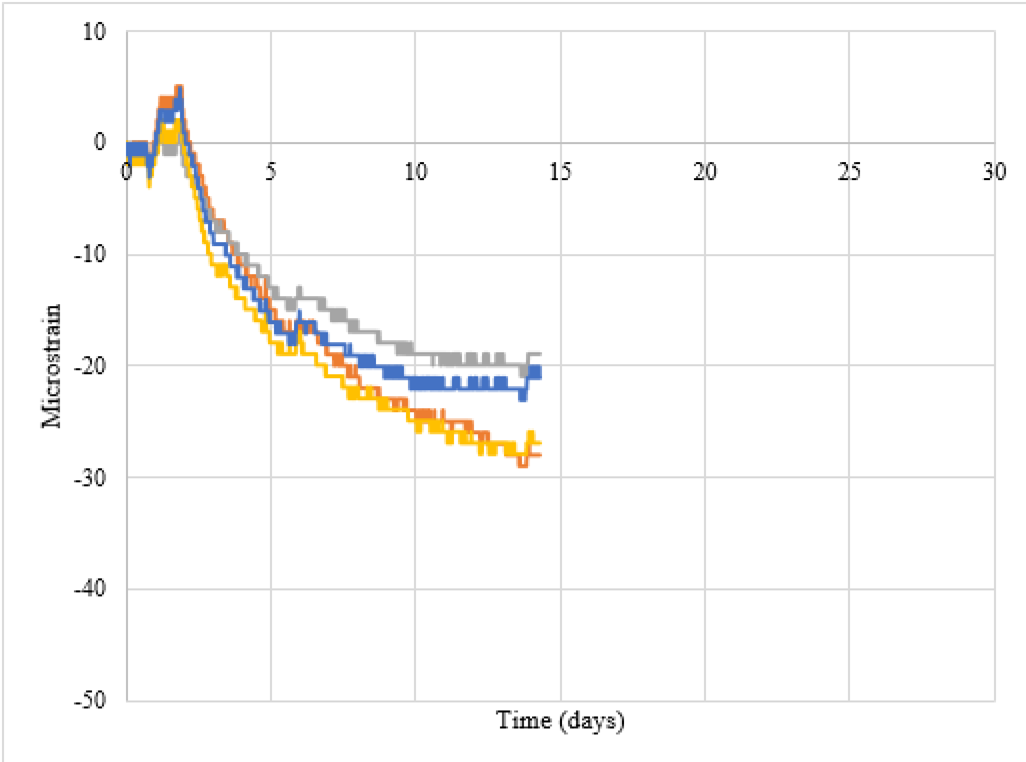


Figure D.39: Microstrain vs. time I1MF3 Specimen 2

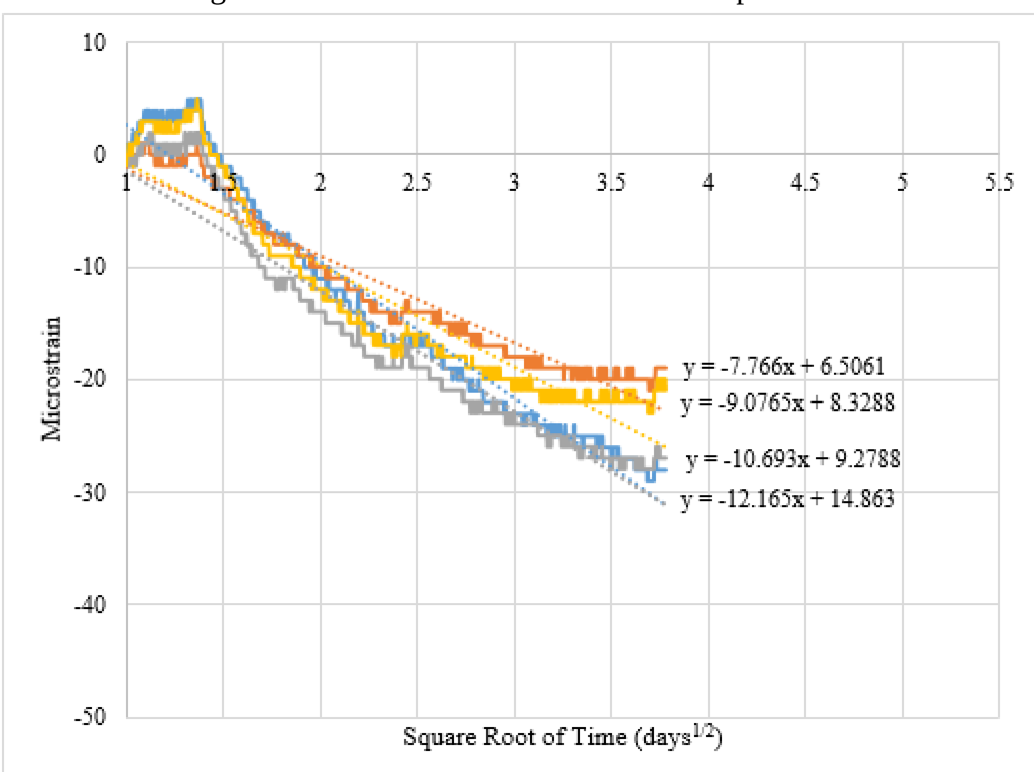


Figure D.40: Microstrain vs. square root of time I1MF3 Specimen 2

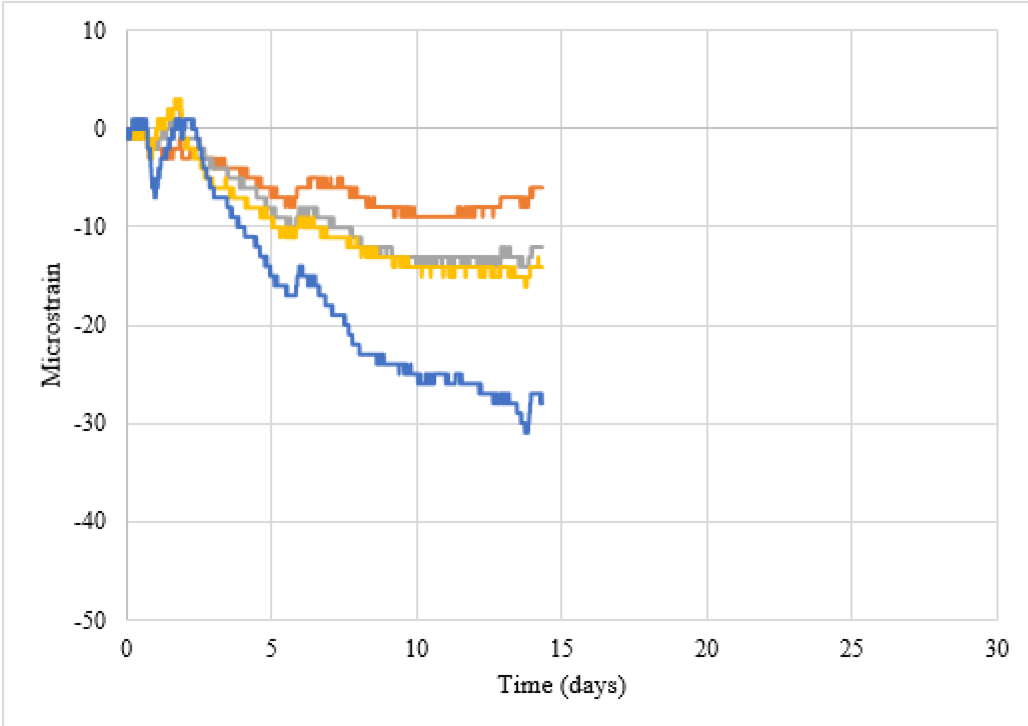


Figure D.41: Microstrain vs. time I1MF3 Specimen 3

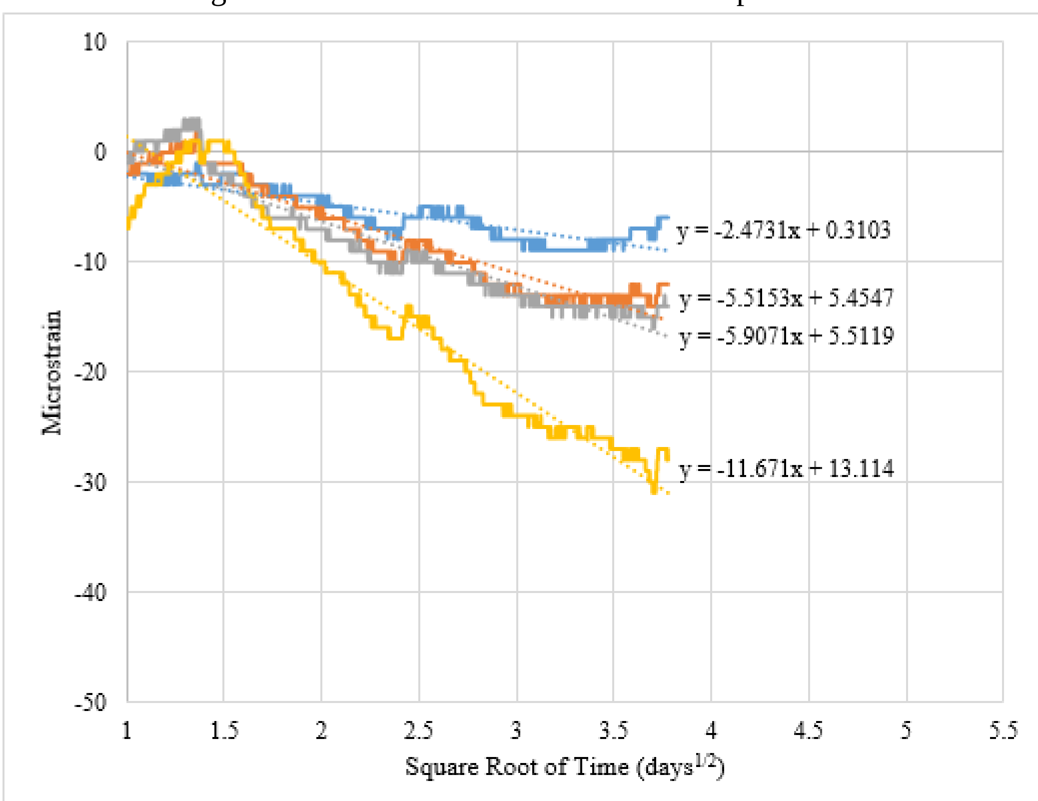


Figure D.42: Microstrain vs. square root of time I1MF3 Specimen 3

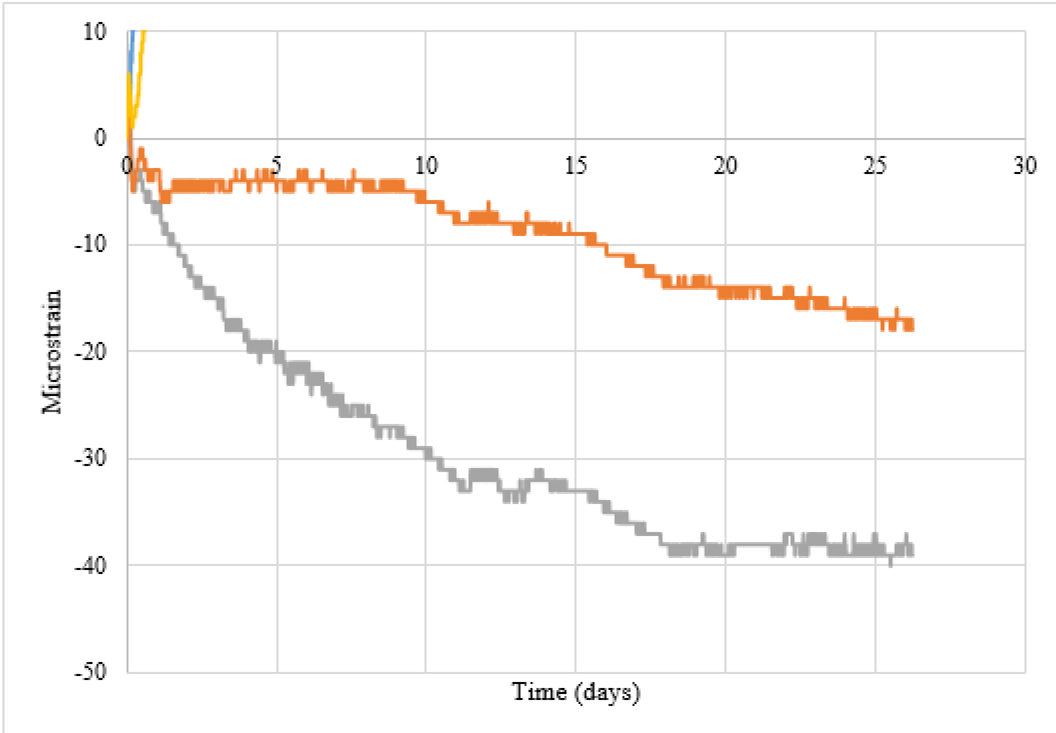


Figure D.43: Microstrain vs. time I2MF3 Specimen 1

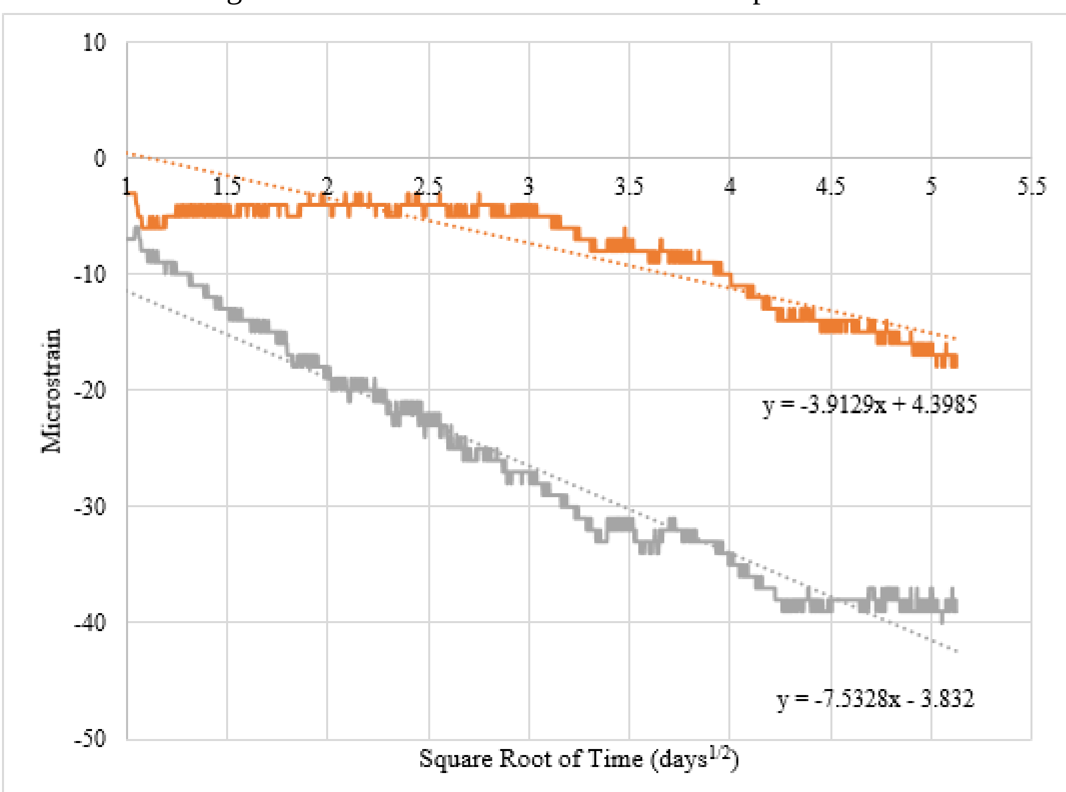


Figure D.44: Microstrain vs. square root of time I2MF3 Specimen 1

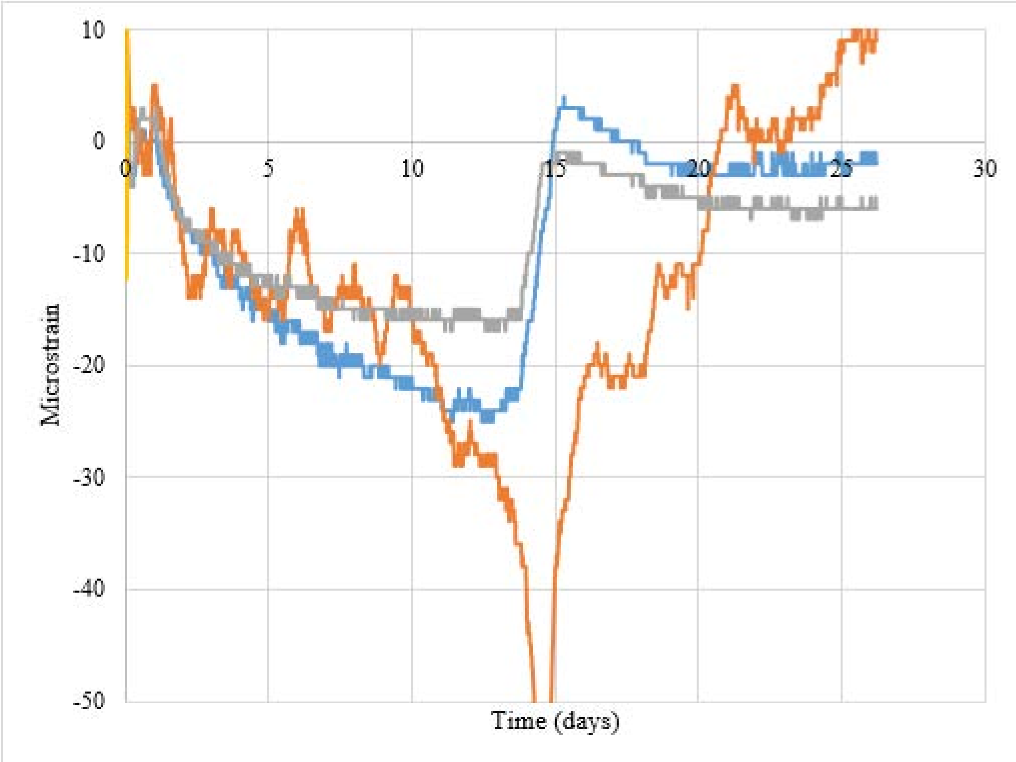


Figure D.45: Microstrain vs. time I2MF3 Specimen 2

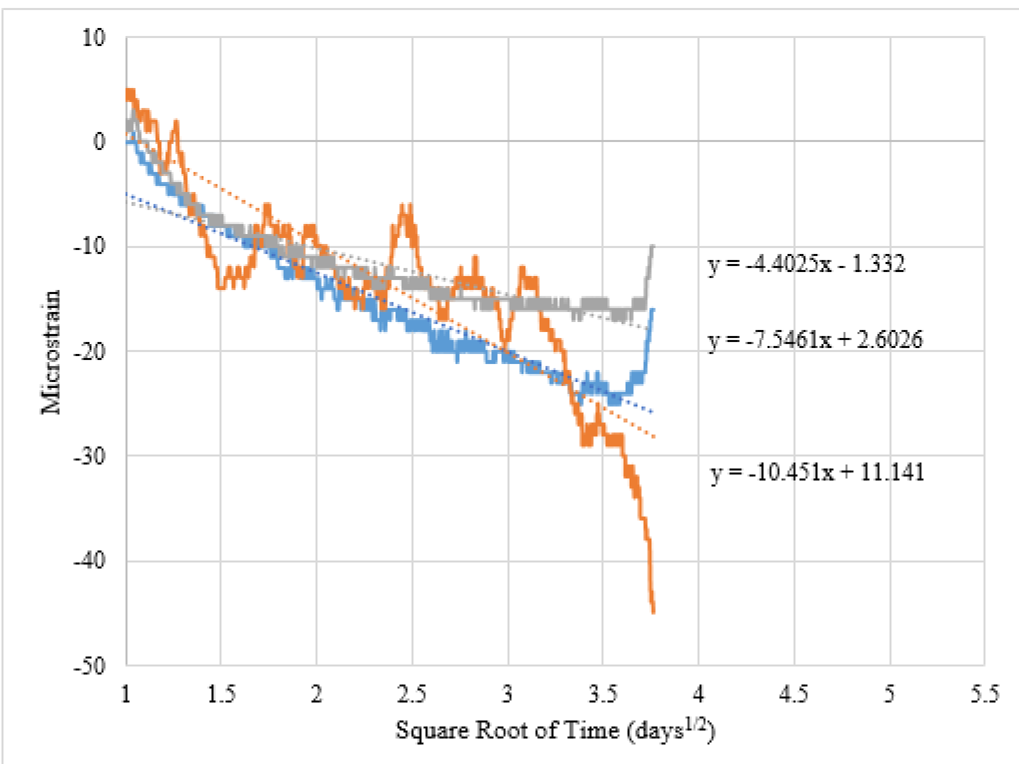


Figure D.46: Microstrain vs. square root of time I2MF3 Specimen 2

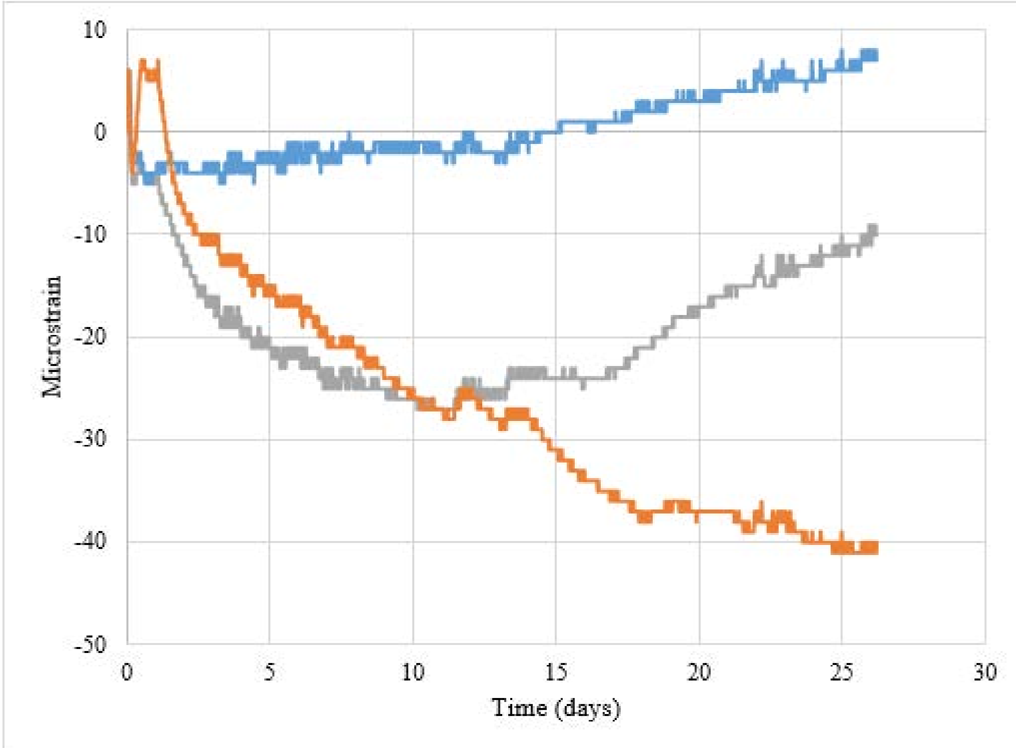


Figure D.47: Microstrain vs. time I2MF3 Specimen 3

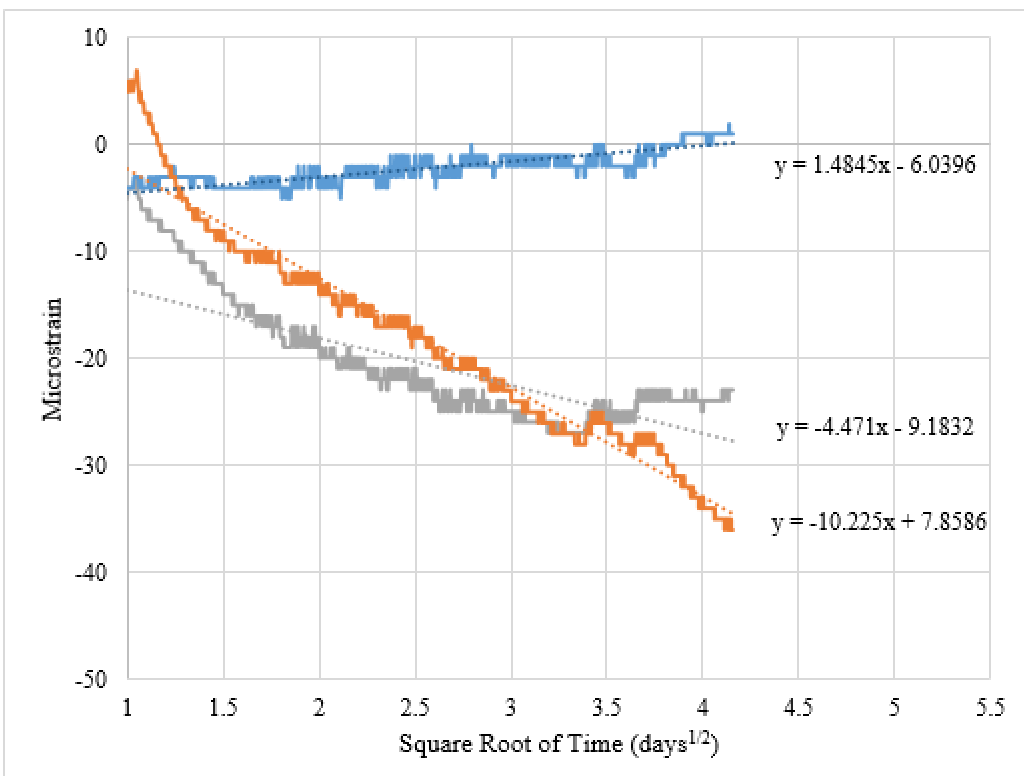


Figure D.48: Microstrain vs. square root of time I2MF3 Specimen 3

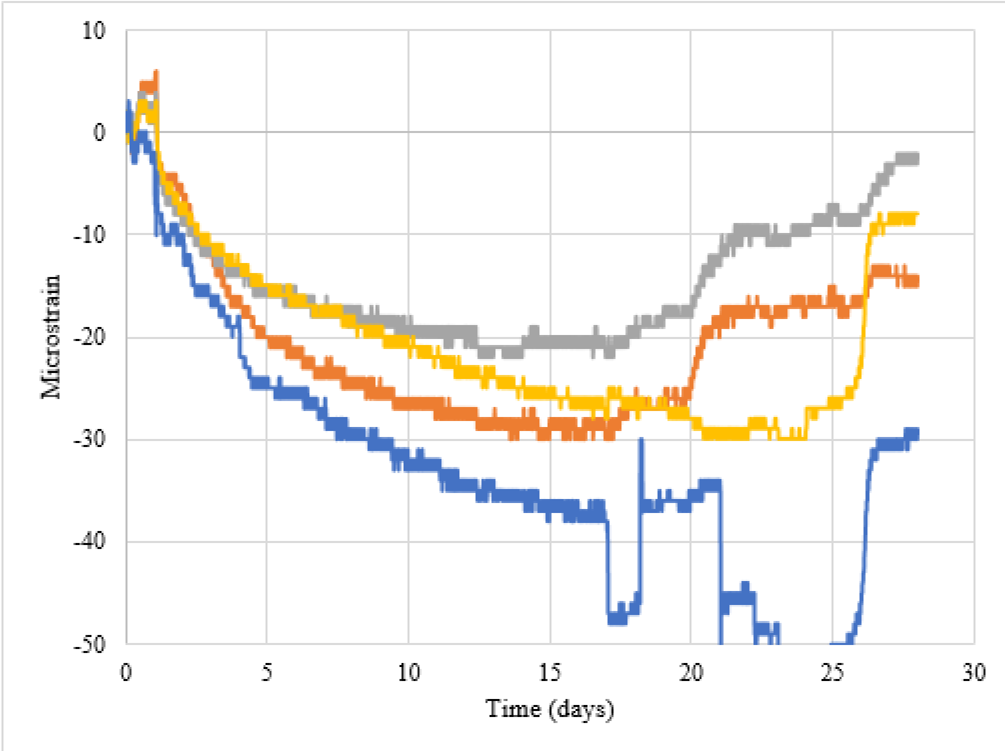


Figure D.49: Microstrain vs. time I1HF3 Specimen 1

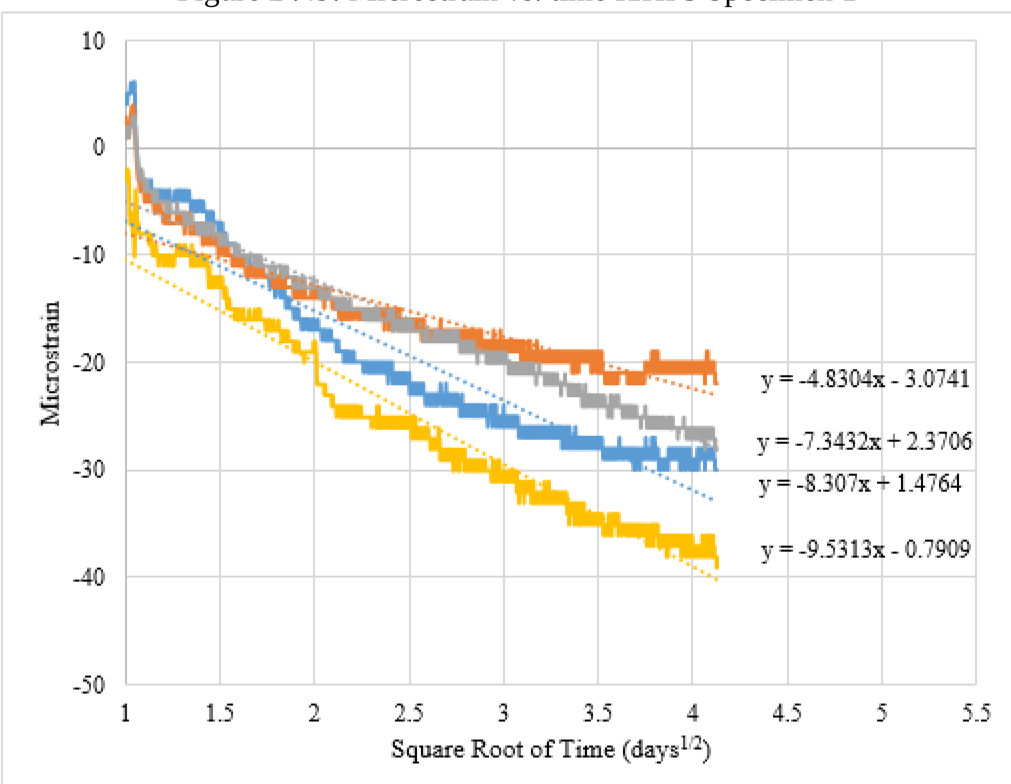


Figure D.50: Microstrain vs. square root of time I1HF3 Specimen 1

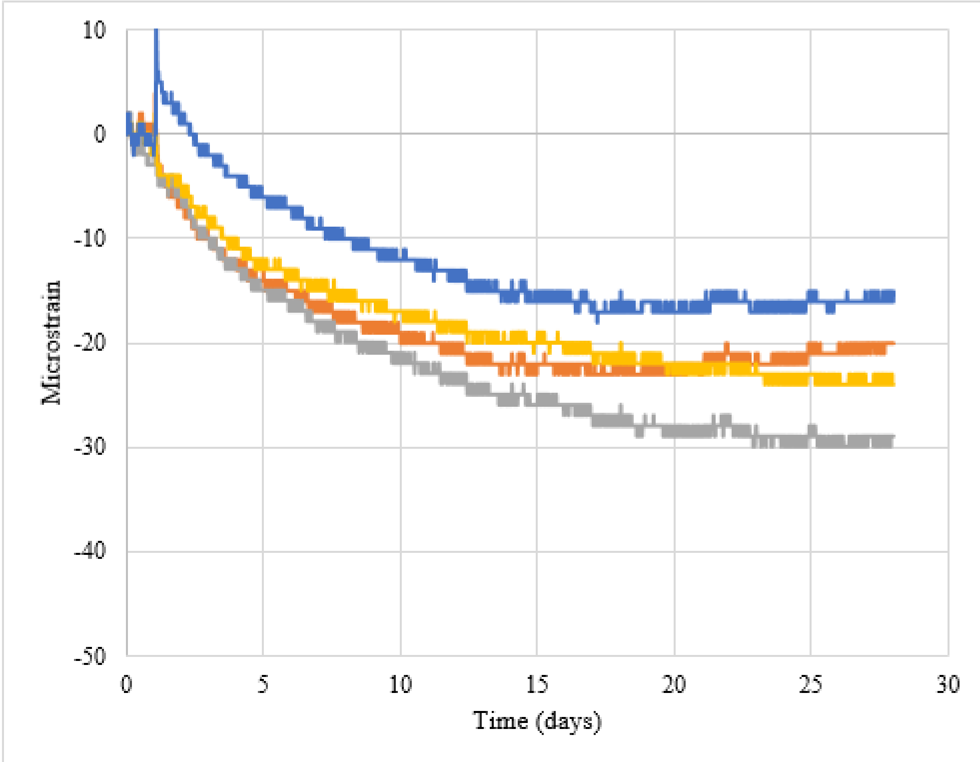


Figure D.51: Microstrain vs. time I1HF3 Specimen 2

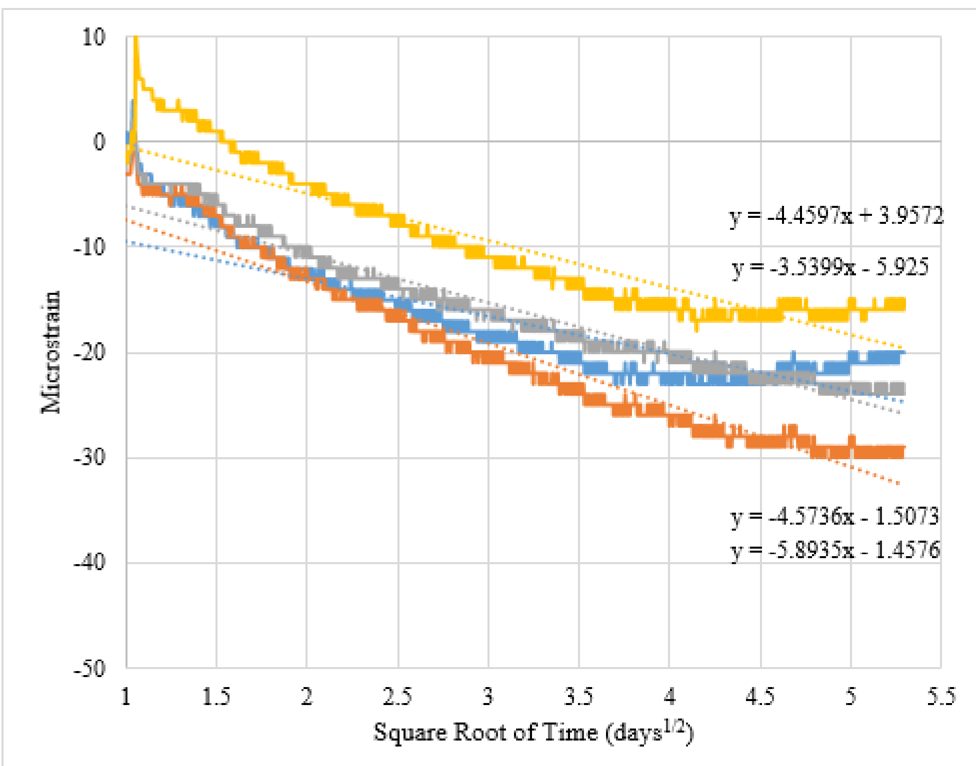


Figure D.52: Microstrain vs. square root of time I1HF3 Specimen 2

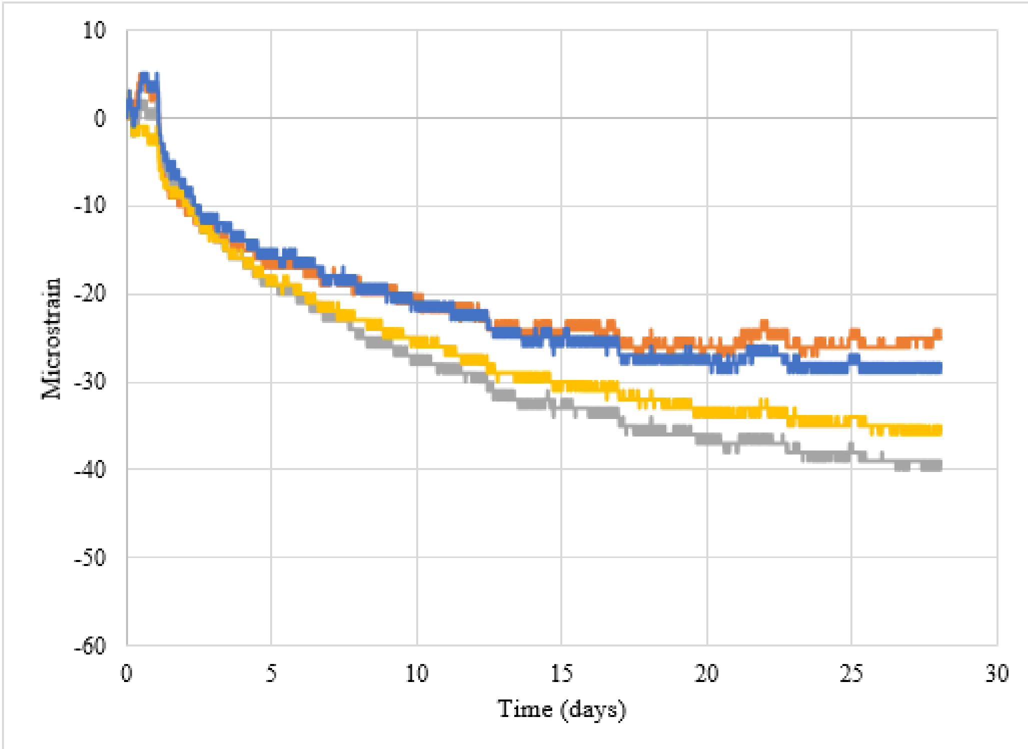


Figure D.53: Microstrain vs. time I1HF3 Specimen 3

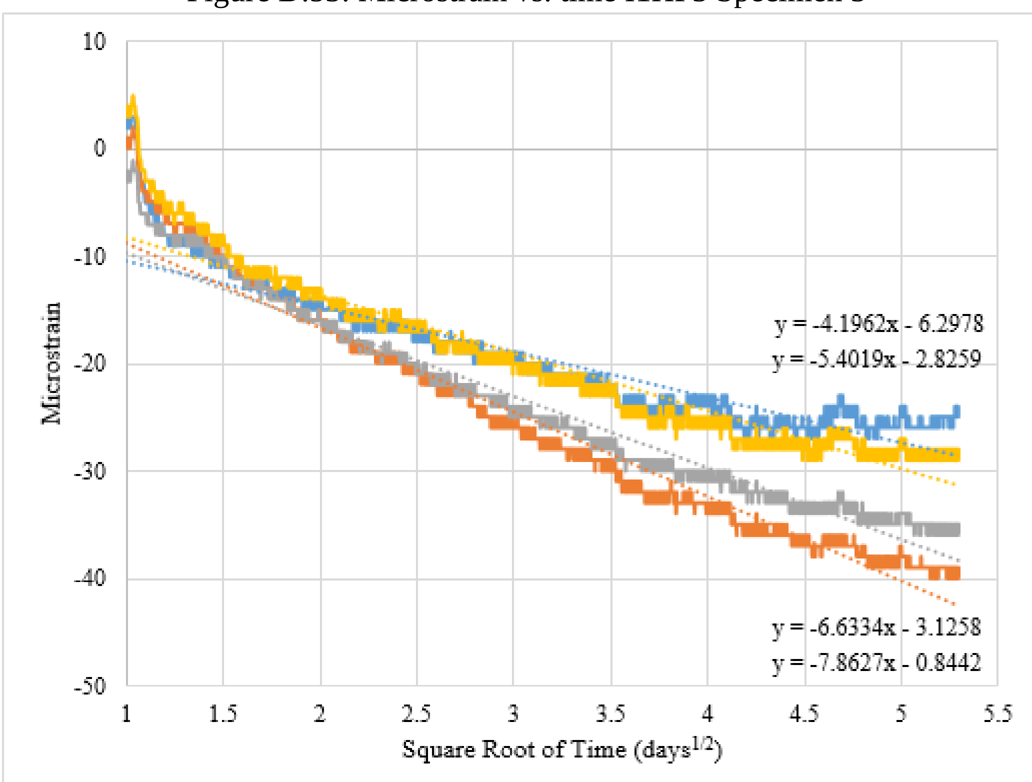


Figure D.54: Microstrain vs. square root of time I1HF3 Specimen 3