RESEARCH & DEVELOPMENT

Internal Curing of Concrete Using Lightweight Aggregate

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NCDOT Project 2016-06

FHWA/NC/2016-06

May 2019

Technical Report Documentation Page

1. Report No. FHWA/NC/2016-06	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Internal Curing of Concrete Using Lightweight Aggregate		5. Report Date May 2019 6. Performing Organization Code
7. Author(s) Tara L. Cavalline, Ph.D., P.E., Brett Q. Tempest, Ph.D., P.E., Jacob W. Leach, Ross A. Newsome, Gregory D. Loflin, Mark J. Fitzner		8. Performing Organization Report No.
9. Performing Organization Name and Address Department of Engineering Technology and Construction Management University of North Carolina at Charlotte 9201 University City Boulevard Charlotte, NC 28223-0001		10. Work Unit No. (TRAIS) 11. Contract or Grant No. FHWA/NCDOT/2016-06
12. Sponsoring Agency Name and Address North Carolina Department of Transportation Research and Development Unit 104 Fayetteville Street Raleigh, North Carolina 27601		13. Type of Report and Period Covered Final Report August 1, 2015 – June 30, 2018 14. Sponsoring Agency Code 2016-06
Supplementary Notes:		

16. Abstract

The increased absorptive capacity of lightweight aggregates (LWA) has been shown to facilitate delivery of moisture to concrete for internal curing. The additional hydration supplied by prewetted LWA to support internally cured concrete (ICC) has been linked to improved concrete performance, including reduced permeability and lower cracking tendency. To assist NCDOT in evaluating the benefits of ICC produced with local materials, as well as to establish the value of using ICC in specific types of projects, laboratory and field studies were performed. A range of concrete mixtures, including ICC mixtures using locally available prewetted LWA in concrete mixtures typically used in bridge decks and pavements were batched and tested. The results of laboratory testing confirmed the benefits of ICC, including the potential for reduced cracking and, in some instances, reduced permeability. Autogenous shrinkage test results clearly demonstrated the impact of use of prewetted LWA, with significantly reduced shrinkage observed for both types of LWA. The reduction in autogenous shrinkage correlated with LWA replacement rate. Use of fly ash, in both conventional and ICC mixtures was more influential in reducing the permeability of mixtures than addition of prewetted LWA. Use of prewetted LWA provided additional workability to conventional and very high early strength (VHES) latex-modified concrete mixtures (LMC), potentially supporting an additional opportunity to achieve superior durability performance through use of lower water to cementitious material (w/cm) ratios. The internally cured VHES LMC mixture appeared to have extended work time, possibly offering construction advantages, although field trials are suggested to explore this potential benefit. A project special provisions (specification addendum) developed as part of this work was included in the construction documents for a pilot project bridge deck to evaluate ICC. The pilot project bridge deck, comprised of both ICC and conventional (control) concrete was constructed, instrumented, and monitored for a period of 31 weeks. During this observation phase the team did not observe substantial differences in the performance of the two materials in terms of moisture retention, drying shrinkage or crack formation. However, the locally available LWA used in the pilot project has a relatively low absorption, and the substitution rate was lower than that recommended by ACI (308-213)R-13. Benefits could be more pronounced at higher substitution rates, and/or later in the project's service life. This field implementation was successful, and lessons learned from the pilot project stakeholders and construction experience were utilized to improve the specification recommendations for future use of ICC by NCDOT. Implementation of ICC in future projects should result in longer-lasting structures with reduced maintenance and preservation costs.

17.	18. Distribution Statement		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 113	22. Price

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

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ACKNOWLEDGMENTS

This research project was sponsored by the North Carolina Department of Transportation (NCDOT) and their continued support is greatly appreciated. The research team would like to express their appreciation to the following:

- The NCDOT personnel serving on the Steering and Implementation Committee for this research study. In particular, we would like to thank Christopher Peoples, Chair of the Steering and Implementation Committee for his insight, assistance, and support. Feedback received from all members of the Steering and Implementation Committee (Chris Peoples, Brian Hunter, Kevin Bowen, Jessica Earley, Clark Morrison, James Phillips, and Tim Sherrill) during the course of this project was greatly appreciated.

- Aaron Earwood, Troy Brooks, and James Nordan with NCDOT for their support and assistance with the pilot project, as well as their invaluable feedback supporting the report and the development of the specification.

- NCDOT Research and Development personnel, particularly Mustan Kadhibhai and Neil Mastin.

- Material suppliers for their generous donations of aggregates, cementitious materials, and admixtures supporting this research.

- Wesley Maxwell, Research Operations Manager, UNC Charlotte Department of Engineering Technology and Construction Management.

- Dr. Katherine Weaver, Manager of the Materials Characterization Laboratory at UNC Charlotte.

- UNC Charlotte Graduate Research Assistants – Edward W. Blanchard, Clayton D. Medlin, Rohit R. Chimmula, Taye Ojo, Robert (Blake) Biggers.

- UNC Charlotte Undergraduate Research Assistants - Casey Schilder, Gunnar Wright, Joe OCampo, and John Wassilchak.

- Industry representatives from the lightweight aggregate suppliers for this project.

- Industry representatives from the contractor and ready-mixed concrete supplier for the pilot project.

- Representatives from the ready-mixed concrete and concrete repair and rehabilitation industries providing information useful to support the cost-benefit analysis.

- Dr. Anthony L. Brizendine, Chair of the Department of Engineering Technology and Construction Management at UNC Charlotte.

EXECUTIVE SUMMARY

The increased absorptive capacity of lightweight aggregates (LWA) has been shown to facilitate delivery of moisture to concrete for internal curing. The additional hydration supplied by prewetted LWA to support internally cured concrete (ICC) has been linked to improved concrete performance, including reduced permeability and lower cracking tendency. To aid NCDOT in evaluating the benefits of ICC produced with local materials, as well as to establish the value of using ICC in specific types of projects, laboratory and field studies were performed. A range of concrete mixtures, including ICC mixtures using locally available prewetted LWA in concrete mixtures typically used in bridge decks and pavements were batched and tested.

The results of laboratory testing confirmed the benefits of ICC, including the potential for reduced cracking and, in some instances, reduced permeability. Autogenous shrinkage test results clearly demonstrated the impact of use of prewetted LWA, with significantly reduced shrinkage observed for both types of LWA. The reduction in autogenous shrinkage correlated with LWA replacement rate. Use of fly ash, in both conventional and ICC mixtures was more influential in reducing the permeability of mixtures than addition of prewetted LWA. Use of prewetted LWA provided additional workability to conventional and very high early strength (VHES) latex-modified concrete mixtures (LMC), potentially supporting an additional opportunity to achieve superior durability performance through use of lower water to cementitious material (w/cm) ratios. The internally cured VHES LMC mixture appeared to have extended work time, possibly offering construction advantages, although field trials are suggested to explore this potential benefit.

A project special provisions (specification addendum) developed as part of this work was included in the construction documents for a pilot project bridge deck to evaluate ICC. The pilot project bridge deck, comprised of both ICC and conventional (control) concrete was constructed, instrumented, and monitored. During a 31 week period, no substantial differences were observed between the performance of the ICC section and the conventional section of the bridge deck based on measurements of internal humidity and shrinkage strains. The results observed in the field may have differed from laboratory observations due to the great degree of restraint offered by the reinforcing bar mats and the steel bridge girders. It is noted that the locally available LWA used in the pilot project has a relatively low absorption, and per NCDOT preferences, the substitution rate was lower than that recommended by ACI (308-213)R-13. Benefits could be more pronounced at higher substitution rates, and/or later in the project's service life.

Field implementation of ICC was successful because it did not present any insurmountable challenges to the contractor or concrete supplier, and lessons learned from the pilot project stakeholders and construction experience were utilized to improve the specification recommendations for future use of ICC by NCDOT. Implementation of ICC in future projects should result in longer-lasting structures with reduced maintenance and preservation costs.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials	MOE	modulus of elasticity
		MOR	modulus of rupture
ACI	American Concrete Institute	NC	North Carolina
ASTM	American Society for Testing and Materials	NCDOT	North Carolina Department of Transportation
°C	degrees Celsius		
CC	conventional concrete	Ω	ohm
cf	cubic foot	OPC	ordinary portland cement
cwt	hundred weight of cement	oz	ounce
cy	cubic yard	P_c	Relative dynamic modulus of elasticity
CTE	coefficient of thermal expansion	PCA	Portland Cement Association
DOT	Department of Transportation	PCC	portland cement concrete
DF	durability factor	pcf	pounds per cubic foot
°F	degrees Fahrenheit	pcy	pounds per cubic yard
FHWA	Federal Highway Administration	psi	pounds per square inch
ft	foot	PSP	project special provisions
g	gram	PVC	polyvinyl chloride
gal	gallon	RCPT	rapid chloride permeability test
hr	hour	RH	relative humidity
kg	kilogram	RP	Research Project
kΩ	kilo-ohm	rpm	revolutions per minute
ICC	internally cured concrete	S	siemens
ID	identification	SAM	Super Air Meter
in	inch	SCM	supplementary cementitious material
”	inch	SCMs	supplementary cementitious materials
ITM	Indiana Testing Method	SSD	saturated surface dry
ITZ	interfacial transition zone	ε	strain
lb	pound	t_r	time to cracking
LMC	latex-modified concrete	TRB	Transportation Research Board
LRFD	load resistance factor design	UNC	University of North Carolina
LWA	lightweight aggregate	US	United States
LWA1	lightweight fine aggregate 1	V	volts
LWA2	lightweight fine aggregate 2	VHES	very high early strength
m	meter	VOC	volatile organic compounds
μ	micro	*w/c*	water to cement ratio
mL	milliliter	*w/cm*	water to cementitious materials ratio
mm	millimeter	yd	yard

1. INTRODUCTION AND RESEARCH OBJECTIVES

1.1 Introduction

Internally cured concrete (ICC) is a promising solution shown to help mitigate early age cracking and to provide improved durability performance in transportation concrete applications (Villareal and Crocker 2007, Delatte and Cleary 2008, Friggle and Reeves 2008, Streeter et al. 2012, Guthrie and Yaede 2013, Barrett et al. 2015, and others). The increased absorptive capacity of lightweight aggregates (LWA) facilitates delivery of moisture to concrete to support internal curing. Water included in the void system of prewetted LWA is removed via capillary suction, promoting additional hydration that continues well after traditional curing methods have ceased. Internal curing allows for a denser paste microstructure to form in concrete, resulting in reduced permeability, better durability performance, and in some cases, higher strength (ACI 2013). A schematic showing the mechanism of internal curing and describing the benefits to the interfacial transition zone (ITZ) is shown schematically in Figure 1.1.

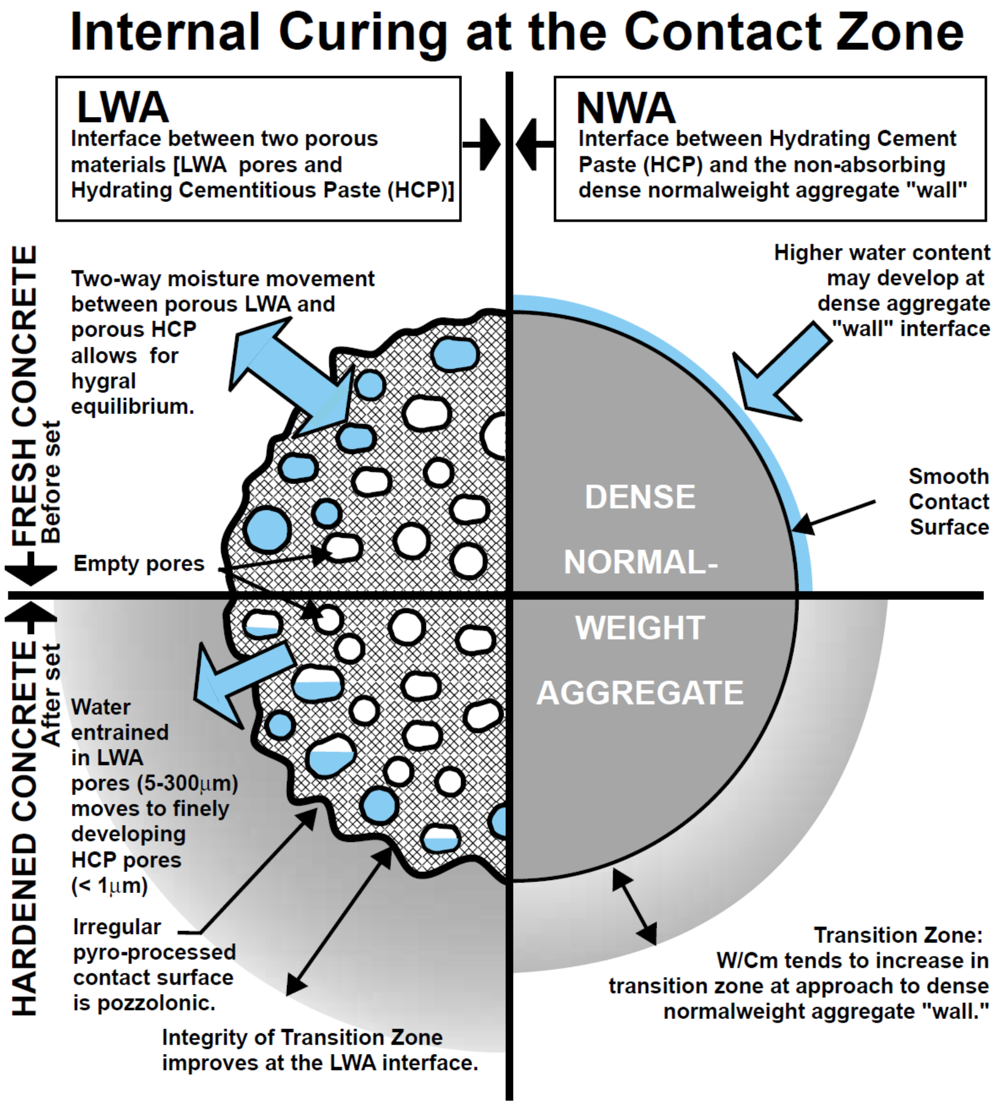

Figure 1.1: Mechanism of internal curing using prewetted LWA (from ESCSI 2006)

The benefits of internal curing have been demonstrated in a number of laboratory and field studies across the United States, and many state highway agencies (SHAs) have used ICC in highway infrastructure projects. These projects have primarily included bridge decks, but internally cured pavements and other elements have also been constructed. Several LWAs are locally available to support construction of North Carolina infrastructure, and the North Carolina Department of Transportation (NCDOT) desires to obtain data to support use of ICC in appropriate projects, where the cost could be justified through the improved performance and reduced maintenance, repair, and rehabilitation (MR&R) costs. Laboratory testing to compare ICC mixtures with conventional mixtures would help determine the performance improvements that could be obtained, assist in development of specifications, and support evaluation of the cost effectiveness of ICC.

Similar to other state highway agencies, the NCDOT has reported issues of early-age cracking of bridge deck concrete mixtures that include high cementitious contents and low water-to-cement ratios, and internal curing may be a useful technology to assist in improving the performance of bridge decks in North Carolina. Bridge deck concrete mixtures, both with and without the supplementary cementitious material (SCM) fly ash, are the focus of this research project. Additionally, NCDOT has been increasingly utilizing latex modified concrete (LMC) and very high early strength (VHES) latex modified concrete mixtures in bridge deck applications. The benefits of internal curing have not been extensively explored in these types of mixtures, and NCDOT desired to determine if use of prewetted LWA for internal curing would benefit these mixtures as well. Also of interest to NCDOT are the potential performance improvements that internal curing could offer in concrete pavement mixtures.

As part of this project, NCDOT sought to gain experience in construction of ICC in a field application (pilot project). Feedback from stakeholders (NCDOT, contractor, and concrete supplier personnel) support development of specification provisions for implementation of ICC in future projects. The findings of this study verify that NCDOT can achieve improved durability performance from ICC mixtures produced with locally available materials, and ICC can be utilized, where economically feasible, to support construction of longer-lasting structures with reduced maintenance, repair, and rehabilitation (MR&R) costs.

1.2 Research Objectives

This research aimed 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 was used to establish a preliminary ICC specification – a project special provision (PSP) for a pilot project for ICC in North Carolina. A field testing and monitoring plan was established and implemented for the pilot project. Findings from the pilot project included concrete test results, visual observations, and stakeholder interviews. This work supported development of recommendations for a general specification for NCDOT to be used in future ICC projects.

2. SUMMARY OF KEY LITERATURE FINDINGS

Note: A summary of key literature findings is presented in this section. The full literature review supporting this work, along with a complete list of references, is provided in Appendix A of this report.

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. Internal curing 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 (ACI 2013). 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 (Bentz and Weiss 2011).

Internally cured concrete mixtures using prewetted LWA have been shown to provide a number of performance benefits over comparable conventional mixtures (Babcock and Taylor 2015). Internal curing can significantly reduce or eliminate plastic shrinkage (Schlitter et al. 2010) and autogenous shrinkage (Bentur et al. 2001, Kovler et al. 2004, Cusson and Hoogeveen 2010, Ardeshirilajimi et al. 2016), reducing cracking potential. Internally cured concrete containing LWA has a lower modulus of elasticity, which has also been linked to reduced cracking tendency (Wolf 2008, Byard and Schindler 2010). Compressive strength (Byard and Schindler 2010) and flexural strength (Roberts 2004 and 2005) of these mixtures can be increased. Durability performance improvements associated with internally cured mixtures using prewetted LWA include: reduced slab warping (Wei and Hansen 2008), reduced chloride permeability (Bentz 2009a, Cusson and Margeson 2010), and enhanced freeze-thaw resistance (Cusson and Margeson 2010). Other benefits are associated with the reduced unit weight of internally cured concrete using prewetted LWA. Structural deadloads are decreased, which can reduce the required sizes of other structural members. Fuel savings for lowered hauling loads could also provide sustainability benefits (Villarreal and Crocker 2007).

Three primary factors influence the effectiveness of internal curing, 1) the volume of water required from the LWA, 2) the ability of the LWA to release that water (desorption), and 3) the distribution of the LWA within the mixture (Henkensiefken 2009, Babcock and Taylor 2015). It has been found that replacing a fraction of conventional fine aggregate with prewetted LWA can successfully facilitate internal curing (Bentur et al. 2001, and many others). Aggregate spacing can be influenced by a number of factors including size of aggregates used and gradation. Since fine aggregate has a much higher surface area than coarse aggregate, and is well-distributed throughout a concrete mixture, internal curing using prewetted fine LWA has been found to be most effective in supplying internal curing moisture to hydrating cement particles (ACI 2013).

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 pre-wetted LWA is used instead of SSD LWA. 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% (Castro et al. 2011). 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 mixture proportioning for ICC. In North Carolina, expanded slate LWA and expanded shale LWA are commonly used in concrete construction.

Optimal use of prewetted LWA as an internal curing agent typically requires that the aggregate be in, or close to, a saturated condition prior to mixing. This condition can be achieved in the laboratory by either the paper towel method prescribed in ASTM C1761 (ASTM 2017) or by the centrifuge method described in Miller et al. (2014), the Indiana Testing Method (ITM) 222 (IDOT 2015)), or the New York State DOT Test Method NY 703-19E (NYSDOT 2008). SSD condition is achieved in the field, far less consistently, through the sprinkling and draining of aggregate stockpiles (NYSDOT 2009).

Water included in the void system of prewetted LWA is removed via capillary suction. Key to ensuring that water delivered into the concrete via the prewetted LWA is that the water release is delayed (Jones et al. 2014). The rate of the water release by the prewetted LWA has been found to be dependent on the size of the pores in the LWA. The pores need to be of a size where water is held within the aggregate during mixing (absorption), but released from the aggregate back into the paste after setting (desorption) (Jones et al. 2014). The characteristics of the internal pore structures of manufactured LWA vary based upon source geology and manufacturing process. The accepted method for determining desorption characteristics of an aggregate is outlined in ASTM C1498.

Any quantity of additional water available to facilitate internal curing will provide benefits to the surrounding cement paste. Mixture proportioning with ICC is quite similar to conventional concrete with the primary difference being the determination of the quantity of pre-wetted LWA to be substituted for normalweight fines. It is noted by Lopez et al. (2006, 2008) that the possibility exists of overdosing prewetted LWA. The addition of pre-wetted 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). Moderate replacement rates of LWA will also ensure that the concrete unit weight will remain above 135 pcf, eliminating the need for additional structural design considerations (ACI 2014a, 2014b, AASHTO 2012).

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 subsequently referred to as Equation 1.1 (ACI 2016, Bentz et al. 2005).

$$C_f \times CS \times \alpha_{max} = S \times \Phi_{LWA} \times M_{LWA} \tag{1.1}$$

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 'pre-wetted' condition, S, the measured sorption capacity of the internal reservoirs in the pre-wetted 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, ACI 2013).

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. 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 the full literature review in Appendix A, Section A.3.

Several states, including Texas, Colorado, Utah, Illinois, Indiana, Ohio, Virginia, West Virginia, and New York, have performed Division of Transportation (DOT) projects that utilize internally cured concrete. Many state agencies tend to simplify the Bentz and Snyder approach (equation 1.1) 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, NYSDOT and Illinois DOT specify that a 30% replacement of LWA for normalweight fines be utilized, regardless of the absorptive capacity of the LWA or the chosen cement content (Speck 2018). However, NYSDOT also specifies the use of a LWA with a minimum absorption capacity of 15%. Indiana DOT utilizes the Bentz and Snyder approach (Equation 1.1) to compute the LWA replacement, but indicates that in no cases should it be less than 30% of sand by volume (Speck 2018). West Virginia DOT also provides an approach to compute the amount of LWA to be included in the mixture, but states that in no case shall the volume of SSD LWA be less than 25% of the total fine aggregate SSD volume of the entire mix (WVDOT 2016). Louisiana DOTD specifies 225 to 275 pounds of prewetted LWA be used per cubic yard of concrete, with the normalweight sand reduced by the volume of the prewetted LWA added (Speck 2018) 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.

3. LABORATORY TESTING PROGRAM AND RESULTS

3.1 Materials Description and Characterization

3.1.1 Cementitous Materials

A Type I/II portland cement meeting ASTM C150 produced by a plant located in Holly Hill, South Carolina was used for this study. This cement is commonly used in concrete infrastructure projects in the Piedmont region of NC. The relative density of the cement is 3.15, and the mill report for the cement is provided in Appendix B in Figure B.1.

Several of the concrete mixtures prepared in the laboratory portion of this project utilized a partial replacement of fly ash for portland cement. The 2012 NCDOT Standard Specifications for Roads and Structures (current at the time of this study) allowed a 20% replacement of cement with Class F fly ash at a replacement rate of 1.2 lb of fly ash per lb of cement replaced. The fly ash used in this study is a Class F fly ash sourced from the Belews Creek power station in Belews Creek, NC. Further characterization information is provided in the Appendix B in Figure B.2.

3.1.2 Coarse Aggregates

Normalweight aggregate materials (both fine and coarse) used for the laboratory portion of this study were selected based upon input from NCDOT personnel regarding frequently utilized aggregates for concrete mixtures used in concrete infrastructure in the Piedmont region of NC. The normalweight coarse aggregate used is a No. 67 crushed granitic gneiss sourced from a quarry in Cary, NC, with a specific gravity of 2.62, an absorption of 0.8%, and a dry rodded unit weight of 96.5 pcf. Sieve analysis results are provided in Appendix B, Table B.1.

3.1.3 Fine Aggregates

The normalweight fine aggregate used for the laboratory portion of the study also comes from the Piedmont region of the state, a natural silica sand meeting ASTM C33 sourced from a pit in Lemon Springs, North Carolina. This natural sand has a density of 97.2 pounds per cubic foot, a specific gravity of 2.44 and absorption capacity of 0.18%. Sieve analysis results are provided in Appendix B, Table B.2 and Figure B.3, and indicated the sand has a fineness modulus of 2.72.

Prewetted fine LWA was utilized as the internal curing agent in this study. Two lightweight aggregate suppliers were selected to provide material for the project based on their ability to supply the NC market for lightweight aggregates. The first lightweight aggregate is an expanded lightweight slate produced in NC, referred to subsequently as Lightweight Aggregate 1 (LWA1). The second lightweight aggregate is an expanded shale produced in Kentucky, referred to subsequently as Lightweight Aggregate 2 (LWA2).

LWA used for this study was characterized for density, specific gravity and absorption in accordance with ASTM C1761. This standard provides a method for determining the absorption of lightweight aggregate known as the 'paper towel method,' which provides similar information to the 'cone method' of ASTM C128 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 high 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, and the time of testing can also be quite extensive. Therefore, an alternative test - the 'centrifuge method' – has been developed 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). Absorption capacities were determined by the 'paper towel method' for 24 hour absorption and by the centrifuge method for 24 and 72 hour absorption. The average results of absorption testing in the laboratory are presented below in Table 3.1.

Table 3.1: Absorption properties of LWA

Method	**ASTM C1761 Paper Towel Method**	**Centrifuge Method (Miller et al. 2014)**	
Absorption Time	**24 hour**	**24 hour**	**72 hour**
LWA1	13.02%	10.33%	11.33%
LWA2	22.40%	20.31%	23.94%

Other procedures in ASTM C128 were utilized to determine mechanical properties of LWAs including the density and specific gravity. Testing of LWA1 determined that this material has a dry rodded unit weight of 65 pounds per cubic foot and a specific gravity of 1.69. Testing of LWA2 yielded a density of 62 pounds per cubic foot and a specific gravity of 1.52. Results of sieve analyses performed per ASTM C136 are presented in Appendix B, Figure B.3. 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 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 conditions similar 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).

3.1.4 Admixtures

A commercially available synthetic air-entraining admixture (BASF MasterAir AE 200) and a mid-range water reducer (BASF MasterPolyheed 997, which also meets ASTM C 494 requirements for a Type F high-range water-reducing admixture) were utilized in all mixtures. NCDOT specifications for bridge deck mixtures allow a maximum slump of 3.5 inches. Some reasonable range of slump variation was anticipated as it was deemed important to maintain a consistent w/c ratio between different mixtures and between batches of the same mixture, and therefore the acceptable slump range for this project was established to be 1.5 to 3.5 inches.

All laboratory concrete mixtures prepared as part of this study used this AEA to achieve the desired air content, except the two mixtures that included latex (polymer) modification. Although NCDOT specifications allow an air content of (5.0% plus or minus 1.5%), a relatively tight allowable air content tolerance of 5.0% to 6.0% was utilized for all batches in order to ensure consistency between test results and to ensure that differences in laboratory test results could be mostly attributed to changes in materials, rather than changes in air content.

The LMC prepared as part of this project utilized a commercially available latex modifying admixture commonly utilized in NC (BASF Styrofan 1186). Review of approved mixtures submitted for LMC provided by NCDOT personnel 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.

3.2 Concrete Mixtures

3.2.1 Mixture Matrix

In collaboration with NCDOT, the research team targeted 18 different concrete mixtures to batch and test for this study. The mixture matrix provided in Figure 3.1 details concrete mixtures included in this study:

- Ten mixtures proportioned to meet **bridge deck Class AA mixture** specifications, with cementitious materials and LWAs varied to provide data comparing the performance of a variety of internally cured and control mixtures.
- Five **latex-modified concrete overlay mixtures** (three internally cured mixtures and two control mixtures).
- Two **very high early strength mixtures** (one internally cured mixture and one control mixture)
- One 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 (Cavalline et al. 2018).

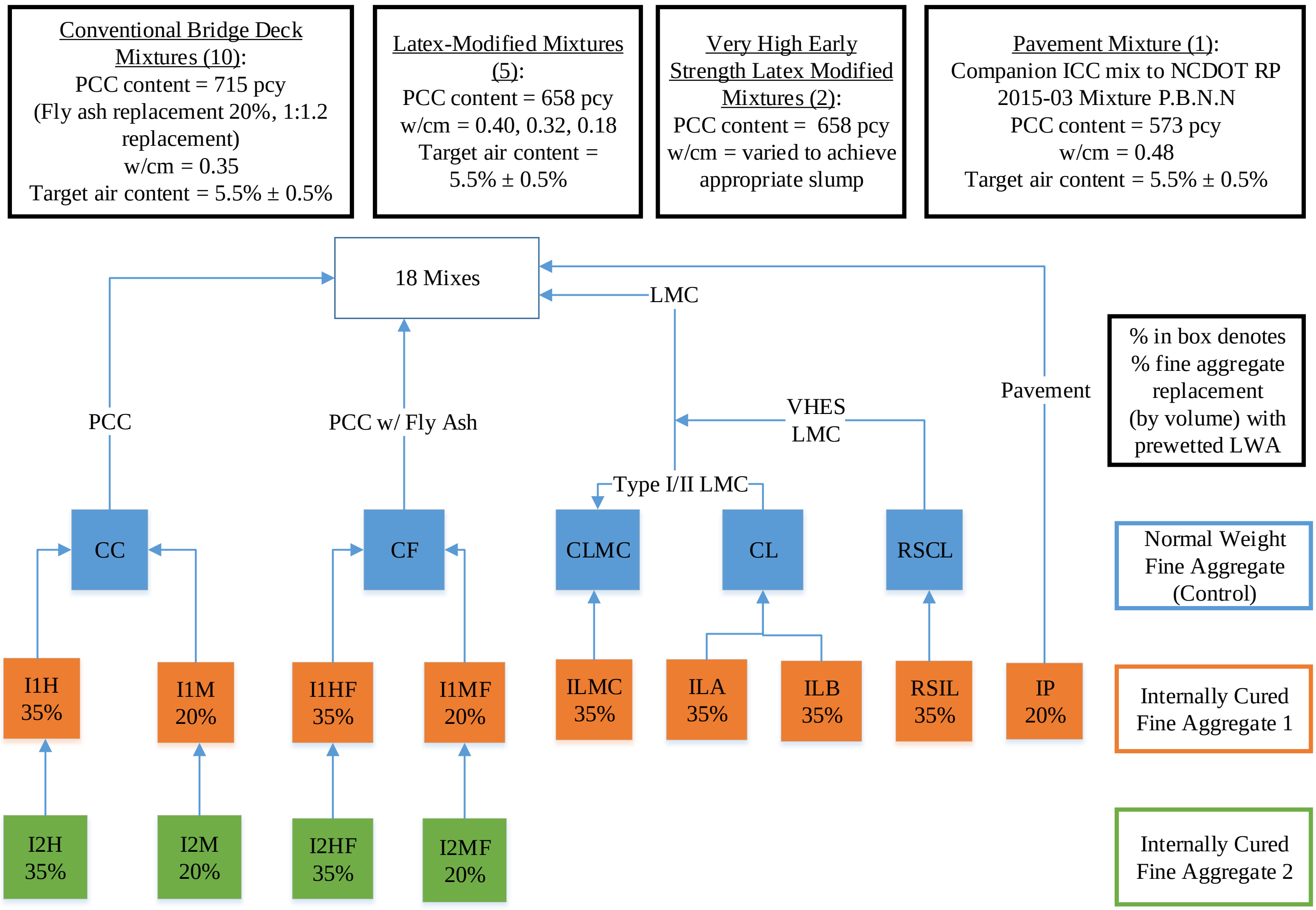

Figure 3.1: Concrete mixture matrix with supporting details

3.2.2 Mixture Design Approach for LWA and Mixture Proportions

To aid in identifying of optimal quantities of LWA replacement for use in future specification provisions, 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 pre-wetted LWA for normalweight sand. To proportion ICC using prewetted LWA, ACI 308-213 suggests use of the relationship developed by Bentz and Snyder (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 shown in Equation 3.1.

$$C_f*CS*\alpha_{max}=S*\Phi_{LWA}*M_{LWA} \qquad (3.1)$$

In this relationship, 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, αmax, 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, αmax, can be assumed to be 1 if the w/c is equal to or greater than 0.36. Otherwise, αmax is given by [(w/c)/0.36] for w/c < 0.36.

Based upon 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 such as LWA1. Converting this volume to a percentage of LWA1 to replace normalweight fine aggregate results in a 56.2% replacement, which exceeds the LWA replacement rates typically utilized by many state agencies currently specifying ICC for use in bridge deck concrete mixtures. **Based upon other state agencies experiences, it was determined by NCDOT to test internal curing mixtures in the laboratory with 20% and 35% replacement percentages of LWA for normalweight sand.** Mixture proportioning per ACI 211.1 procedure was performed to determine the batch quantities of other materials. The concrete mixture proportions are presented in Table 3.2. Batch weights in Table 3.2 assume that the LWA is in SSD condition and the normalweight aggregate (coarse and fine) is oven dry.

Conventional bridge deck mixtures

Parameters held constant for all 12 conventional bridge deck mixtures included a w/cm of 0.35, a coarse aggregate factor of 0.66, a target air content of 5.5% ± 0.5%, a maximum slump without water reducing admixture of 1.5 inches, and a target slump with a mid- range water reducing admixture of 1 to 4 inches. NCDOT provided a range of cement contents commonly used in bridge deck mixtures within the state. This ranged 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 3.1, 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 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.

Latex-modified mixtures

LMC concrete mixtures utilized Type I/II cement, and since LMC overlays are typically fairly thin, #78 coarse aggregate, rather than #67 coarse aggregate, was used in the LMC (sourced from the same quarry). Internally cured LMC concrete mixtures contained prewetted LWA1 at a 35% replacement rate. In these mixtures, 210.2 pcy of latex modifying admixture (48% polymer, 52% water) was used in addition to the amount of water listed in Table 3.2. Initially, two LMC mixtures were batched using the proportions most commonly submitted to NCDOT for approval, one control (CLMC) and one internally cured (ILMC), with a w/cm of 0.40. When batched, slumps for CLMC and ILMC significantly exceeded NCDOT requirements (3 to 6 inches). Conversations with BASF representatives indicated that, in practice, most suppliers were withholding a significant amount of water from the approved mixture design. To provide insight into the performance of these LMC mixtures as-batched and placed, three supplemental mixtures of the Type I/II LMC were also batched and tested. The research team re-batched the control LMC mixture (CLMC) using the typical NCDOT-approved proportions, withholding water to achieve a 3 to 4 inch slump (mixture CL, with a w/cm of 0.32). A corresponding internally cured LMC mixture (ILA) was also batched at the same w/cm ratio, 0.32, facilitating direct performance comparison with mixture CL. However, this internally cured mixture again resulted in an unacceptably high slump (about 8 inches), likely due to the increased workability provided by the inclusion of the prewetted lightweight aggregates (LWA). Therefore, a third internally cured LMC mixture (ILB) was batched, withholding water until a 3 to 4 inch slump was achieved (w/cm = 0.18). Although not directly comparable to the other mixtures due to the much-reduced w/cm ratio, this mixture provided insight into the potential performance improvements that can be gained by using prewetted LWA for IC LMC due to the reduction in water demand facilitated by the increased workability.

Very high early strength latex modified mixtures

Two very high early strength VHES LMC were batched with CTS Rapid Set Cement, and are representative of a latex-modified overlay mixture used in locations where short term lane closures are required. These VHES LMC concrete mixtures also utilized #78 coarse aggregate, rather than #67 coarse aggregate (sourced from the same quarry). In these mixtures, 210.2 pcy of latex modifying admixture (48% polymer, 52% water) was used in addition to the amount of water listed in Table 3.2. A technical support representative from CTS Rapid Set Cement visited UNC Charlotte’s laboratories to observe and guide batching of the VEHS LMC (a condition required by CTS Cement as part of the material donation). As shown in Figure 3.1, one mixture was a control mixture with natural silica sand (RSCL), and the other was an IC mixture with 35% replacement of natural sand with prewetted LWA (RSIL).

Pavement mixtures

The materials and mixture proportions of the internally cured pavement mixture IP (shown in Figure 3.1) correspond to a control pavement mixture from another study (NCDOT RP 2015-03, mixture P.B.N.N.), with a lower cement content (573 pcy compared to 715 pcy) and a higher w/c ratio (0.48 compared to 0.35) than the bridge deck mixtures included in the study. The internally cured pavement mixture batched as part of this study (IP) utilized the same coarse aggregate, normalweight fine aggregate and cement source as mixture P.B.N.N from NCDOT RP 2015-03 (Cavalline et al. 2018, Blanchard 2016, Chimmula 2016, Medlin 2016). Pre-wetted LWA1 was substituted at a 20% volumetric replacement for normalweight fine aggregate for the internal curing pavement mixture.

Table 3.2: Mixture proportions for laboratory testing

Mixture Type	ID	Description	Weight (lb/cy)					
			Cement	Fly Ash	Water	CA	NWFA	PWLA
Conventional Bridge Deck	CC	Conventional concrete	715	0	266.0	1720	1113	0
	I1M	Internal curing, LWA #1, moderate replacement	715	0	265.6	1720	890	154
	I2M	Internal curing, LWA #2, moderate replacement	715	0	265.6	1720	890	139
	I1H	Internal curing, LWA #1, high replacement	715	0	265.3	1720	723	270
	I2H	Internal curing, LWA #2, high replacement	715	0	265.3	1720	723	243
	CF	Conventional concrete, with fly ash	572	172	266.0	1720	1113	0
	I1MF	Internal curing, LWA #1, moderate replacement, with fly ash	572	172	265.6	1720	890	154
	I2MF	Internal curing, LWA #2, moderate replacement, with fly ash	572	172	265.6	1720	890	139
	I1HF	Internal curing, LWA #1, high replacement, with fly ash	572	172	265.3	1720	723	270
	I2HF	Internal curing, LWA #2, high replacement, with fly ash	572	172	265.3	1720	723	243
LMC	CLMC	Conventional latex-modified concrete	658	0	153.3*	1304**	1510	0
	ILMC	Internally cured latex-modified concrete, LWA #1, high replacement	658	0	153.3*	1304**	921	345
	CL	Conventional latex-modified concrete, w/cm to achieve desirable slump	658	0	101.3*	1304*	1510	0
	ILA	Internal curing latex-modified concrete, LWA #1, w/cm to match mixture CL	658	0	101.3*	1304*	921	345
	ILB	Internal curing latex-modified concrete, LWA #1, high replacement, w/cm to achieve desirable slump	658	0	9.1*	1304*	921	345
VHES LMC	RSCL	Very high early strength conventional latex-modified concrete	658	0	121.0*	1304*	1510	0
	RSIL	Very high early strength internal curing latex-modified concrete, LWA #1, high replacement	658	0	81.5*	1304*	921	345
Pavement	P.A.N.N	Conventional pavement mixture from RP 2015-03	573	0	298.2	1798	1184	0
	IP	Internal curing pavement mixture, LWA #1, moderate replacement	573	0	298.2	1798	770	252

- First Letter: ‘C’ indicates conventional concrete mixture, ‘I’ indicates an internally cured concrete mixture.
- For internally cured concrete mixtures (mixtures starting with ‘I’):
  - The number indicates which LWA was used: ‘1’ for LWA #1, and ‘2’ for LWA2.
  - The letter ‘M’ indicates that pre-wetted LWA was substituted at a moderate replacement level of 20% while an ‘H’ indicates that pre-wetted LWA was substituted at a high replacement level of 35%.
  - The letter ‘F’ indicates mixtures that contain fly ash while the absence of the letter ‘F’ indicates no fly ash was used (straight cement mixtures).

3.2.3 Batching Procedures

The batching, mixing, transportation, placing and finishing of ICC is not significantly different from approaches used for conventional concrete (ACI 308-213). 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 challenge for successful ICC implementation. For batching of ICC 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 48 to 72 hours. As batching of laboratory specimens typically occurred in the morning, the LWA was drained at the end of the previous workday. 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, placed on a piece of plastic sheeting (to avoid contamination) in the shape of a small stockpile, and then allowed to drain overnight, for a period of 12 to 15 hours.

During the morning of batching, the LWA stockpile was "turned" several times using a shovel. Care was taken to not disturb the bottom 2 to 3 inches of the stockpile as this was where the drained water collected. Free water present in the LWA was determined using the centrifuge method as described by Miller et al. (2014), so that the batch water could be adjusted. After several iterations of laboratory batching and mixing, it was observed that the laboratory stockpile management procedures described above resulted in reasonably consistent surface moistures for each type of LWA. Therefore, an average surface moisture for each LWA was calculated and used 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.

3.3 Testing Program and Results

The testing program for fresh and hardened concrete properties is summarized in Table 3.3. After the LWA prewetted and conditioned, batching was performed in general accordance with ASTM C685. For most mixtures, due to constraints related to the mixer drum size in relation to the required volume of concrete required to produce enough concrete to facilitate casting of all specimens, each mixture was batched three times. Slump and air content were measured for each batch, and mixtures that did not meet the slump and air tolerances were discarded. To verify consistency in batches, compressive strength tests were performed on cylinders prepared from each batch. Batches were utilized as follows:

- Batch 1: compressive strength, modulus of elasticity, Poisson's ratio, surface resistivity, bulk conductivity, and rapid chloride penetration test (RCPT)
- Batch 2: drying shrinkage, freeze thaw testing, and Super Air Meter
- Batch 3: cracking potential rings

Table 3.3: Testing Program for Laboratory-Batched Mixtures

	Test	Protocol	Age(s) in days	Replicates
Fresh	Air content	Pressure meter (ASTM C231) SAM (AASHTO TP 118-17)	Fresh	Type B meter each batch, SAM Batch 2
	Slump	ASTM C143	Fresh	1
	Fresh density (unit weight)	ASTM C138	Fresh	1
	Temperature	AASHTO T309	Fresh	1
Hardened	Compressive strength	ASTM C39	3, 7, 28, 90	3 each age
	Resistivity	AASHTO T 358-17	3, 7, 28, 90	3 each age
	Modulus of elasticity and Poisson's ratio	ASTM C469	28	2
	Coefficient of thermal expansion	AASHTO T336	28	3
	Shrinkage	ASTM C157	per standard	3
	Cracking potential	ASTM C1581	per standard	3
	Autogenous Shrinkage	ASTM C1698	per standard	3
	Rapid chloride permeability	ASTM C1202	28	2
	Freezing and thawing resistance	ASTM C666, procedure A	per standard	3
	Modulus of rupture * pavement mixture only	ASTM C78	28	2

3.3.1 Conventional Bridge Deck Mixtures

3.3.3.1 Fresh Concrete Properties

The slump and air content of all bridge deck mixtures met NCDOT standard specifications. A summary of test results for each fresh concrete property test for each batch is presented in Appendix B, in Table B.3. To mitigate the influence of a wide range of air contents on the test results, air contents for these mixtures was restricted to to 5.0% to 6.0%. This relatively tight acceptable air content resulted in the wasting of a number of batches of concrete for air contents outside of this narrow range. However, review of the test results indicates that this was a sound decision, as general trends likely attributable to materials (and not air content differences) are evident in hardened concrete test results.

A summary of results for the unit weight of fresh concrete and total air content are shown in Figure 3.2. As 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 this result. Mixtures that contained fly ash had a slightly lower unit weight than mixtures that did not contain fly ash. 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.

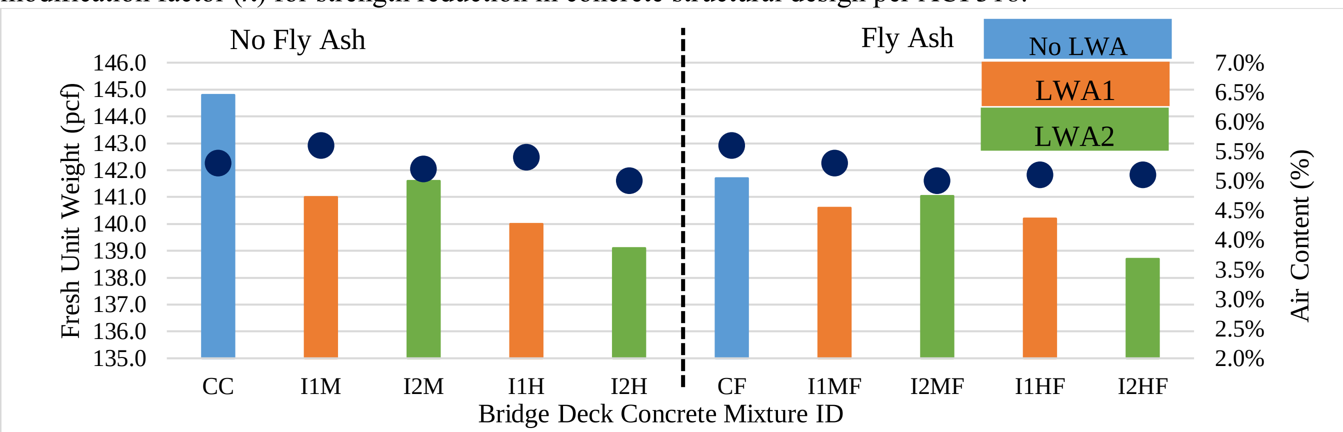

Figure 3.2: Fresh unit weight and average air content of bridge deck mixtures

3.3.1.2 Mechanical and Thermal Properties

Mechanical and thermal property tests were performed using the test methods outlined in Table 3.2. A summary of these results is provided in Table 3.4, with a discussion of results presented in subsequent sections. Supporting data, providing the result of each test and averages and standard deviations is provided in Appendix B, Tables B.4 through B.7.

Compressive Strength

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 3.3 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 showed the typical increase in later-age strength achieved by the fly ash mixtures.

Table 3.4. Results of laboratory testing for mechanical and thermal properties

Mixture ID	Compressive Strength (psi)				MOE (psi)	Poisson's Ratio	CTE (in./(in.×°F))
	3 day	7 day	28 day	90 day	28 day	28 day	28 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	0.22	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}

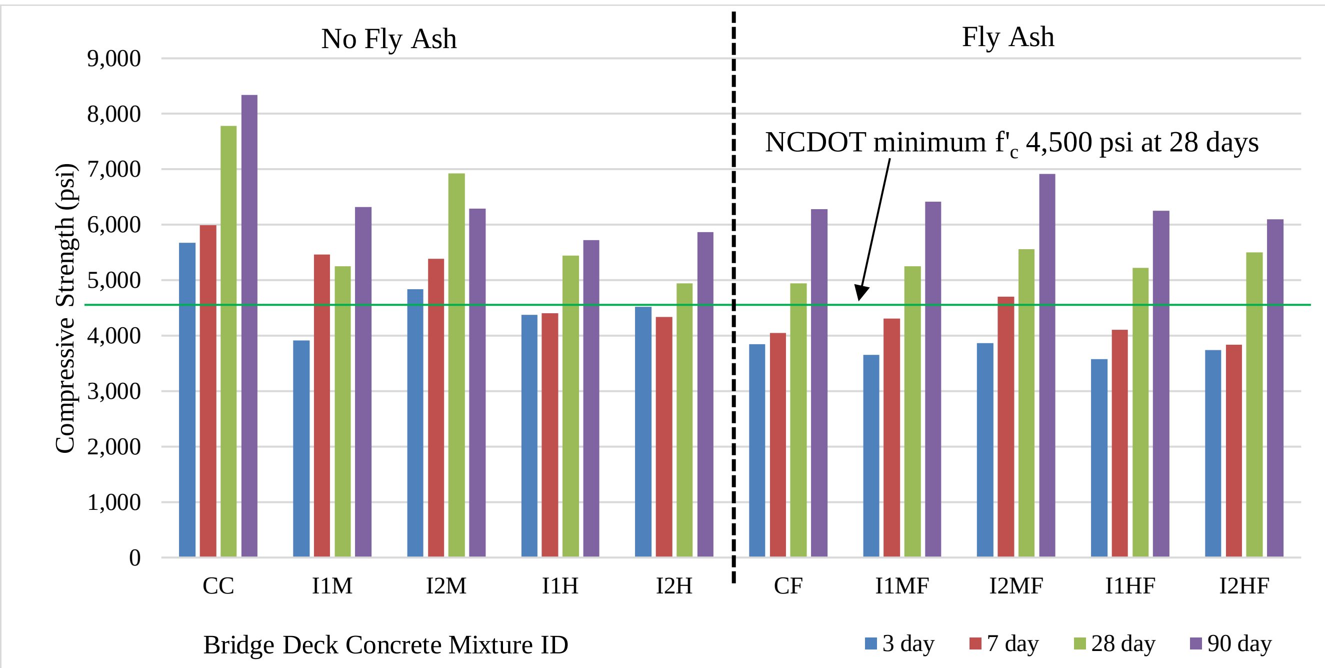

Figure 3.3: Compressive strength results of bridge deck mixtures

Modulus of Elasticity and Poisson's Ratio

The results of modulus of elasticity (MOE) tests ranged from 3,050,000 psi to 4,390,000 psi. If the conventional concrete without fly ash mixture (CC), 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 3.4, 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, E_c 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 3.4, 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 observed in Figure 3.4, 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 using these local materials in future structures and pavements.

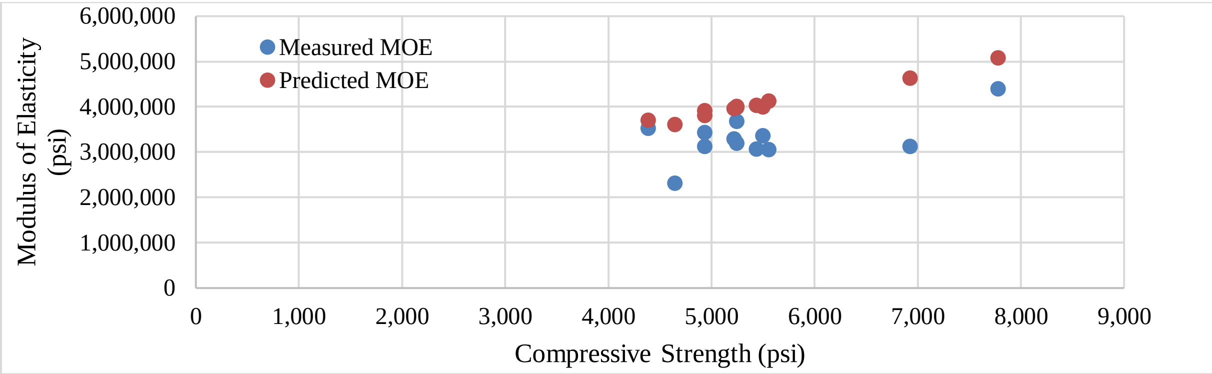

Figure 3.4: Measured MOE compared to predicted MOE calculated from ACI 318

Coefficient of Thermal Expansion

A figure illustrating the differences in CTE, including range bars, is presented in Figure 3.5. Other research has reported that lightweight aggregate concretes tend to have a lower CTE than those using normalweight aggregate (Maruyama and Teramoto 2012, Castrodale and Cavalline 2017). 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.

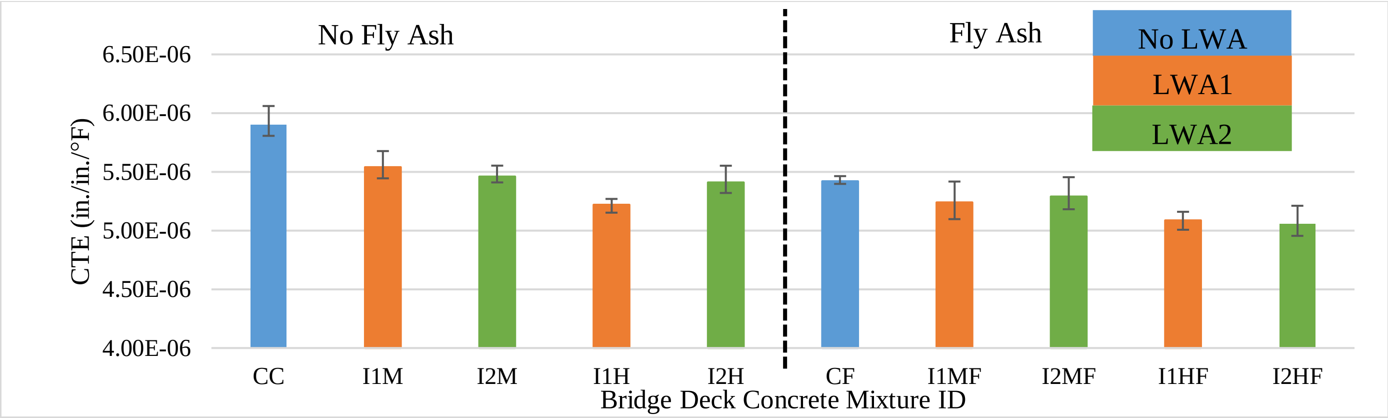

Figure 3.5: Coefficient of thermal expansion (CTE) results

3.3.1.3 Durability Performance Tests

Although concrete is an inherently durable material, its long term performance is strongly linked to permeability, since the transport of aggressive and deleterious agents is typically facilitated by water (Mindess et al., 2003). The most substantial permeability-related durability issue for concrete infrastructure is reinforcing steel corrosion. 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 3.5, and as could be expected, 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. Results for all tests are provided in Table B.6 through B.12 in Appendix B.

Table 3.5: Summary of durability performance test results

Mixture ID	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	9.9	6.1	2,930	2,190
I1M	9.9	10.6	13.7	14.7	13.9	6.3	3,380	2,620
I2M	9.3	10	13.3	16.7	12.5	9.1	3,180	2,550
I1H	9.3	10.7	13.6	14.7	10.2	6.2	3,220	2,890
I2H	8.1	9.1	12.5	14.4	11.6	5.2	2,920	2,790
CF	10.4	11.1	18.1	46.1	9.9	3.7	2,440	590
I1MF	8.8	10.3	17.5	45.7	9.5	3.3	2,140	590
I2MF	7.4	8.1	17	36.9	9.3	4.1	2,710	960
I1HF	7.1	9.2	17	43.3	11.6	3.6	2,470	810
I2HF	6.9	7.6	16.5	43.6	10.1	3.4	2,320	750

Surface Resistivity

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 expected to have better durability performance. Surface resistivity test results are presented in Appendix B, Table B.8, and are graphically summarized below in Figure 3.6. Results show that the conventional concrete mixtures (CC and CF) possessed a slightly higher surface resistivity than the corresponding internally cured mixtures at each test age. However, the slight difference could likely be attributed to the extra water contained in the LWA within the internal curing mixtures, since water conducts electricity. Results also show that use of fly ash provides the significant benefit of increased surface resistivity at 28 and 90 days. All concrete mixtures tested in this study, both conventional and internal curing, fell into an AASHTO T 358-17 (Table 3.6) permeability classification of moderate at 28 days. 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.

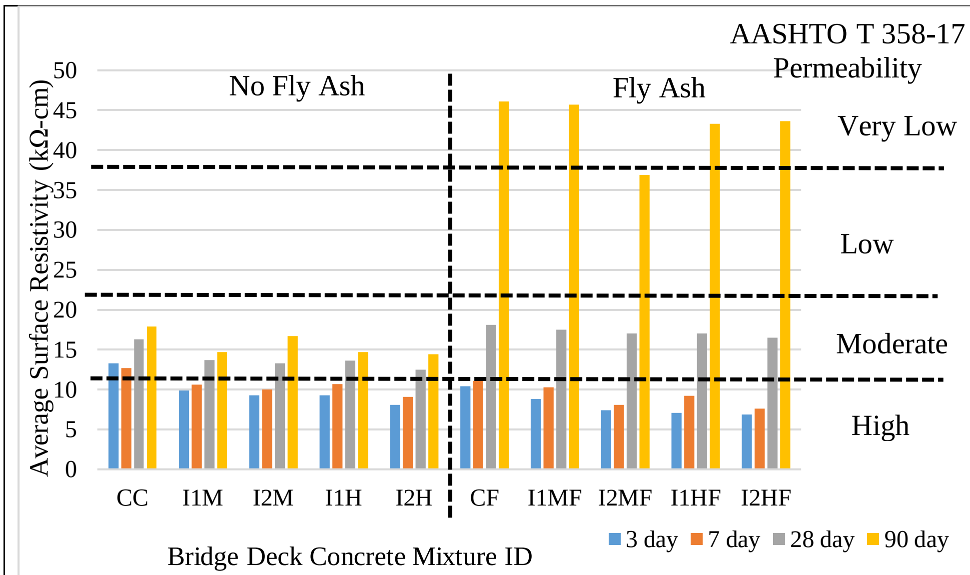

Figure 3.6: Average surface resistivity test results

Table 3.6: AASHTO T 358-17 surface resistivity index

Resistivity measured with 4”x8” Cylinder (kΩ·cm)	Chloride Ion Permeability
<12	High
12-21	Moderate
21-37	Low
37-254	Very Low
>254	Negligible

Bulk Conductivity

Bulk conductivity results are presented in Appendix B, Table B.9, and are graphically summarized below in Figure 3.7, with the average of three specimens plotted at each age. Test results show that the bulk conductivity did not significantly change due to incorporation of prewetted LWA to promote internal curing. Similar to surface resistivity results, several 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 consistent for the ten 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.

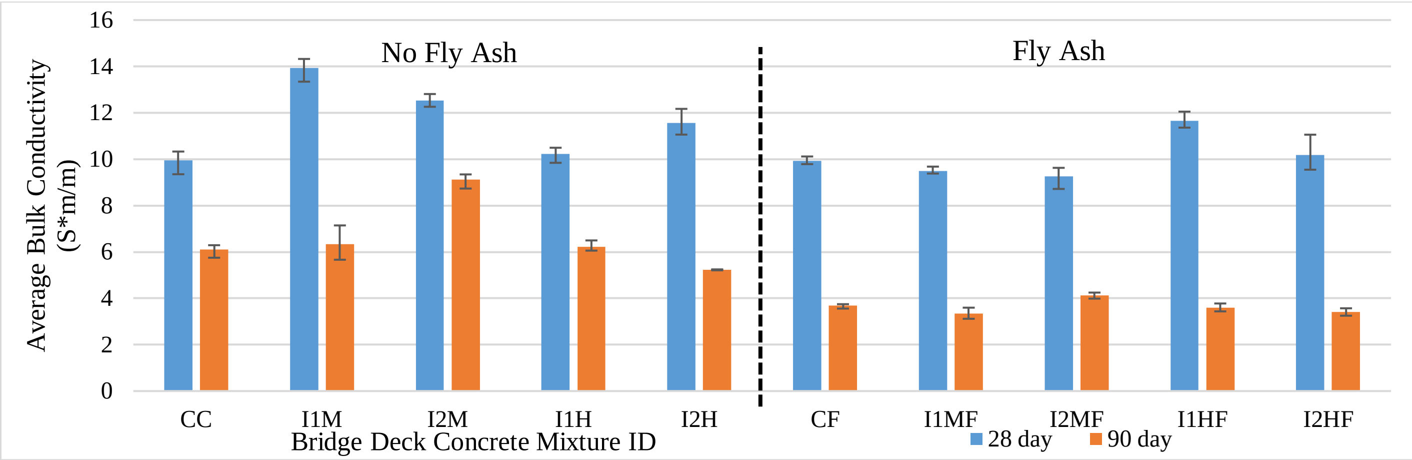

Figure 3.7: Average bulk conductivity test results

Rapid Chloride Permeability Test (RCPT)

RCPT results are presented in Appendix B, Table B.10, and are graphically summarized below in Figure 3.7, with the results shown as the average of three specimens tested for each mixture at 28 days and 90 days of age. 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, as expected, 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 3.8 provides an analysis of RCPT results and the corresponding permeability classifications (listed in Table 3.7), and it can be seen that RCPT test results correlate well with the results of surface resistivity and bulk conductivity testing.

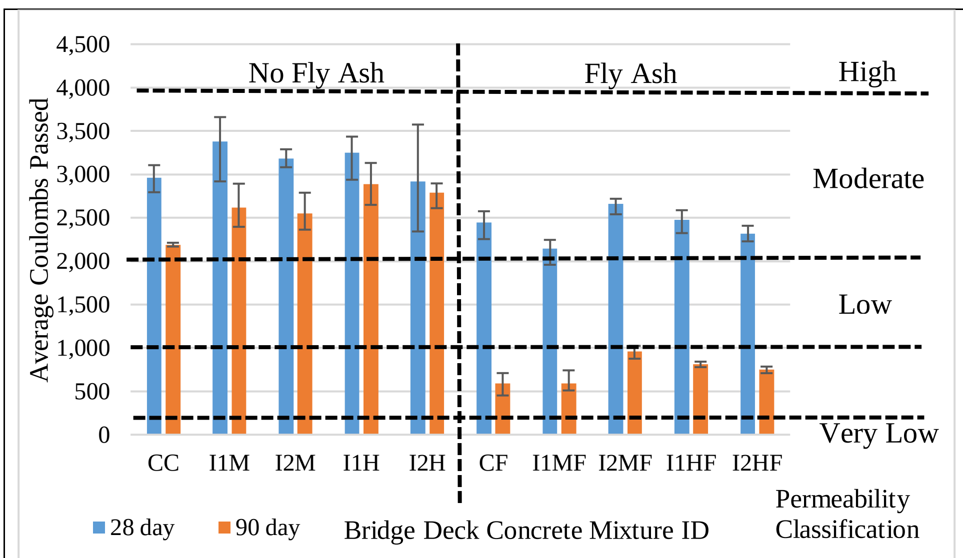

Figure 3.8: Average rapid chloride permeability test results

Table 3.7: ASTM C1202 chloride ion penetrability based on charge passed

Charge passed (coulombs)	Chloride Ion Permeability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very Low
<100	Negligible

At 28-days of age, all mixtures were within the range of 2,000 to 4,000 coulombs passed, classified as moderate permeability. 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 3.9, 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 appears 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.

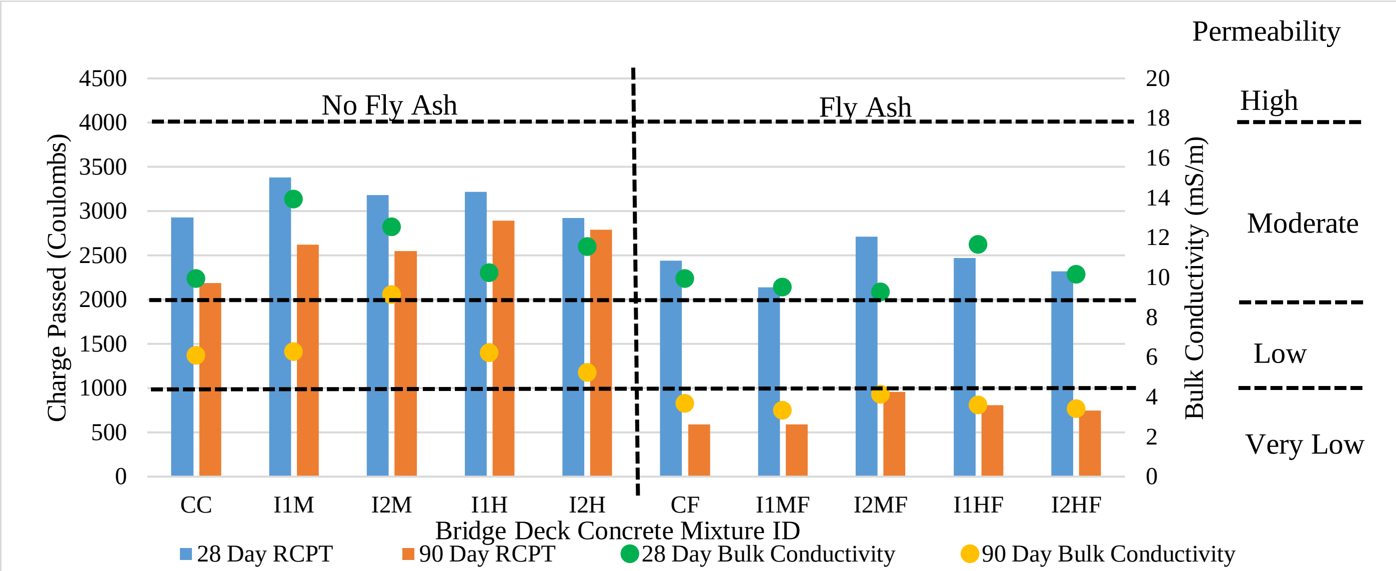

Figure 3.9: RCPT and bulk conductivity comparison

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 3.10. Plotting results of testing from this work against the relationship proposed by Rupnow and Icenogle (2011) 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.

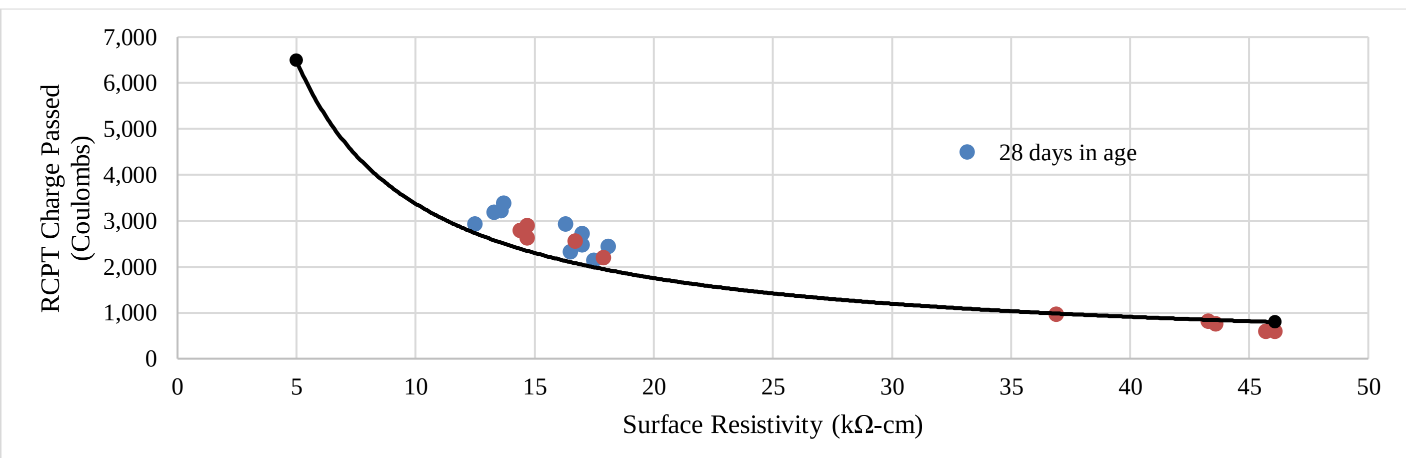

Figure 3.10: Surface resistivity vs. RCPT and Rupnow and Icenogle (2011) prediction model

Freeze-thaw durability

Freeze thaw test results are provided in Appendix B, Table B.11, and are graphically summarized below in Figure 3.11. Many state agencies specify that to be freeze-thaw durable, concrete mixtures should have a durability factor no less than 80 at 300 cycles, although failure criteria ranging between 60 and 95 are found in specifications. Non-internally cured mixtures (mixtures CC and CF) and mixtures using LWA1 performed satisfactorily. The use of LWA1 for internal curing (with an appropriate entrained air void system) seemed to provide improved freeze-thaw performance, as evident by the consistently higher durability factor of the I1 mixtures compared to the non-IC mixtures CC and CF. Of note, freeze thaw testing showed that fly ash mixtures with LWA2 did not perform as well. Batch I2HF was tested twice following a poor result, and again showed a significant amount of mass loss and low durability factor. Mixture I2M, however, exhibited satisfactory performance.

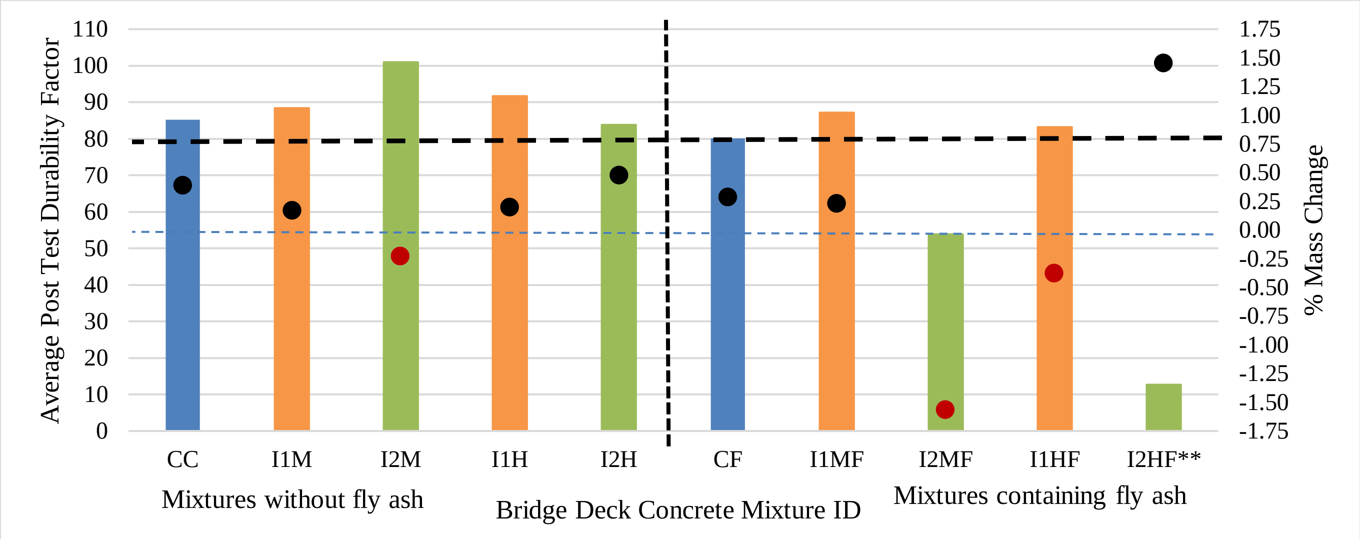

Figure 3.11: Freeze-thaw test results

Unrestrained (Drying) Shrinkage

A summary of ASTM C157 unrestrained shrinkage ("drying shrinkage") test results for the ten bridge deck mixtures is shown in Figure 3.12. Results provided are the average of three specimens, with supporting data provided in Appendix B in Table B.12. Similar to the findings of other researchers (Ardeshirilajimi et al. 2016), ASTM C157 length changes for ICC mixtures did not differ significantly from those of the control mixtures both with and without fly ash. 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.

Results tabulated in Figure 3.12 and Table 3.8 indicate that at 28 days, all mixtures had length changes less than 0.04% (or roughly 400με). In research for Virginia DOT, 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 fall beneath these thresholds. Drying shrinkage performance guidance suggested (for pavements) in AASHTO PP 84-17 is 420με at 28 days. The ten bridge deck mixtures range from 255 to 348 με at 28 days which, although a pavement specification, passes this prescriptive guidance.

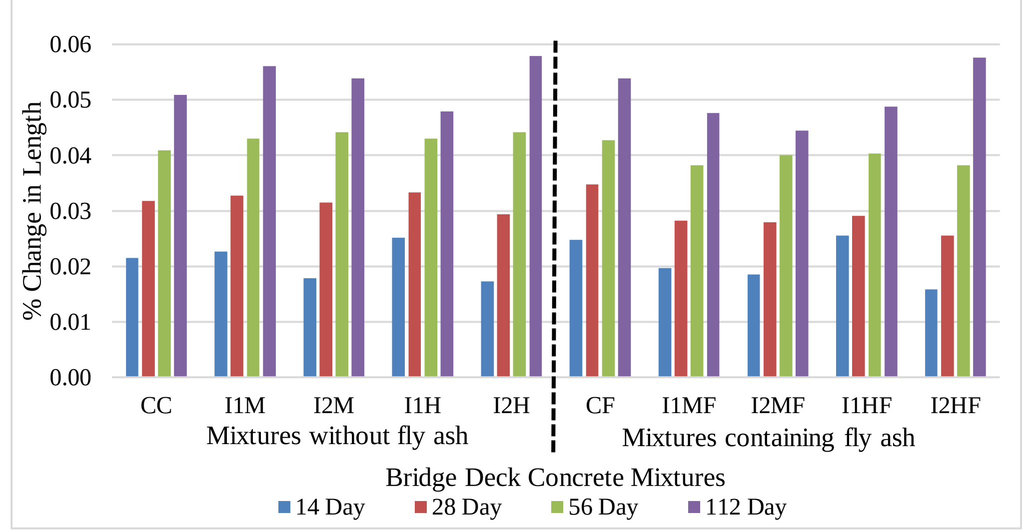

Figure 3.12 Summary of ASTM C157 shrinkage tests

Table 3.8: Strain at 28-days of drying, ASTM C157

Mixture ID	Average 28-day microstrain
CC	318
I1M	327
I2M	315
I1H	333
I2H	294
CF	348
I1MF	282
I2MF	279
I1HF	291
I2HF	255

Restrained Shrinkage

Restrained shrinkage tests, 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. Results from ASTM C1581 cracking potential tests are shown in Table 3.9. Three specimens were tested for each mixture, with four strain gages mounted on each testing apparatus. Strain data for each specimen was plotted against time, with a sudden release of strain associated with crack formation in the specimen. An example of this plot, showing time to cracking, is provided in Appendix B Figure B.4 (for Mixture CC3, Specimen 2).

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. 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. 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. An example of this plot (for Mixture CC3, Specimen 2) is provided in Appendix B, Figure B.5. The three stress rates are averaged to determine an average stress rate for each concrete mixture, measured in psi/day. The stress rate of the specimen, q, is then determined by Equation 3.2, a function of average strain rate, a_{avg}, geometry of the concrete and steel ring, G, and time to cracking, t_r.

$$q = G \left| a_{avg} \right| / 2\sqrt{t_r} \tag{3.2}$$

A summary of these results are provided in Appendix B, Table B.13, and plots of the raw data for all specimens are presented in Appendix B, in Figures B.4 through B.38. Plots of strain vs. time and strain vs. the square root of time and linear regression for each specimen are presented in the Appendix in Leach (2017).

As shown in Table 3.9, the time to cracking ranged from 0 days to no cracking at 28 days, when the test was concluded. The time to cracking of conventional mixes was earlier than internally cured mixes. Average stress rates, along with the ranges exhibited by the three specimen for each mixture, are shown in Figure 3.13. The conventional concrete mixture without fly ash developed the highest average stress rate, while 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 3.13 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 may be useful to extend this analysis.

Table 3.9: ASTM C1581 cracking potential test results

Mixture ID	Average Pre-crack Rate of Strain (psi/day)				Time to Cracking (Days)				Duration of Test (days)
	Specimen			Average	Specimen			Avg.	
	1	2	3		1	2	3		
CC	11.2	12.6	9.3	11.0	13.0	11.0	13.8	12.6	28
I1M	9.9	9.3	10.7	10.0	20.5	17.5	12.5	16.8	28
I2M	8.7	12.7	4.3	8.6	15.0	17.5	N.C.	16.3	28
I1H	10.2	11.5	10.0	10.6	27.3	18.8	N.C.	23.0	28
I2H	3.1	5.9	4.9	4.6	N.C.	N.C.	23.8	23.8	28
CF	12.3	4.1	7.3	7.9	10.3	N.C.	25.5	17.9	28
I1MF	11.4	10.3	6.3	9.3	18.8	25.5	N.C.	22.1	28
I2MF	5.7	10.4	6.8	7.6	N.C.	14.3	17.3	15.8	28
I1HF	7.7	4.6	6.0	6.1	26.0	N.C.	N.C.	26.0	28
I2HF	8.3	10.8	8.1	9.1	19.5	15.0	24.8	19.8	28

Key
Early crack (<=14 days)
Late crack (14-28 days)
N.C. = Did not crack at 28 days

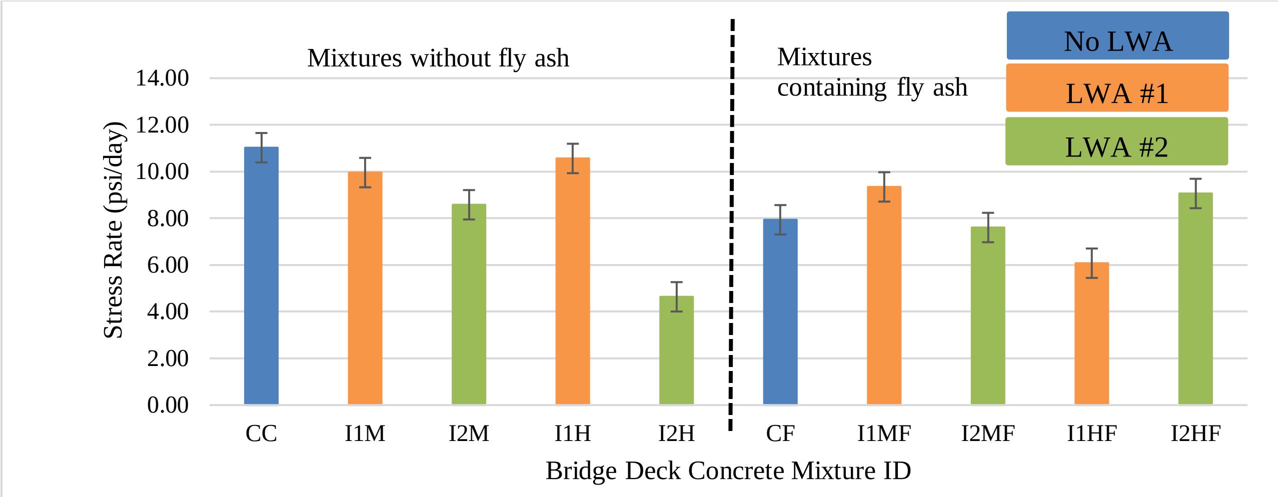

Figure 3.13: Restrained shrinkage average stress rate results

Autogenous Shrinkage

The primary benefit of using ICC is 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, theoretically reducing shrinkage and the potential for cracking (Henkensiefkin et al. 2009). 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. This test method utilizes mortar specimens measured frequently for length change using a dilatometer, pictured in Appendix B, Figure B.39. The mortar specimens consist of the same proportions of materials as the concrete mixtures designs shown in Table 3.2, with the exception being the removal of the coarse aggregate and the use of no chemical admixtures. Autogenous shrinkage specimens have a geometry similar to that shown in Appendix B, Figure B.40. 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.

A summary of autogenous shrinkage test results is shown in Figure 3.14, with supporting data provided in Appendix B, Table B.14. In this figure, the impact of use of prewetted LWA for internal curing is clearly evident. The autogenous strain of the non-internally cured mixtures with and without fly ash (CC and CF) is clearly higher than the strain exhibited by all other internally cured mixtures. The replacement rate appeared to be significant in reducing autogenous strain for LWA2, but not for LWA1. The lowest autogenous shrinkage strain was exhibited by mixtures using LWA2 at the higher replacement rate. Unrestrained shrinkage rate test results (shown in Figure 7) did not clearly show performance improvements for internally cured concrete bridge deck mixtures. These findings are consistent with those by other researchers, who indicate that internal curing effects will be more evident in autogenous shrinkage tests than other drying shrinkage tests (such as unrestrained volume shrinkage per ASTM C157). 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).

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).

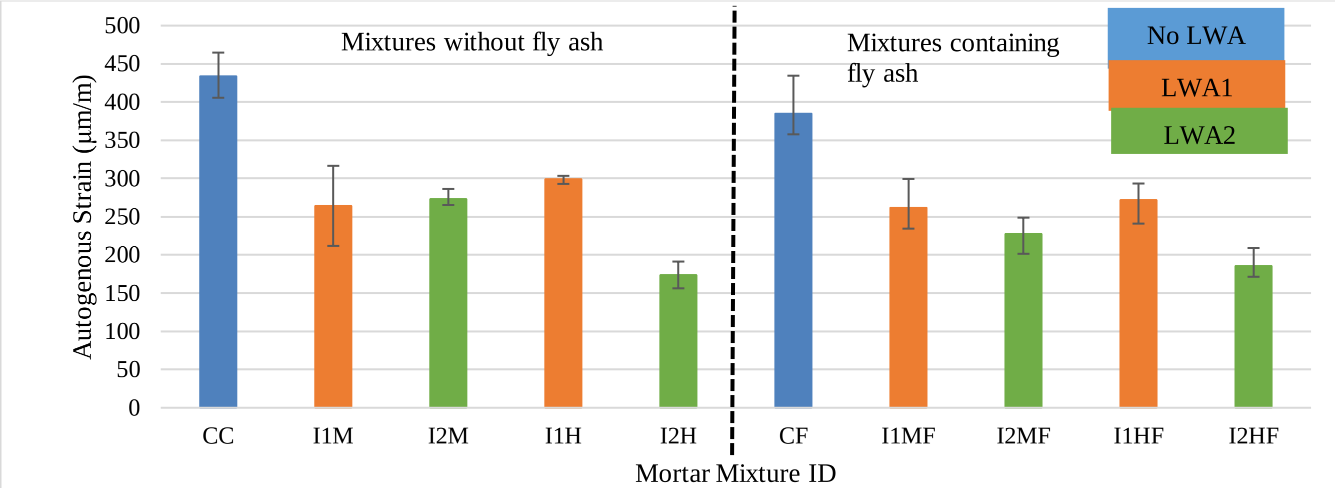

Figure 3.14: Average autogenous strain results

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 3.15). 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), indicating these tests can reasonably be used to evaluate internal curing.

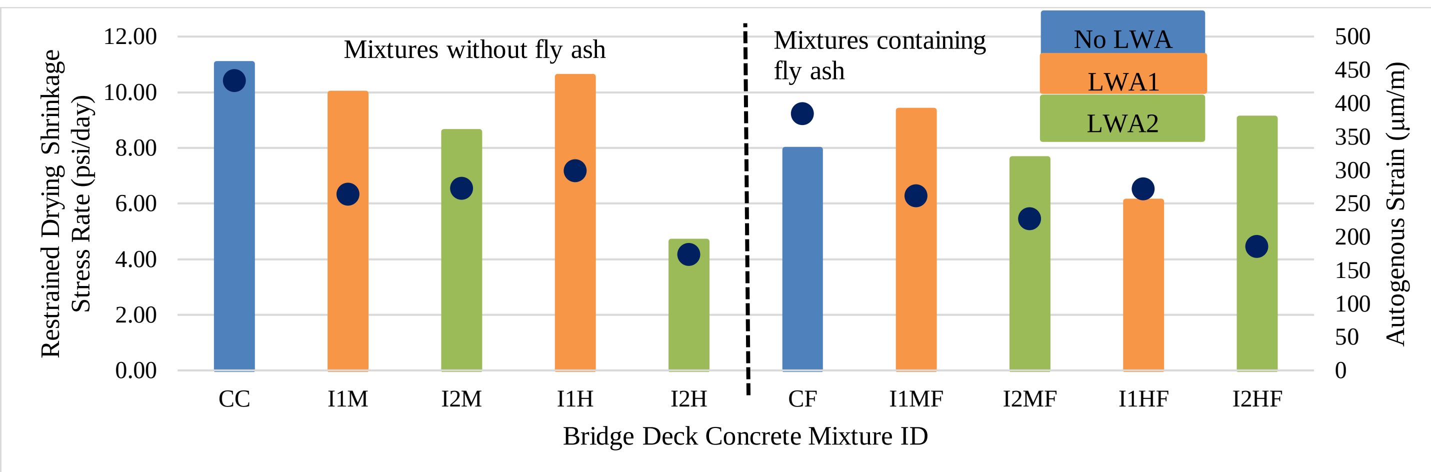

Figure 3.15: Restrained drying shrinkage compared with autogenous strain

Research by NIST (Bentz and Snyder 1999) provided an often-utilized approach to determine the exact mass of saturated LWA needed to supply the water demand (ACI 2013). As described in Section 3.2.2, due to the absorption capacity of the LWA this approach results in the suggested replacement rate of LWA1 of 56.2% replacement of conventional sand (significantly higher than many state agencies utilize). The suggested replacement rate of LWA2 is almost identical to the 35% replacement rate selected as the High (H) replacement rate for this study. Of interest for this study was the potential reduction in autogenous shrinkage of IC concrete batched with prewetted LWA1 at this high replacement rate suggested by the Bentz and Snyder method. Therefore, additional autogenous shrinkage tests were performed on specimens of IC mortars batched with LWA 1 and LWA 2 incorporated at replacement rates computed with the Bentz and Snyder (1999) approach. Results from testing of these mixtures, denoted as I1SH, I2SH, I1SHF, and I2SHF, are shown in Figure 3.16 along with the previously reported test results for the 20% (M) and 35% (H) replacement rates. Mixtures internally cured with LWA1 using the very high replacement rate computed with the Bentz and Snyder approach (I1SH and I1SHF) had significantly reduced autogenous shrinkage, with the I1SHF showing the lowest autogenous shrinkage of all mixtures. Mixtures internally cured with LWA2 at the Bentz and Snyder replacement rate (I2SH and I2SHF), which was fairly consistent with the 35% replacement rate used as the high replacement rate for this study, performed relatively similar to mixtures (I2H and I2HF), as could be expected.

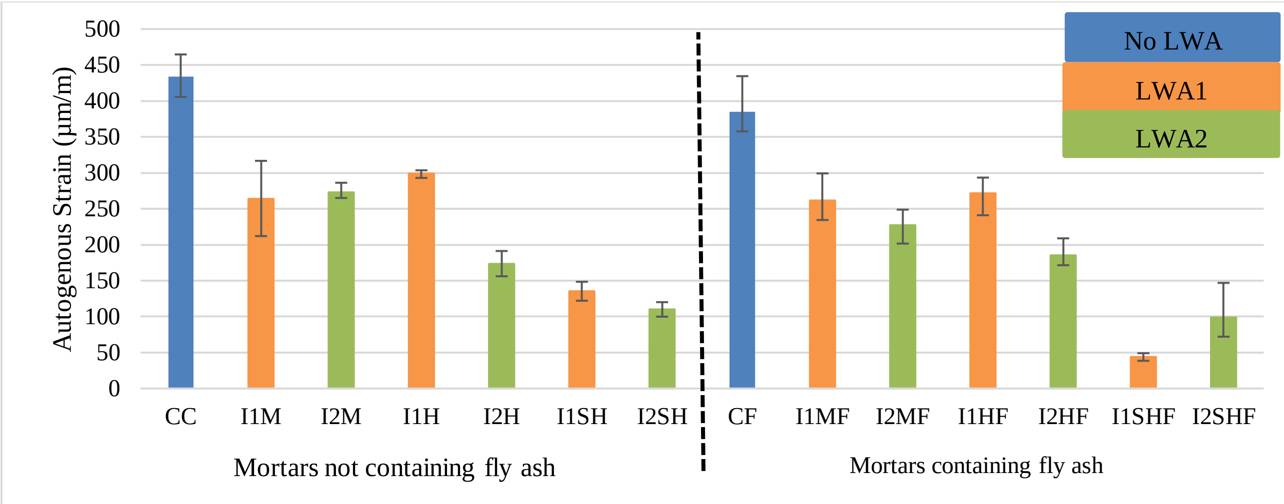

Figure 3.16: Autogenous shrinkage test results at 28 days, mortars with Type I/II cement

3.3.2 Latex Modified Bridge Deck Mixtures

A summary of test results for the control and internally cured LMC mixtures is provided in Table 3.10. As discussed in Section 3.2.2, the mixtures currently approved by NCDOT for LMC very often have a w/cm of 0.40, which resulted in the laboratory (and reportedly, often in the field) in slumps greater than the allowable 3 to 6 inches. The approved mixtures at w/cm = 0.40 provided acceptable 7-day compressive strength, greater than 3000 psi. However, it is evident that a significant reduction in w/cm, and potentially durability performance benefits could be achieved using prewetted LWA as an internal curing agent in LMC mixtures.

Table 3.10: Results of LMC testing

	Age (days)	CLMC	ILMC	CL	ILA	ILB
w/cm	Fresh	0.40	0.40	0.32	0.32	0.18
Slump		10.25	9.75	4.50	11.00	3.50
Compressive Strength (psi)	3	4,050	3,610	1,965	2,845	5,010
	7	4,710	4,500	4,465	3,220	5,490
	28	7,230	5,220	5,305	4,155	5,275
	90	7,610	6,150	6,610	5,205	7,980
MOE (psi)	28	3,651,000	3,801,000	3,215,000	2,344,000	2,962,000
Poisson's Ratio	28	0.22	0.24	0.21	0.22	0.23
CTE (in./in./°F)	28	6.726×10^{-6}	5.963×10^{-6}	---	---	---
Surface Resistivity (kΩ-cm)	3	18.7	12.6	18.7	14.4	22.5
	7	22.0	16.7	24.5	17.5	31.1
	28	---	---	33.8	24.1	42.2
	90	---	---	39.8	28.6	51.5
RCPT (Coulombs)	28	2,170	3,250	---	---	---
	90	1,290	2,740	---	---	---
Freeze-thaw test (DF)	300 cycles	N/A	N/A	108.8	100.2	109.2
Freeze-thaw test (% mass loss)		N/A	N/A	0.33	0.53	0.29
Unrestrained drying shrinkage (% change in length)	14	0.0327	0.0170	---	---	---
	28	0.0353	0.0287	---	---	---
	56	0.0530	0.0497	---	---	---

All mixtures met NCDOT specifications for LMC of 3,000 psi compressive strength at 7 days. 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. 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.

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. 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.

3.3.3 Very High Early Strength Latex-Modified Mixtures

Two very high early strength (VHES) LMC mixtures were batched using CTS Rapid Set Cement, representative of a latex-modified overlay mixture: a control mixture with natural silica sand (RSCL), and an internally cured mixture with 35% replacement of natural sand with prewetted LWA (RSIL). A summary of test results is provided in Table 3.11. Construction challenges associated with VHES mixtures are often the result of the very short working time. Although not quantified, the research team agreed that use of prewetted LWA in the internally cured VHES mixture (RSIL) extended the time of workability of the mixture by several minutes over that of the control mixture (RSCL).

Table 3.11: Results of VHES concrete testing

	Age (days)	**RSCL**	**RSIL**
w/cm	fresh	0.35	0.29
Slump (in)	fresh	3.50	3.75
Compressive Strength (psi)	3	5,812	6,196
	7	6213	7018
	28	6749	7397
	90	7993	8082
MOE (psi)	28	3,965,000	3,456,000
Poisson's Ratio	28	N/A	N/A
CTE (in./in./°F)	28	N/A	N/A
Surface Resistivity (kΩ-cm)	3	263.7	119.0
	7	201.2	78.5
	28	127.3	49.4
	90	130.4	76.3
RCPT (Coulombs)	28	120	425
	90	260	310
Freeze-thaw test (DF)	300 cycles	93.8	107.8
Freeze-thaw test (% mass loss)		0.78	0.77
Unrestrained drying shrinkage (% change in length)	14	0.0005	0.0010
	28	0.0020	0.0005
	56	0.0055	0.0005

The internally cured VHES latex modified mixture (RSIL) obtained the highest compressive strength of all mixtures included in this study with the exception of the control bridge deck mixture (CC). As expected, the RSCL and RSIL mixtures achieved the highest compressive strengths and resistivities of all mixtures batched and tested, with the exception of the control mixture (CC). These mixtures also exhibited the lowest chloride permeabilities of all mixtures tested, as well as some of the best performance in the freeze-thaw durability test and lowest shrinkage in the unrestrained drying shrinkage test.

RCPT results indicate that using VHES cement produces a mixture highly resistant to chloride ingress at both earlier (28 days) and later ages (90 days). Although specimens from both mixtures allowed for a negligible amount of charge passed during the test, it is evident that the internal curing mixture's chloride resistance was slightly less than that of the control mixture. However, this may be a result of the charge-carrying capacity of the water-filled voids contained in the LWA in the internally cured specimens. Also of note, the surface resistivity readings on LMC mixtures tended to fluctuate. It can be seen in Table 3.11 that the resistivity of the very high early strength mixtures decreased over time, which is inconsistent with the resistivity behavior of conventional concrete. It is assumed that the latex admixture in the binder influences this behavior.

To determine if internal curing using prewetted LWA can provide advantages to VHES mixtures, autogenous shrinkage tests were performed on specimens of IC mortars batched with LWA 1 and LWA 2 along with VHES and latex modifying admixture. To complete these tests, the ASTM C1698 test procedure was utilized. However, due to the rapid set and very high early strength gain, an earlier starting time (t_0) for measurements was selected as 2 hours. Results for 28-day autogenous shrinkage measurements shown in Figure 3.14 indicate that the IC VHES mortars using both LWA 1 and LWA 2 exhibited a reduced amount of autogenous shrinkage compared to the control VHES mortar (VHECL). Increasing the proportion of prewetted LWA resulted in a reduction in autogenous shrinkage, although it is noted that there is some overlap in the range bars.

It is likely that a significant amount of shrinkage of the VHES mortars may have occurred prior to t = 2 hours, and additional differences in the autogenous shrinkage rates could be observed if an earlier initial measurement time was utilized. However, because the autogenous specimens are long, narrow cylinders in flexible tubing, the team was concerned that selecting a t_0 too early (before sufficient strength was gained) could result in damaging the specimens during measurement. Figure 3.17 shows test results for these same VHES mortars, with the autogenous shrinkage computed in a manner that complies with ASTM C1698 (t_0 = 1 day). Although not directly comparable with the Figures 3.13 and 3.14 results for Type I/II cements, it is evident from Figures 3.17 and 3.18 that the addition of prewetted LWA to VHES mortars was effective in reducing autogenous shrinkage at 28 days.

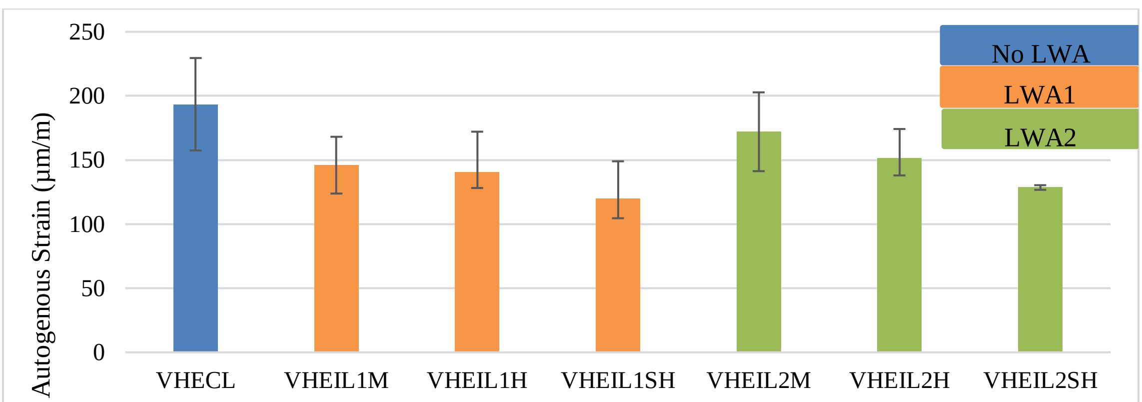

Figure 3.17: Autogenous shrinkage test results at 28 days for VEHS mortars using t_0 = 2 hours

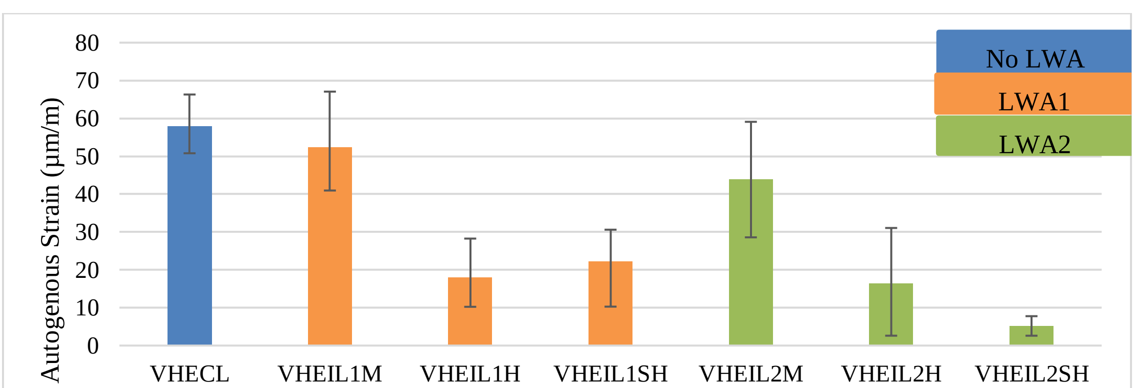

Figure 3.18: Autogenous shrinkage test results at 28 days for VEHS mortars using t_0 = 1 day

3.3.4 Pavement Mixtures

Test results for the internal curing pavement mixture (referred to as IP) are compared to test results of P.B.N.N. from NCDOT RP 2015-03 in Table 3.12. Results presented in this table are the averages of the multiple replicates of each specimen (same number of test replicates as described in Sections 3.3.1.2 and 3.3.1.3, with supporting data provided in Appendix B, Tables B.3 to B.12 and Figures B.36 to B.38). 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, 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.

Table 3.12 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
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}
Surface Resistivity (kΩ-cm)	3	8.0	5.3
	7	8.7	6.4
	28	10.7	8.2
	90	11.0	9.1
RCPT (Coulombs)	28	4,390	6,190
	90	3,230	2,800
Freeze-thaw test (DF)	300 cycles	81.83	84.39
Freeze-thaw test (% mass change)		-1.09	-0.66
Unrestrained drying shrinkage (% change in length)	14	0.0109	0.0315
	28	0.0182	0.0388
	56	0.0245	0.0500
Restrained shrinkage test –time to cracking (days)	-	3 specimens – 20.28, 20.60, 15.85 Average - 19.6 days	2 specimens – no crack at 28 days 1 specimen – cracked at 17.5 days
Restrained shrinkage test – average rate of strain (in/in/day)	-	4.388×10^{-6}	7.02×10^{-6}
Restrained shrinkage test – average stress rate (psi/day)	-	45.94	7.55

The internally cured pavement mixture developed compressive strength more rapidly the control mixture, although the 90-day compressive strength was slightly lower. The MOE was also slightly lower than the control mixture, likely due to the addition of LWA. Lower MOE values have been shown to be associated with reduced cracking potential (Rao and Darter 2013). Trends in the durability performance test results were similar to those observed 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 the control mixture 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. The freeze-thaw durability of the internally cured pavement mixture was slightly improved.

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. Of note, the internally cured pavement mixture did outperform the conventional mixture in the restrained shrinkage test, with two specimens not cracking during the 28 day test, and a notably lower stress rate. 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 help explain the findings presented above.

3.4 Summary of Laboratory Findings

Based upon the results presented in previous sections of this report, laboratory findings of this project are:

Fresh Properties

- For bridge deck mixtures, internally cured concrete mixtures using locally available LWA exhibited fresh properties similar to control concrete. These mixtures could be readily adjusted with admixtures to meet NCDOT specifications.

- 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, λ (ACI 2016).

- Use of prewetted LWA for internally cured pavement, LMC, and VHES mixtures resulted in significantly increased slump. The additional workability offered by the prewetted LWA may allow for further reduction of water content, lower w/cm ratios, and potentially superior durability performance for these mixtures.

- Use of prewetted LWA in the VHES mixtures appeared to increase the time that the mixtures were workable. Internally cured VHES mixtures could offer construction advantages in the field, in addition to the improved durability performance, although field trials are recommended.

Mechanical and Thermal Properties

- Internally cured concrete mixtures using locally available LWA exhibited:
  - Adequate compressive strength to meet NCDOT specifications.
  - Lower MOE than control mixtures, which could further aid in preventing early age cracking.

- All mixtures met NCDOT specifications for compressive strength for the appropriate concrete use (bridge deck or pavement).

- Fly ash mixtures, both internally cured and non-internally cured, exhibited lower early age strengths and significant later age strength gain, as expected for these types of mixtures.

- Mixtures containing LWA tended to have a reduced MOE, roughly 20% to 25% lower than the MOE of the control mixture. In the case of the pavement mixtures, the MOE of the internally cured mixture was 34% lower than the control mixture.

- 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.

- Poisson's ratios for all mixtures tended to range between 0.20 and 0.24, with one mixture (I1M) having a higher value of 0.26.

- For bridge deck mixtures, addition of LWA fines for internal curing tended to reduce the CTE by about 0.5 to 1×10^{-6} in./(in.×°F) from the control concrete result (mixture CC). Reductions ranged from 3% (fly ash mixtures) to 11% (LMC), indicating potential for improved thermal performance.

- Many LMC mixtures approved by NCDOT for LMC have a w/cm of 0.40, which resulted in the laboratory (and reportedly, often in the field) in slumps greater than the allowable 3 to 6 inches. The approved mixtures at w/cm = 0.40 provided acceptable 7-day compressive strength, greater than 3000 psi. However, it is evident that a significant reduction in w/cm, and related durability performance benefits, could be achieved using both conventional aggregates and using prewetted LWA as an internal curing agent in LMC mixtures.

- As expected, the VHES LMC mixtures achieved the highest compressive strengths and resistivities of all mixtures batched and tested, with the exception of the control mixture (CC). The internally cured VHES latex modified mixture (RSIL) obtained a higher compressive strength than the control mixture RSCL.

- The CTE of the internally cured pavement mixture IP (5.008×10^{-6} in./(in.×°F) is also lower than the control pavement mixture (P.B.N.N.) from RP 2015-03 (5.309×10^{-6} in./(in.×°F), indicating that the internally cured mixture could potentially provide improved performance over the conventional mixture.

Durability Performance

- Evaluation of permeability using surface resistivity and bulk conductivity indicate that performance of internally cured mixtures is similar to that of conventional mixtures.

- The use of fly ash was more influential in reducing the permeability of mixtures than addition of prewetted LWA for internal curing. Significant reductions in permeability at later ages were evident in all mixtures.

- Surface resistivity and bulk conductivity test results show a correlation to RCPT test results, indicating the suitability of these electrical methods for evaluation of the durability of ICC mixtures. However, measurements of ICC mixtures made with each of these tests appear to be somewhat influenced by the water contained in the LWA. Separate prediction models may be required for these types of mixtures.

- Surface resistivity and bulk conductivity tests (both electrical tests) could not be reliably performed at the later ages on LMC mixtures. The readings exhibited significant drift, and would not stabilize long enough for measurements to be obtained with confidence. The reason for this issue may be related to the chemical bonding of the latex in the paste, but more research may be needed to confirm this finding.

- The use of LWA1 for internal curing (with an appropriate entrained air void system) seemed to provide improved freeze-thaw performance, as evident by the consistently higher durability factor of the I1 mixtures compared to the non-internally cured mixtures. Freeze thaw testing showed that some fly ash mixtures with LWA2 did not perform as well.

- VHES LMC mixtures exhibited the lowest chloride permeabilities of all mixtures tested, as well as some of the best performance in the freeze-thaw durability test

- Surface resistivity readings from the conventional and VHES LMC mixtures exhibited fluctuations, possibly due to the latex admixture. VHES LMC mixtures exhibited a decrease in resistivity over time, which is inconsistent with the resistivity behavior of conventional concrete.

Shrinkage Tests

- Autogenous shrinkage test results clearly demonstrate the impact of use of prewetted LWA for internal curing. The autogenous strain of the non-internally cured mixtures – control (no fly ash) CC, and control with fly ash CF – is higher than the strain exhibited by all other internally cured mixtures.

- 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.

- The replacement rate appears to be significant in reducing autogenous strain for both LWA1 and LWA2.

- Use of prewetted LWA in VHES latex modified mixtures resulted in similar trends in the reduction of autogenous shrinkage, with reductions in autogenous shrinkage increasing with increased replacement rate.

- For bridge deck mixtures, benefits of internal curing were not as readily observed in unrestrained shrinkage. This is consistent with other research (Henkensiefken et al. 2009, Ardeshirilajimi et al. 2016). The time to cracking of conventional mixtures tended to be shorter than internally cured mixtures. 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, an approximately 13% reduction.

- Unrestrained shrinkage rate test results did not clearly show performance improvements for internally cured concrete bridge deck mixtures. These findings are consistent with those by other researchers, who indicate that internal curing effects will be more evident in autogenous shrinkage tests than other drying shrinkage tests (such as unrestrained volume shrinkage per ASTM C157).

- Results for unrestrained volumetric shrinkage (ASTM C157) indicated that at 28 days, all mixtures had length changes less than 0.04% (or roughly 400$\mu\varepsilon$). In work for Virginia DOT, 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 fall beneath these thresholds. Drying shrinkage performance guidance suggested in AASHTO PP 84-17 is 420$\mu\varepsilon$ at 28 days, these test results have also outperformed this threshold.

- VHES LMC mixtures exhibited the lowest shrinkage in the unrestrained drying shrinkage test.

- The internally cured pavement mixture significantly outperformed the conventional pavement mixture in the restrained shrinkage test. In the unrestrained shrinkage test, the conventional pavement mixture outperformed the internally cured mixture, however.

4. FIELD IMPLEMENTATION OF INTERNALLY CURED CONCRETE

4.1 Introduction

To assist in development of specification provisions for internally cured concrete, a pilot project was included in this research study. The goals of this field implementation effort, which included construction of a bridge deck in Durham, NC, were to:

- develop a provisional specification (project special provisions, or PSP) to guide construction of the pilot project bridge deck, which included both ICC and conventional concrete
- assist NCDOT, contractor, and concrete supplier personnel with development and approval of the ICC and conventional concrete deck mixtures
- develop, install, and monitor an in-situ data acquisition system to support collection of humidity and strain data from both the ICC and conventional deck sections
- monitor and document the batching and placement process of ICC
- perform field and laboratory testing of concrete used to construct the pilot project
- interview stakeholders regarding their experience with ICC and provisional specification
- evaluate the performance of the ICC and conventional concrete bridge decks
- revise specification provisions to support use in future projects where use of ICC is desired

4.2 Pilot Project Overview

The pilot project selected for field implementation of ICC was Project U-3308 in Durham County, a bridge on NC 55 Alston Avenue crossing over NC 147 Durham Expressway. The site (prior to construction) is shown in Figure 4.1. The project consisted of two phases of work associated with the bridge deck: Stage 1 included the northbound lanes, and Stage 2 included the southbound lanes. Figure 4.2 provides an overview of the staging of the project, along with concrete pour IDs. A summary of the types of concrete used for each pour, along with the dates of construction, are provided in Table 4.1. Both Stage 1 and Stage 2 included an ICC and a conventional (control) concrete section of the concrete bridge deck. Stage 1 of the bridge deck was poured in July and August of 2017, and Stage 2 was completed in December 2018 and January 2019. Due to constraints associated with the long timeframe between these two stages of work, most of the research team's effort (instrumentation and field testing) was focused on the Stage 1 portion (northbound lanes) of the bridge deck. Stakeholder interviews were conducted prior to Stage 1 construction and after Stage 2 construction. Limited field and laboratory data was also collected for the southbound lanes included in Stage 2.

Figure 4.1: Original bridge on NC 55 Alston Avenue crossing over NC 147 Durham Freeway

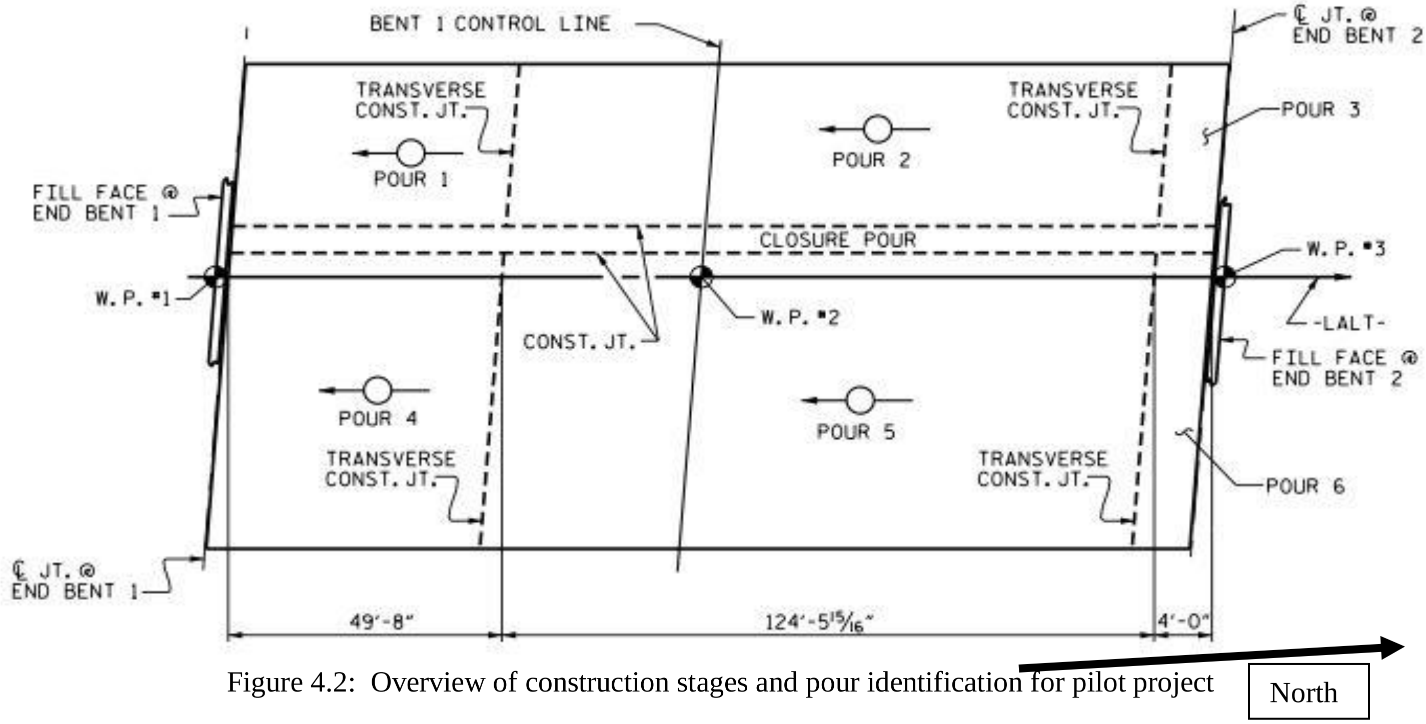

Figure 4.2: Overview of construction stages and pour identification for pilot project

Table 4.1: Type of concrete used for each pilot project pour and date of construction

Stage	Pour ID	Type of Concrete	Date of Construction
1 (Northbound lanes)	1	Conventional	July 26, 2017
	2	Internally cured	August 6, 2017
2 (Southbound lanes)	4	Internally cured	December 19, 2018
	5	Conventional	January 7, 2019

Instrumentation efforts on the northbound lanes included humidity and strain sensors. In addition to construction of the pilot project deck, two instrumented companion slabs were constructed adjacent to the pilot project bridge deck. These slabs also contained strain and humidity sensors, which were returned to UNC Charlotte's laboratory in mid-August 2017, and stored/monitored in laboratory conditions. The hope was that these companion slabs would provide insight into the material performance when not influenced by other structural components, outside weather conditions, construction loads, or traffic. These tests were discontinued during late March 2018. The companion slabs are shown in Figure 4.3, below. In this photo, the Stage 1 control concrete companion slab has been poured (rear) and the Stage 1 ICC companion slab formwork and instrumentation is being prepared for concrete placement.

Figure 4.3: Preparing second companion slab form, sensors, and data acquisition system for placement of Stage 1 ICC mixture.

4.3 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 C. A summary of the contents of the PSP are presented in subsequent sections.

4.3.1 Materials, Mixture Proportioning and Batching

The concrete mixtures used for the pilot project bridge deck were Class AA concrete meeting NCDOT Standard Specifications Section 1000-4, and cementitious materials utilized included fly ash at substitution rates allowable per NCDOT Specifications (NCDOT 2012). The internally cured concrete mixture contained the same materials and mixture proportions utilized for the Class AA concrete mixture, but were modified by substituting 30% pre-wetted lightweight fine aggregate (meeting AASHTO M195) by volume for the standard fine aggregate. The LWA used to construct the pilot project was the same as LWA1 from the laboratory portion of this study. Both concrete mixtures, conventional and internally cured, included air entraining and water-reducing admixtures as required to meet NCDOT specifications for Class AA concrete and project specifications.

The proposed concrete mix designs, supporting test data, were 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 pre-wetted LWA. The moisture content of the pre-wetted 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 pre-wetted LWA should be determined immediately prior to batching and adjust batch weights accordingly. The mix water should not be adjusted 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 if the pre-wetted LWA is saturated prior to use.

4.3.2 Stockpile Management for Prewetted Lightweight Aggregate

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 (in this case, 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 pre-wetted 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.

4.3.3 Concrete Placement

The PSP states that placement of the conventional normal weight 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.

4.4 Summary of Pre- and Post-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. A summary 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?

4.4.1 Concrete Supplier Interviews

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. Prior to the project, he indicated he believed ICC production should be fairly straightforward, and confirmed this after the Stage 1 ICC placement. He also commented that it was appreciated that the standard Type B pressure meter could be utilized with ICC, rather than the volumetric method to determine air content used on conventional lightweight mixtures. Prior to the placement, the 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 worked 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 regarding the amount of work be performed to manage a small amount of material for the pilot project (roughly a truckload of LWA). Overall, 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.

A follow-up interview occurred post-construction with the concrete supplier representative. Due to personnel changes, this individual was a new member of the supplier's team. He indicated that other than the effort required to stockpile and manage an additional material (prewetted LWA fine aggregate), production of the material was straightforward. This technical services specialist echoed his colleague's concerns regarding production of ICC in locations where appropriate technical personnel are not readily available to support development and quality control of the mixture.

If ICC becomes more commonly specified for use in certain markets, the supplier's representative felt that certain plant operations could be modified to accommodate a greater volume of the prewetted LWA fine aggregate.

4.4.2 Contractor Interviews

The contractor representative 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.

Post-construction, contractor personnel indicated that the ICC mixture for both Stage 1 and Stage 2 construction were not problematic to place or finish. Contractor personnel did not need additional equipment, labor, or time in order to construct the ICC portions of the deck.

4.4.3 NCDOT Interviews

The NCDOT personnel interviewed were the Division 5 Resident Engineer and Senior Assistant Resident Engineer. Prior to construction, NCDOT personnel stated they had no concerns with the PSP prior to the project. During post-construction interviews NCDOT personnel indicated they felt that the PSP was well written, and they did not field questions or concerns regarding the PSP from the contractor. The inclusion of background information on ICC was appreciated, since it provided insight into the material for stakeholders not familiar with ICC. The Resident Engineer felt that the supplier and contractor were able to comply with the PSP, and were able to successfully construct the bridge deck. The provisions of the PSP appear to be readily transferrable to a more general specification for ICC, and no suggestions for modification were apparent after construction of the pilot project.

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.

4.5 Bridge Deck Instrumentation

The field testing and monitoring exercise included installing embedded instruments in the bridge deck, collecting materials samples during concrete placement, and monitoring the embedded instruments after the concrete cured. Sixteen sensors were installed in the bridge deck prior to concrete placement and were suspended between the top and bottom rebar mats with epoxy coated rebar ties. One inch diameter, schedule 40 PVC conduits were used to route and protect the sensor cabling as it passed through the deck and into the data acquisition enclosure, which was placed atop the wing wall. The instruments used are described in Table 4.3. Placement of the sensors is shown in Figures 4.3 and 4.4. Additional photographs of the installation and concrete pours are available in Appendix D.

Table 4.2: Sensors and data acquisition equipment used in the pilot project

Equipment	Instrument Model	Manufacturer	Modification
relative humidity sensor	HTM2500LF	TE Connectivity	Wrapped in GORTEX fabric for waterproofing
concrete embedment strain gage	4200	Geokon	None
Data Logger	CR1000	Campbell Scientific	None
Vibrating Wire Analyzer	AVW200	Campbell Scientific	
Multiplexer	A16	Campbell Scientific	

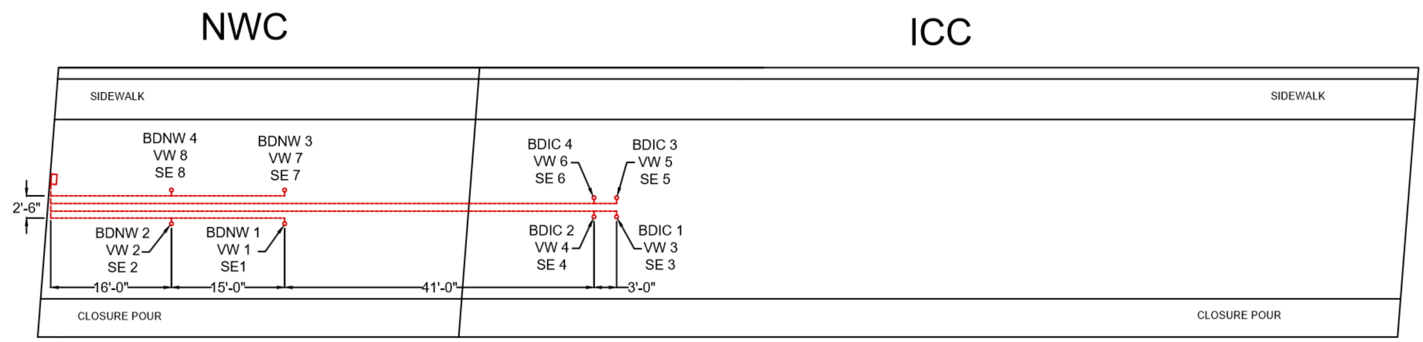

Figure 4.4: Plan view of span A with sensor locations, and conduit locations

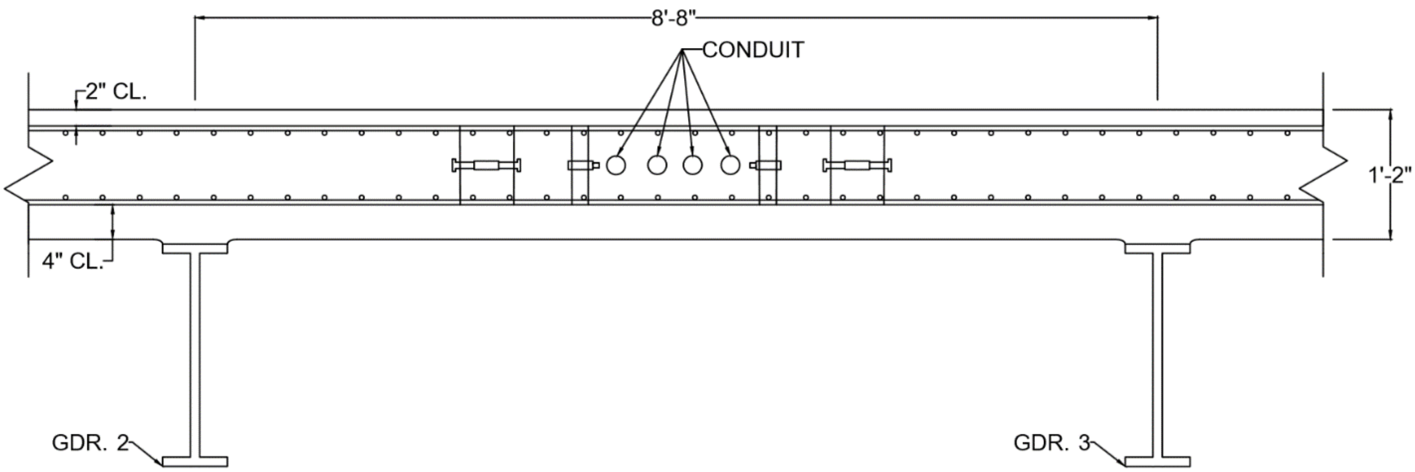

Figure 4.5: Typical intermediate cross section of span A with conduit locations shown

4.6 Field Monitoring Program

Concrete mixtures were developed by the batch plant operator and approved by the NCDOT. The ICC and non-ICC mixture contained the components shown in Table 4.4.

Table 4.3: Concrete mixtures used in pilot project

Material	Quantity (pcy)	
	Control Mixture	**Internally Cured Mixture**
Cement (type I/II)	620	620
Pozzolan (type F fly ash)	186	186
Fine Aggregate	846	592
Prewetted Lightweight Fine Aggregate	0	169
Coarse Aggregate	1850	1850
Water	287	204.3
Air entrainer, Superplasticizer and retarder	Batched as needed	Batched as needed

All of the strain sensors installed in both the ICC and NWC sections of the deck survived placement and provided good quality data. Of the eight humidity sensors installed in the deck, two sensors in the ICC area did not survive the construction process. These were most likely damaged by construction equipment during concrete placement. Data was recorded from the functioning sensors hourly for a period of 31 weeks. The data records presented in this report begin at 24 hours after concrete placement for each of the conditions described. Prior to this post-pour period, substantial noise is present in the data due to ongoing construction activities and early-stage hardening and curing phenomena. Photographs of the instrumentation and construction process are provided in Appendix D.

During construction of the pilot project deck, two instrumented companion slabs were also constructed adjacent to the pilot project bridge deck. These slabs contained the same strain and humidity measuring instrumentation (four of each type of sensor in each slab – sixteen total sensors). The companion slabs were returned to UNC Charlotte's laboratory in mid-August 2017, and were being stored in ambient laboratory conditions. These companion slabs were intended to provide insight into the material performance without the influences of other structural components, weather conditions, construction loads and traffic.

Because the construction of the Stage 1 normal weight and ICC concrete sections occurred at different times, separated by several weeks during the summer, the research team had some concern that the different weather conditions would influence the results. However, as is shown in Figure 4.5, the internal temperature of the two deck sections was not substantially different.

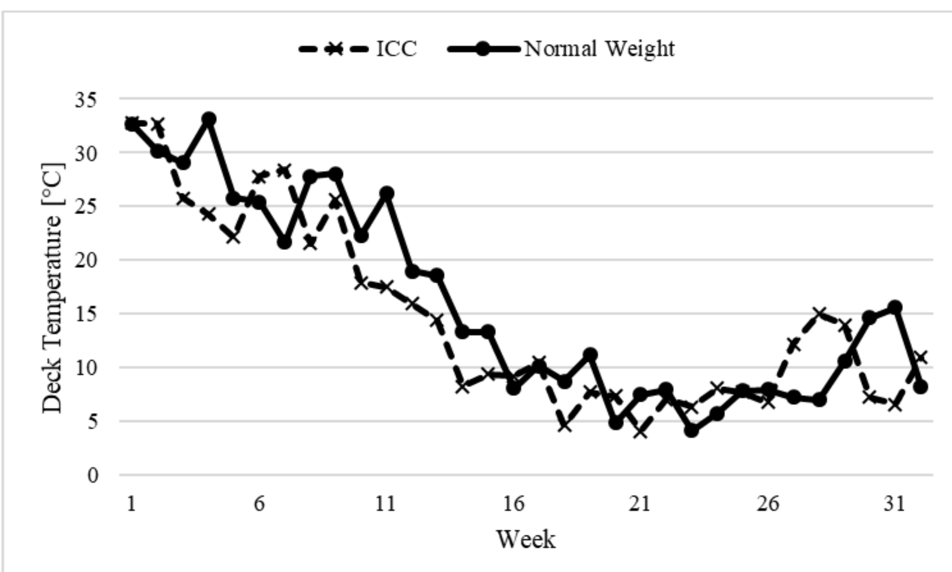

Figure 4.6: Internal deck temperatures

Concrete Internal Humidity

The influence of the internal curing materials on the availability of internal moisture for curing and hydration reactions was measured by relative humidity sensors. Four sensors were installed in each section of the deck, however two of the sensors in the internally cured section were damaged during construction. The two surviving instruments were in close agreement. Figure 4.6 shows the measured internal relative humidity measured over 31 weeks in the two sections. Counter to anticipated results, the internal relative humidity of the internally cured area decreased more quickly than the humidity in the normal-weight area. These sections were nearly equal in their internal conditions during 16-21 weeks of age. It is unclear why this contrary result was encountered, however, it could be that the internal curing materials were not sufficiently pre-wetted and that they were a moisture sink rather than a moisture source. It could also be that the external curing conditions during the first several weeks of age were different. However, as is shown in Figure 4.7, there is no appreciable difference in the exposure to rain between the two materials.

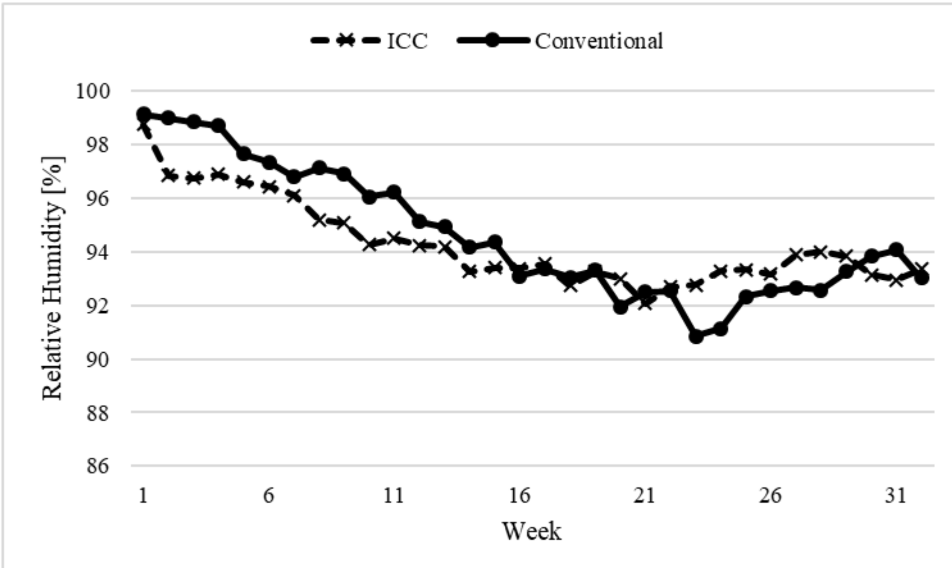

Figure 4.7: Internal relative humidity of ICC and conventional deck areas

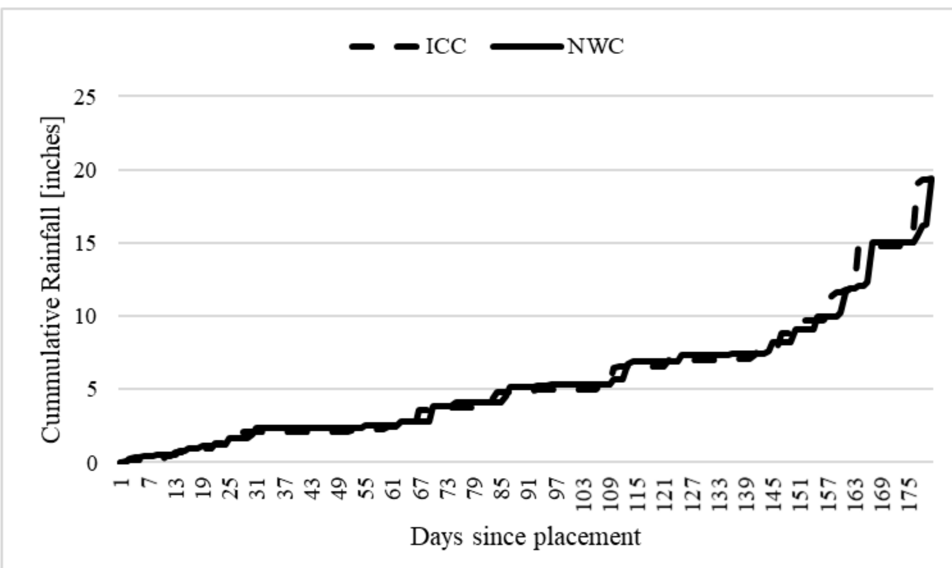

Figure 4.8: Cumulative rain during first 180 days of curing for each deck area

Strain Measurements

Vibrating wire strain gages were used to measure transverse and longitudinal strains during the monitoring period. Measurements were commenced 24 hours after the concrete was placed. Because the bridge deck was part of an active construction site, there were many factors that interfered with the quality of the collected data. These included the presence of equipment on the bridge deck that created elastic strains, diurnal temperature changes that created thermal strains, and substantial restraint against shrinkage provided by the dense rebar mat and the structural response of the deck and girders to the continual addition of dead load as the bridge construction continued. In order to reduce the impact of these influences, the strains reported are weekly averages. Averaging over a week of time reduced the influence of transient loads and daily activity.

Figure 4.9 displays the measured longitudinal strains. The concrete in the internally cured region indicates greater longitudinal strains related to shrinkage. Most strain occurred during the first three weeks of curing, however the change after this time was minimal. The area without internal curing materials achieved less shrinkage-related strain during the first 10 weeks of curing and then increased more during later weeks. The relationship between internal humidity and shrinkage is depicted in Figure 4.10. There is a stronger relationship between increased shrinkage strain and lower relative humidity in the normal weight concrete. This means that concrete shrank more as the humidity dropped in the concrete that did not contain internal curing materials. However, this effect was quite small.

Figure 4.11 displays the transverse shrinkage behavior of concrete in the two areas as registered by tensile strains in this direction. In the case of the transverse behavior, the concrete in the internally cured area of the deck registered less strain than the material in the normal weight concrete area. This may imply that the addition of the ICC materials affected the Poisson's ratio of the concrete, or it could be related to peculiarities in the field testing such as temperature differences. The difference measured strains in the two areas is very small.

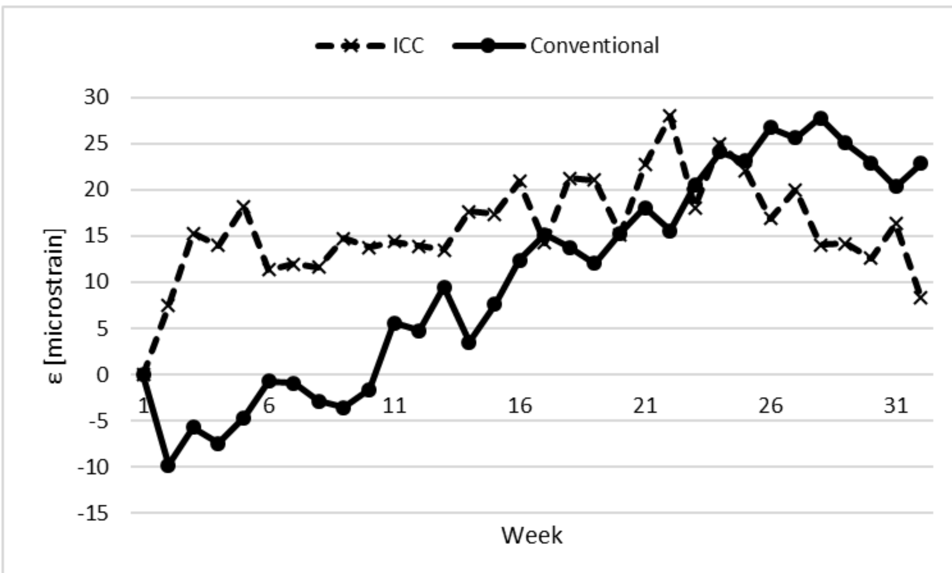

Figure 4.9: Longitudinal strains measured in the ICC and NWC areas of the deck

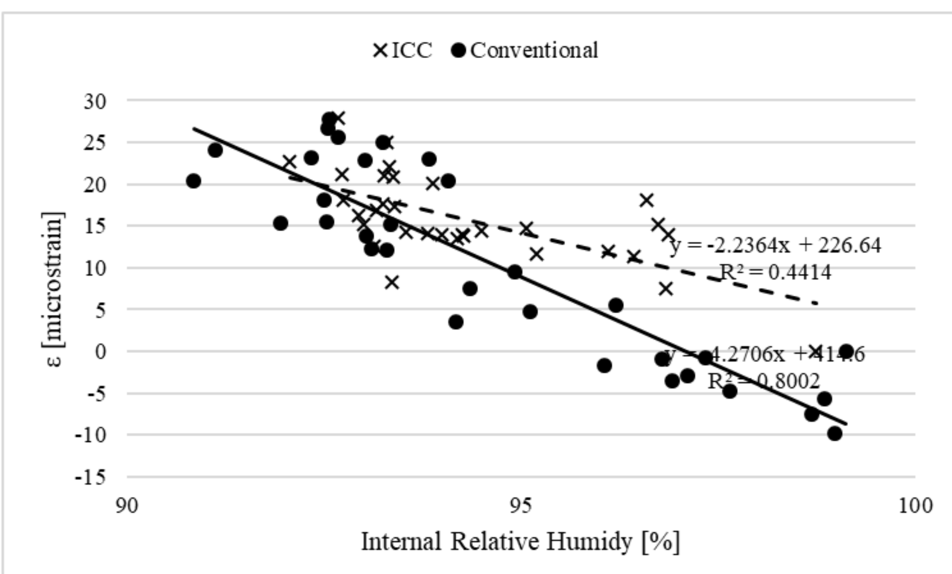

Figure 4.10: Relationship between strain and relative humidity in ICC and NWC areas of the deck

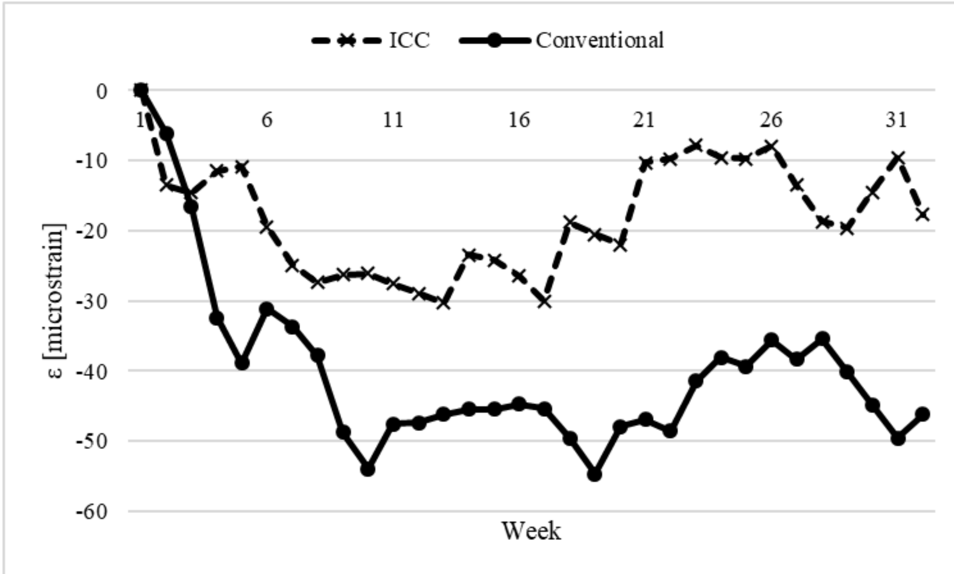

Figure 4.11: Transverse strains measured in the ICC and NWC areas of the deck

Measurement of relative humidity in the laboratory slabs was accomplished by casting and instrumenting models of the deck slab as is shown in Figure 4.12. A wooden form was prepared with two casting compartments. One side was reinforced with mats identical to those installed in the field and the other side was unreinforced. Strain and relative humidity sensing instruments (as listed in Table 4.2) were installed in each slab before concrete was cast. The slabs were prepared with material delivered to the pilot project site. The slabs were allowed to cure for approximately one week at the site and were treated in the same manner as the bridge deck for post-casting protection and watering. After this initial curing period, the slabs were moved back to the laboratory at UNC Charlotte, where they were monitored for a period of 28 weeks. The data recovered from these specimens was somewhat noisy as it appears that voltage spikes in the system caused by nearby equipment resulted in several erroneous readings. In order to mitigate this, the data was averaged by week, and clear outliers were manually removed from the data set. In the graphs that follow, data that was removed is shown as missing data points.

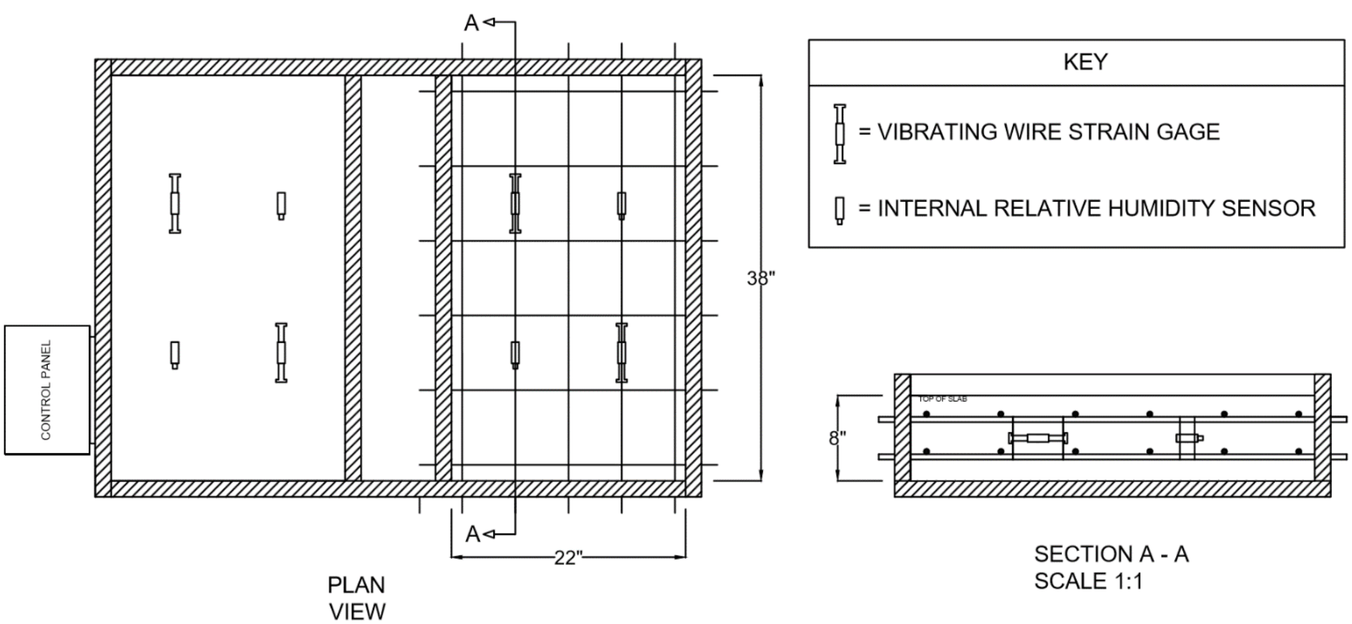

Figure 4.12: Layout of instruments in the bridge deck model slabs

Figure 4.13 relates the relative humidity measured in the slabs with internal curing materials, to the internal relative humidity of slabs cast with conventional concrete. It was assumed that the reinforcing steel had no impact on the moisture available in each specimen, so the averages shown in Figure 4.13 include measurements from all four gages placed within ICC slabs and two gages placed in the conventional concrete areas (two gages in the conventional area failed shortly after concrete placement and provided erroneous readings for the duration of the monitoring period). Similar to the result measured in the field deck, internal relative humidity decreased slightly more rapidly in the concrete that contained internal curing materials.

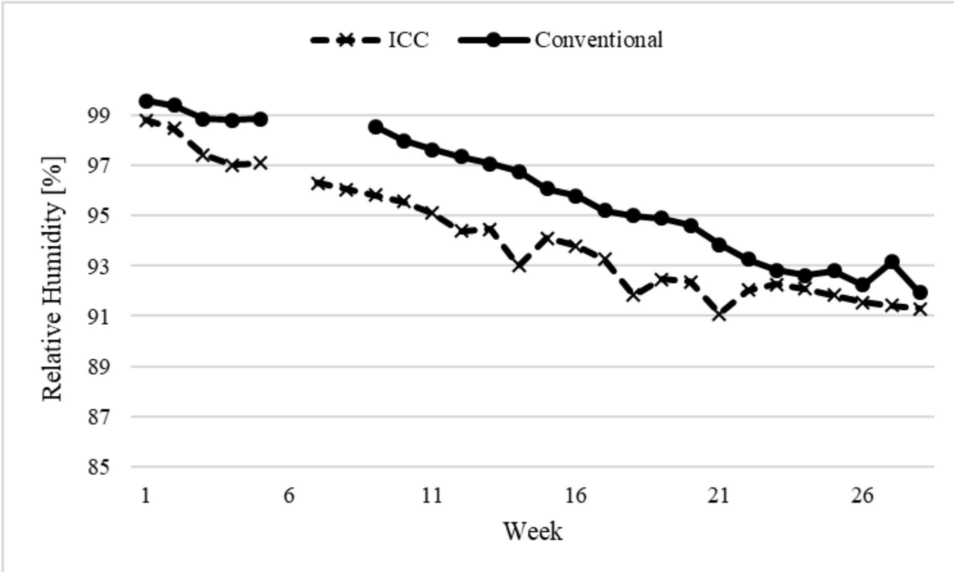

Figure 4.13: Internal relative humidity measured for ICC and conventional concrete slabs

Figure 4.14 relates the longitudinal strains measured in the reinforced slabs containing conventional and ICC concretes. The internally cured concrete registered maximum shrinkage strains of less than 150 με, whereas the conventional concrete exhibited maximum shrinkage strains of around 200 με. Although the percent difference between these two values is large, the actual difference is fairly small. The unreinforced slabs showed very similar trends. In Figure 4.15, it is shown that the internally cured concrete reached around 150 με and the conventional concrete areas reached around 200 με. The magnitudes of the strains measured in the lab setting were substantially larger than those measured in the field. Several factors most likely contributed to this difference, including the low ambient humidity of the lab setting and absence of any rain events, as well as the absence of considerable restraint provided by the steel girders that was present in the field condition but not the lab condition.

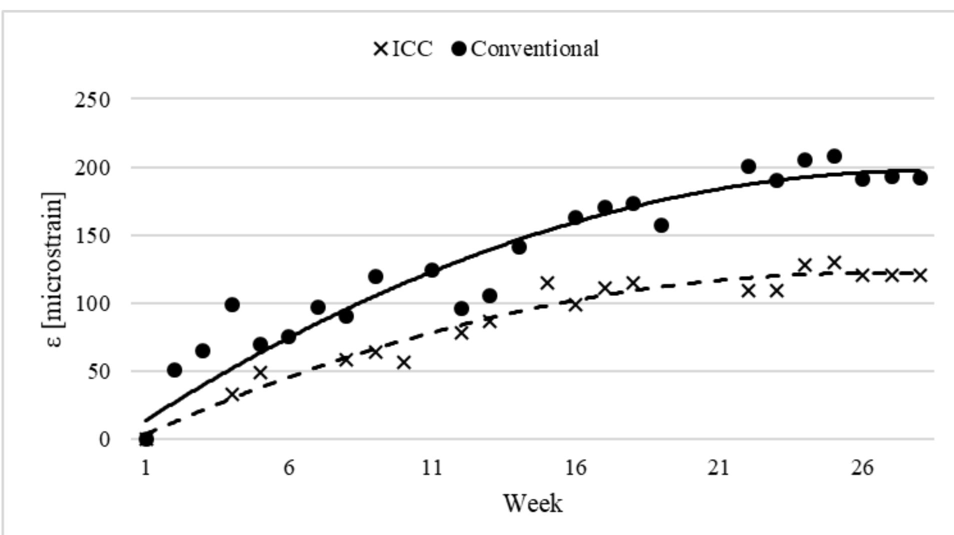

Figure 4.14: Longitudinal Strain in measured for reinforced ICC and conventional concrete model slabs

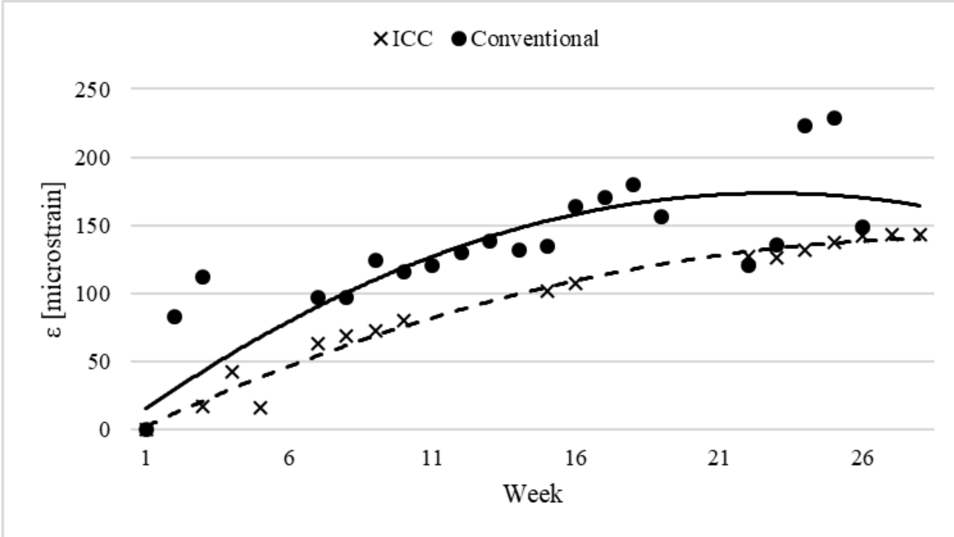

Figure 4.15: Longitudinal Strain in measured for unreinforced ICC and conventional concrete model slabs

4.7 Concrete Tests – Field and Laboratory Results

Prior to the Stage 1 ICC bridge deck pour, UNC Charlotte personnel visited the ready-mix plant to observe preparation and pre-wetting of the lightweight fine aggregate. UNC Charlotte personnel also provided assistance with moisture content testing via centrifuge method prior to batching of the ICC for the pilot project deck. Fresh concrete tests, including air content and SAM, were performed on samples of conventional and ICC concrete obtained from multiple trucks during each pour.

UNC Charlotte personnel also cast cylinders and freeze-thaw beams were cast from both the control and ICC mixtures used to construct the pilot project bridge deck, and returned them to the laboratory for testing. Tests performed on field produced concrete included fresh properties, mechanical properties, CTE, and durability performance tests. A summary of these test results from Stage 1 construction is provided in Table 4.2. Due to placement of the construction joint on the plans at a location offset from midspan, the Stage 1 ICC was placed over approximately 2/3 the length of the bridge. Two sets of test specimens were cast, from early (designated as ICBD1) and late (designated as ICBD2) in the placement Concrete comprising the conventional concrete bridge deck section, approximately 1/3 of the deck, was sampled once (designated as CCBD). Fresh concrete tests from the Stage 1 placements had slumps ranging from 3.5 inches to 4.5 inches, and air contents ranging from 4.5% to 5.5%. Fresh concrete tests from Stage 2 placements had slumps ranging from 4.0 to 4.5 inches, and air contents ranging from 4.2% to 6.2%.

All mixtures gained adequate strength to meet NCDOT's 28-day compressive strength requirements. Surface resistivity values at 90 days exceed recommended values utilized by other agencies to provide adequate resistance to chlorides (such as Louisiana DOT, which often specifies a minimum resistivity measurement of 27 kΩ-cm). The internally cured mixtures have a slightly lower CTE, which may result in improved performance due to reduced thermal stresses.

As could reasonably be expected, test results from specimens cast from concrete sampled early (ICBD1) and late (ICBD2) in the internally cured concrete pour showed some variation in mechanical properties. This may be due to changes in water content as the stockpile was utilized or due to changes in air content. Freeze-thaw test specimens cast from both the control portion and ICC portion of the bridge deck performed adequately based upon the 80% DF performance criteria utilized by many state DOTs. However, it is noted that specimens cast from both the early and later pour sections of the ICC deck exhibited outstanding performance in this test, exhibiting minimal damage during the 300 freeze-thaw cycles.

Placement of the ICC and control sections for the northbound lanes (Stage 2) occurred approximately 18 months after the Stage 1 deck construction. Of note, weather conditions for Stage 2 placement were very different from those during Stage 1, since the placements occurred in December 2018 (ICC) and January 2019 (conventional concrete). All mixtures gained adequate strength to meet NCDOT's 28-day compressive strength requirements. Due to the limited number of test specimen cast, surface resistivity and bulk conductivity tests (non-destructive) were performed. Results are summarized below.

Table 4.4: Results of laboratory testing of test specimens cast from Stage 1 of the pilot project

Mix ID	Compressive Strength (psi)				Surface Resistivity (kΩ-cm)		MOE (psi)	CTE (in./in./^{0}F)	Freeze-thaw test		
	3 day	7 day	28 day	90 day	28 day	90 day			Ave. Post Test DF	% Mass change	Cycles completed
CCBD – stage 1	3680	4510	5090	7110	12.0	34.2	3,567,000	5.371×10^{-6}	80.5	-0.12	300
ICBD1 – stage 1, beginning of pour	3045	3990	4675	7245	10.6	29.1	3,277,000	5.085×10^{-6}	104.7	-0.5	300
ICBD2 – stage 1, end of pour	4190	5560	6365	8045	13.3	33.8	3,715,000	5.081×10^{-6}	98.5	0.02	300

Table 4.5: Surface resistivity and bulk conductivity tests from test specimens cast from Stage 2 of the pilot project

Mix ID	Surface Resistivity (kΩ-cm)		Bulk conductivity (mS/m)		RCPT (coulombs)	
	28 day	90 day	28 day	90 day	28 day	90 day
CCBD – stage 2, beginning of pour	7.1	18.3	35.4	75.2	5070	1566
CCBD – stage 2, end of pour	7.0	19.6	36.2	64.3	5608	1579
ICBD1 – stage 2, beginning of pour	6.3	25.9	32.3	82.7	---	2681
ICBD2 – stage 2, end of pour	7.0	26.4	48.3	89.8	---	2380

4.8 Visual Surveys

Visual inspections of the pilot project bridge deck (Stage 1 only) were made on August 31, 2017 and April 13, 2018. On August 31, 2017 has revealed one crack that is observable with the naked eye while bending at the waist. The location of this crack is on the internally cured portion of the bridge deck, with the crack located approximately 5 feet north of the construction joint separating the first pour from the second pour. The crack is approximately 10 feet long and is oriented transverse to the bridge span. The crack coincides with the bridge pier in the middle of the bridge, and may have resulted due to design-related causes.

On April 13, 2018, a series of smaller cracks was observed along both the internally cured and conventional concrete portions of Stage 1 of the bridge. Seven new cracks were evident at this time: four cracks in the ICC deck and three cracks in the conventional concrete deck. Crack widths ranged from 0.005 to 0.010 inches. All of the cracks present in the ICC section of the deck stemmed from the west side parapet wall at locations (roughly) where expansion joints were planned to be installed. Mr. Aaron Earwood of NCDOT confirmed that the contractor had inadvertently omitted expansion joints in the integrally cast parapet wall, and the cracks in the deck coincided with shrinkage cracks in the parapet wall.

The three cracks found on the NC deck all propagated on the eastern wall which was not impacted by the parapet wall. These three cracks could potentially be the result of drying shrinkage. Photographs of cracks observed on April 13, 2018, along with length and width measurements are provided in Appendix F. This information can be referenced during future observations to evaluate the performance of the internally cured concrete used for this pilot project.

4.9 Recommendations for Specifications

As verified during laboratory and field testing as part of this study and others, ICC concrete has similar workability to conventional concrete. Quality control and quality assurance tests such as slump and air content are also performed in the same manner as conventional concrete. Placing and finishing operations for ICC were also reportedly identical to that of conventional concrete. Based upon test results and interviews with stakeholders, it is apparent that the bulk of the effort to successfully implement ICC in a field project is related to the role of the concrete supplier. If the concrete supplier is able to successfully batch the ICC, the contractor should experience very little difference with ICC construction compared to conventionally cured concrete.

Specification provisions addressing operations at the batch plants should be provided, and stockpile management of the LWA should be addressed. The batch plant must 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 for drainage flow as possible available in an effort to minimize non-uniformity in surface moisture throughout the pile. The stockpile should 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 drainage 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). 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.

On the basis of observations made during the field trials of this research project, as well as the best practices developed by other state departments of transportation, the following provisions are recommended for inclusion in an NCDOT specification for use of internally cured concrete:

- 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.
- 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” (which includes the centrifuge method).
- 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 C231 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.

These specification recommendations have been prepared as a PSP document for bridge deck concrete (Class AA), provided in Appendix G. This PSP differs from the one used for the pilot project (provided in Appendix B) in that it has been prepared in a generic format, and details specific to the pilot project have been removed. In the PSP used for the pilot project, the quantity of LWA specified for use is 30%. Laboratory research demonstrated suitable performance of LWA1 and LWA2 at substitutions up to 35%. Additionally, the field study indicated that for some lower absorption LWA, even higher substitution rates may be required to provide sufficient moisture to support internal curing. Therefore, the PSP recommended for use by NCDOT (provided in Appendix G) states that the amount of lightweight fine aggregate used in the mixture should be determined using mixture proportioning guidance provided in ACI (308-123)R-13 and ASTM C1761/C1761M-17.

4.8 Cost Benefit Analysis

Surface resistivity measurements for the ICC and conventional concrete (Stage 1) construction sections of the pilot project are nearly identical. The strong correlation that exists between surface resistivity and RCPT indicates that the chloride resistance of these two decks is nearly identical (consistent with the literature). Similar findings were made with laboratory mixtures. Since changes in chloride permeability between the ICC and control concrete do not appear to be anticipated (or measurable using current test methods), the team worked to identify and utilize an alternative measure of benefit. As discussed previously, the most notable insight into the potential performance benefits of use of ICC is provided by autogenous shrinkage test results. However, models or performance evaluations linking this type of cracking potential data to quantified benefits over the service life have not been published. Performance evaluation data obtained over coming years from many in-service ICC decks, paired with available laboratory data, may help provide this type of deterioration model in the future.

Future evaluation of the pilot project bridge deck performance could provide the insight required to perform a cost-benefit analysis. For example, a similar pilot project was conducted by the NYS DOT involving twenty decks using to determine the effectiveness of ICC. This project resulted in a successful reduction of cracking by approximately thirty percent as well as a general trend of reduced crack width (Streeter 2012). Cracking increases the frequency of maintenance and repair needed as well as decreasing the service life of the structure. An epoxy injection repair is quoted to cost anywhere from $250 to $300/LF to repair on top of an estimated traffic control cost of $4,000-$8,000 per night for a bridge located in an urban area. Multiple road closures may be required to perform all MR&R activities required for an entire deck. It is suggested that NCDOT monitor the performance of the pilot project bridge deck, quantifying distresses that occur beyond those shown in Appendix D. This information, along with MR&R costs and benefits, could be utilized to quantify the benefits of ICC (Cusson et al. 2010).

Cost data to quantify the potential increase in cost of locally produced ICC also proved challenging to obtain. The research team requested information on the cost of the ICC and conventional concrete used at the site from the concrete supplier. During the bidding phase, however, the supplier was not aware that a portion of the project was to be comprised of ICC, and therefore did not provide separate prices for the two types of concrete to the contractor. The research team requested that, retrospectively, the supplier develop pricing information for both the control concrete and ICC. The supplier was asked to consider the costs of the additional QA support and material costs of the ICC as if they were bidding a similar job in the Durham, NC area as a supplier with experience in ICC. The supplier indicated that they are currently not comfortable providing a price to produce ICC that would reasonably cover a range of anticipated job requirements. The supplier's representative indicated that the cost of ICC would be dependent on (but not limited to): plant location and capabilities, distance to the job, availability and cost of lightweight materials, volume of concrete, and internal quality assurance/control capabilities. The research team asked if the published value of a 10% increase in ICC cost would seem reasonable, and the supplier indicated that number could be reasonable, but seemed this figure could be slightly low for projects with some characteristics.

5. SUMMARY AND CONCLUSIONS

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. Key findings from the laboratory testing are:

- For bridge deck mixtures, internally cured concrete mixtures using locally available LWA exhibited fresh properties similar to control concrete. These mixtures could be readily adjusted with admixtures to meet NCDOT specifications.

- At the replacement rates utilized in the laboratory, 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, λ (ACI 2016).

- Use of prewetted LWA for internally cured pavement, LMC, and VHES mixtures resulted in significantly higher workability, and in the case of VHES LMC mixtures, a perceived longer workable time. Use of prewetted LWA in these mixtures may allow for further reduction of water content, lower w/cm ratios, and potentially superior durability performance for these mixtures.

- Internally cured concrete mixtures using locally available LWA exhibited:
    - Adequate compressive strength to meet NCDOT specifications.
    - Lower MOE than control mixtures, which could further aid in preventing early age cracking.
- All mixtures met NCDOT specifications for compressive strength for the appropriate concrete use (bridge deck or pavement).

- For bridge deck mixtures, addition of LWA fines for internal curing tended to reduce the CTE by about 0.5 to 1×10^{-6} in./(in.×°F) from the control concrete result (mixture CC). Reductions ranged from 3% (fly ash mixtures) to 11% (LMC), indicating potential for improved thermal performance. The CTE of the internally cured pavement mixture IP (5.008×10^{-6} in./(in.×°F) is also lower than the control pavement mixture (P.B.N.N.) from RP 2015-03 (5.309×10^{-6} in./(in.×°F), indicating that the internally cured mixture could potentially provide improved performance over the conventional mixture.

- Surface resistivity, bulk conductivity, and RCPT test results for ICC are similar to that of conventional mixtures. The use of fly ash was more influential in reducing the permeability of mixtures than addition of prewetted LWA for internal curing. Significant reductions in permeability at later ages were evident in all mixtures.

- Surface resistivity and bulk conductivity test results show a correlation to RCPT test results, indicating the suitability of these electrical methods for evaluation of the durability of ICC mixtures. However, measurements of ICC mixtures made with each of these tests appear to be somewhat influenced by the water contained in the LWA. Separate prediction models may be required for these types of mixtures.

- Autogenous shrinkage test results clearly demonstrate the impact of use of prewetted LWA for internal curing. 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. The replacement rate appears to be significant in reducing autogenous strain for both LWA1 and LWA2.

- Benefits of internal curing were not as readily observed in restrained and unrestrained. This is consistent with other research (Henkensiefken et al. 2009, Ardeshirilajimi et al. 2016).

- In the restrained shrinkage tests, the time to cracking of conventional mixtures tended to be shorter than internally cured mixtures. Internally cured concrete mixtures in this study tended to have lower stress rates in restrained drying shrinkage. Unrestrained shrinkage rate test results did not clearly show performance improvements for internally cured concrete bridge deck mixtures.

- Results for unrestrained shrinkage indicated that at 28 days, all mixtures had length changes less than 0.04% (or roughly 400$\mu\varepsilon$), which meet the performance thresholds suggested by AASHTO PP 84-17 and other researchers (Mokarem et al. 2004) to reduce the probability of cracking.

- Preliminary specifications for use of ICC were developed and implemented. A pilot project bridge deck, including both ICC and conventional concrete sections, was constructed.

- Longitudinal and transverse strains were measured the pilot project bridge deck for a period of 31 weeks. Strains measured in the ICC area were comparable to those measured in the conventional concrete area.

- Internal relative humidity was measured in the pilot project bridge deck for a period of 31 weeks as an indication of the presence of the availability of curing water. Internal humidity measured in the ICC area were comparable and slightly lower than those measured in the conventional concrete area.

- The concrete supplier, bridge construction contractor and NCDOT personnel assigned to project oversight reported that the implementation of internally cured concrete did not substantially increase the difficulty of project delivery.

- The concrete supplier estimates that the additional costs of delivering ICC were nearly equal to or slightly greater than 10% more than the costs of delivering conventional concrete.

Based on laboratory and field findings, stakeholder interviews, and review of the literature and other SHA specifications, a revised specification for ICC has been prepared. Of note, the substitution rate of LWA used in the pilot ICC deck trial was 30%. However, the findings of the pilot project indicated that a 30% substitution of LWA may not be enough to provide sufficient moisture to support internal curing when some lower absorption LWA materials available locally in NC are utilized. Greater benefits may be realized by including LWA at a higher substitution rate (>30%), while maintaining the concrete unit weight above 135 pcf. Therefore, the PSP recommended for use by NCDOT (provided in Appendix G) states that the amount of lightweight fine aggregate used in the mixture should be determined using mixture proportioning guidance provided in ACI (308-123)R-13 and ASTM C1761/C1761M-17.

A cost-benefit analysis planned as part of this work could not be successfully completed due to the similarity of performance between ICC and conventional concrete using test methods included in the Life-365 prediction software. Preliminary visual survey results of the ICC and conventional concrete bridge deck indicate similar performance, with cracks evident after several months likely attributable to structural and construction issues. Future visual surveys to quantify performance, along with more reliable cost information on ICC, should facilitate a cost-benefit analysis in the future.

6. VALUE OF RESEARCH FINDINGS and RECOMMENDATIONS

6.1 Value of Research Findings

Findings of this study confirmed that use of ICC should support enhanced durability performance of North Carolina concrete mixtures. Durability performance advantages would be enhanced if used with fly ash in concrete infrastructure. Adding ICC to the types of mixtures utilized by NCDOT should have positive long-term economic impacts. These will be the result of longer lasting concrete components, as well as reduced MR&R costs. One of the performance management goals of the Moving Ahead for Progress in the 21st Century Act (MAP-21) is infrastructure condition. Implementation of ICC will help NCDOT exhibit activity towards this goal.

As part of the pilot project included in this study, a contractor, concrete supplier, and NCDOT division personnel gained experience in construction of an ICC bridge deck. Stakeholder interviews confirmed that use of ICC should not present substantial technical challenges to suppliers, and placing/finishing crews should not be adversely impacted. Specifications developed as part of this work will provide guidance to contractors to ensure the performance both fresh and cured properties of ICC, and should allow NCDOT to immediately implement ICC as an option or a specified material for upcoming projects.

6.2 Recommendations

It is recommended that NCDOT promote use of ICC in future bridge deck projects. Implementation should focus in locations where concrete suppliers have technical staff willing and capable of supporting the development, testing, and QC. Additionally, the supplier should be made aware that ICC is specified for use prior to bidding, so that appropriate cost data can be obtained and utilized. Ensuring that the concrete supplier is aware that the project includes ICC prior to bidding should also assist in ensuring that the logistical challenges associated with use of an additional material (prewetted LWA) are addressed in an economical fashion. Recommendations for future work include the following:

- Use of LWA1 at substitution rates computed by the ACI (2013) approach are significantly higher than the 30 to 35% utilized by many SHAs. However, the benefits of reduced autogenous shrinkage were observed in laboratory mortar mixtures when the replacement rate of 56%, which was empirically determined to supply adequate moisture for curing, was used. It is suggested that further study of ICC at this increased replacement rate be performed. Of note, the unit weight of the concrete should be monitored to ensure that the design provisions for lightweight concrete (below 135 pcf) would need to be utilized.

- Findings from future evaluation of the pilot project should allow completion of a cost-benefit analysis.

- Use of prewetted LWA in VHES LMC mixtures provided workability and (perceived) work-time advantages. Since this type of overlay occasionally experiences construction challenges and early age cracking, use of prewetted LWA in these types of mixtures should be further explored.

7. IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

The findings of this research study support use of ICC in bridge decks, as well as other applications. The specification provisions developed as part of this project could be utilized immediately for construction of ICC in bridge decks. Similar field implementation efforts using ICC in other types of concrete elements such as pavements, conventional LMC, and VHES LMC could provide confidence in the constructability of, and benefits of use of ICC in these types of infrastructure projects. If NCDOT moves forward in specifying ICC for future North Carolina construction, the research team can provide feedback to stakeholders as requested.

Research Product 1	Laboratory test data confirming the benefits ICC in Class AA and pavement mixtures.
Suggested User	Structures Management Unit, Pavement Design & Collection Unit, Materials & Tests Unit
Recommended Use	This information could be used to support decisions to specify ICC in future projects.
Recommended Training	None recommended at this time.

Research Product 2	Specification to support construction of ICC.
Suggested User	Structures Management Unit, Materials & Tests Unit, Construction Unit, and Pavement Design & Collection Unit
Recommended Use	This specification could be utilized immediately to support implementation of ICC in future projects. Bridge deck projects are the focus of this specification, but it could be modified for use in other types of projects such as overlays and/or pavements.
Recommended Training	None recommended at this time.

Research Product 3	Laboratory test data indicating fly ash provides significant durability enhancements in Class AA mixtures
Suggested User	Pavement Management Unit, Structures Management Unit and Materials & Tests Unit
Recommended Use	Information contained in this database could serve as reference data for evaluation of concrete mixtures and/or test methods in future work. Data could also be used to supplement additional databases on maintained by the Materials and Tests Unit.
Recommended Training	None recommended at this time.

Research Product 4	Digital database of test results from laboratory testing
Suggested User	Structures Management Unit, Materials & Tests Unit, and Pavement Design & Collection Unit
Recommended Use	This information could be utilized to supplement other data from Materials & Tests unit or other sources to support design, construction, and decision making.
Recommended Training	None recommended at this time.

Research Product 5	**S**urface resistivity measurements of a variety of North Carolina concrete bridge and pavement mixtures.
Suggested User	Materials & Tests Unit
Recommended Use	Surface resistivity tests results have been shown to strongly correlate with results of ASTM C1202 RCPT results by this study and others. Surface resistivity measurements could be utilized to specify more durable concrete and to evaluate the durability of existing concrete.
Recommended Training	If surface resistivity is integrated into procedures utilized by the Materials & Tests Unit, minimal training on the device would be required. AASHTO standard T 358-17 can be used as guidance for use of the surface resistivity meter in the laboratory setting. UNC Charlotte personnel are working to implement technology transfer and training as part of NCDOT RP 2019-41.

Research Product 6	Super Air Meter (SAM) tests of a variety of North Carolina concrete pavement mixtures.
Suggested User	Materials & Tests Unit
Recommended Use	The SAM device has been shown to be useful in evaluating the potential freeze-thaw durability of a concrete mixture prior to being placed. This data can be used in the ongoing evaluation of this test device for use in North Carolina.
Recommended Training	This test is outlined in AASHTO TP 118, and online training videos are available. UNC Charlotte personnel are working to implement technology transfer and training as part of NCDOT RP 2019-41.

8. REFERENCES

Note: References listed below are cited in the body of the report. A full list of references utilized to support this work is provided at the end of Appendix A, which contains the complete Literature Review for this project.

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APPENDICES

FOR
FINAL REPORT

North Carolina Department of Transportation
Research Project No. 2016-06

Internal Curing of Concrete Using Lightweight Aggregates

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May 2019

APPENDIX A – LITERATURE REVIEW AND REFERENCES

A.1 Motivation for Study and Research Needs

The necessity of, and benefits from, traditional "external" curing have been understood for many years. Curing provisions that maintain adequate moisture and temperature conditions for a sufficient duration to allow the concrete achieve the desired properties is essential. Inadequate curing results in cessation of hydration, and loss of moisture that can lead to cracking (Weiss et al. 2012). External curing methods include tactics to supply additional moisture to replace moisture lost to hydration and evaporation, sealing in the mix water, and applying moisture and heat to accelerate hydration (ACI 2016). Fogging, sprinkling, and application of wet coverings, plastic sheeting, and membrane compounds are commonly utilized methods of external curing (NRMCA 2000, Kosmatka and Wilson 2016).

In recent years, studies have shown that providing reservoirs of additional water inside concrete can be used to facilitate hydration or replace moisture lost due to evaporation or self-desiccation, which can lead to early-age cracking (Bentz and Weiss 2011). This method, known as internal curing, is defined by ACI "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)." A variety of materials possess the characteristics necessary to support internal curing, 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 LWA were utilized to facilitate internal curing. Therefore, for this literature review, focus is placed on use of LWA for internal curing of concrete.

Internally cured concrete mixtures using prewetted LWA have been shown to provide a number of performance benefits over comparable conventional mixtures (Babcock and Taylor 2015). Internal curing can significantly reduce or eliminate plastic shrinkage (Schlitter et al. 2010) and autogenous shrinkage (Bentur et al. 2001, Kovler et al. 2004, Cusson and Hoogeveen 2010, Ardeshirilajimi et al. 2016), reducing cracking potential. Internally cured concrete containing LWA has a lower modulus of elasticity, which has also been linked to reduced cracking tendency (Wolf 2008, Byard and Schindler 2010). Compressive strength (Byard and Schindler 2010) and flexural strength (Roberts 2004 and 2005) of these mixtures can be increased. Durability performance improvements associated with internally cured mixtures using prewetted LWA include: reduced slab warping (Wei and Hansen 2008), reduced chloride permeability (Bentz 2009, Cusson and Margeson 2010), and enhanced freeze-thaw resistance (Cusson and Margeson 2010). Other benefits are associated with the reduced unit weight of internally cured concrete using prewetted LWA. Structural deadloads are decreased, which can reduce the required sizes of other structural members. Fuel savings for lowered hauling loads could also provide sustainability benefits (Villarreal and Crocker 2007).

Due to the tendency of suppliers to prewet lightweight aggregates prior to batching, internally cured concrete has been inadvertently utilized over the last hundred years in many conventional lightweight concrete mixtures. However, until recently, the internal curing technology has not been intentionally incorporated into concrete mixtures at the proportioning stage (Bentz and Weiss 2011). 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 state highway agencies (SHAs), including those serving Texas, Colorado, Utah, Illinois, Indiana, Ohio, Virginia, West Virginia, and New York, have performed 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 SHAs 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 are not the only factors contributing to cracking and other distresses. Factors that are generally accepted to be influential in concrete cracking include: structural design, concrete mixture design, materials used in the concrete mixture, and construction practices including placing, finishing and curing (TRB 2006). A discussion on distresses due to the design or construction of concrete 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) potential for cracking.

A.2 Overview of Internal Curing

A.2.1 Mechanisms

While traditional curing is commonly specified in concrete construction, it is an additional step that is sometimes overlooked or is 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 (ACI 2016).

Adequate curing of the concrete is important to prevent two primary 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). Sufficient curing is essential to reducing permeability and preventing early-age cracking, ultimately extending service life and yielding low maintenance costs (Delatte and Cleary 2008, Cusson et al. 2010). In mixtures typically used for slabs-on-grade, research has also shown that internal curing can reduce warping (Wei and Hanson 2008).

A diagram illustrating the comparison between external and internal curing is provided in Figure A.1. It can be seen in the diagram that the effectively cured zone is significantly higher in the internally cured concrete, and concrete throughout the element (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 should be performed in addition to external curing, not as a substitution.

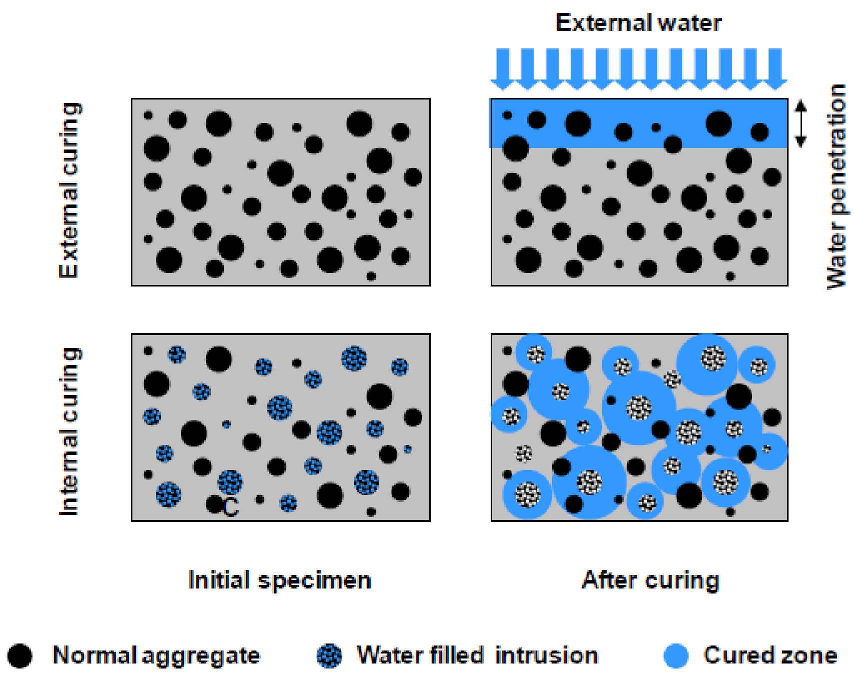

Figure A.1: Comparison of external and internal curing (Castro et al. 2011)

Since the curing of concrete highly influences its final mechanical properties and durability performance, 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). Concrete mixtures with relatively low water-to-cementitious materials content ratios (*w/cm* of 0.36 or less) have a higher likelihood of the mixing water not satisfying the curing requirements (Bentz et al. 2005). Powers et al. (1959) also noted 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 unhydrated 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).

High strength concrete mixtures typically utilize low w/cm ratios and 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.

Over the past 50 years, portland cement has become finer and contains higher contents of tricalcium silicate and alkalis (Bentz 2007). These changes have led to generally faster hydrating cements that produce much of their strength in only a few days (ACI 2013). 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 (ACI 2013 and others). 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-desiccation.

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). 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.

A.2.2 Materials

The benefits of internal curing can be utilized throughout a variety of materials including prewetted LWAs, prewetted crushed returned concrete fines, superabsorbent polymers (SAP), perlite, and prewetted wood fibers (Byard and Schindler 2010, Bentz and Weiss 2011). Each of these materials are able to provide an internal source of available moisture to replace that consumed by chemical shrinkage during hydration, however, several do not provide structural strength to the mixture (Byard and Schindler 2010). Prewetted manufactured LWAs provide both internal curing benefits, as well as structural capacity, and are therefore the subject of this literature review and study.

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 relatively large 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 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). 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). The moisture must be readily transported into the paste matrix in order to be effective (Kovler et al. 2004). It is noted by Castro et al. (2011) that each of the aggregates tested exhibited what can be considered 'good desorption behavior.' Using a NIST-developed model, research by Henkensifeken et al. (2008) indicated that even at low LWA replacement levels, internal curing moisture could be assumed to protect most of the paste volume if the moisture could travel 1.0 mm. More impermeable mixtures could require higher LWA replacement rates to provide similar paste protection.

As mentioned previously, LWAs are generally classified into three 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). 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 (Castro et al. 2011, Rao and Darter 2013).

A.2.3 Methods

Three primary factors influence the effectiveness of internal curing, 1) the volume of water required from the LWA, 2) the ability of the LWA to release that water (desorption), and 3) the distribution of the LWA within the mixture (Henkensiefken 2009, Babcock and Taylor 2015). In relatively early research in this area, Philleo (1991) recommended the replacement of normal weight 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, as described in research by Henkensiefken (2008). The use of coarse LWA for internal curing purposes results in a microstructure similar to diagram (a) in Figure A.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 A.2. The moisture reservoirs are now much more evenly distributed within the paste microstructure resulting in roughly 100% protection of the paste.

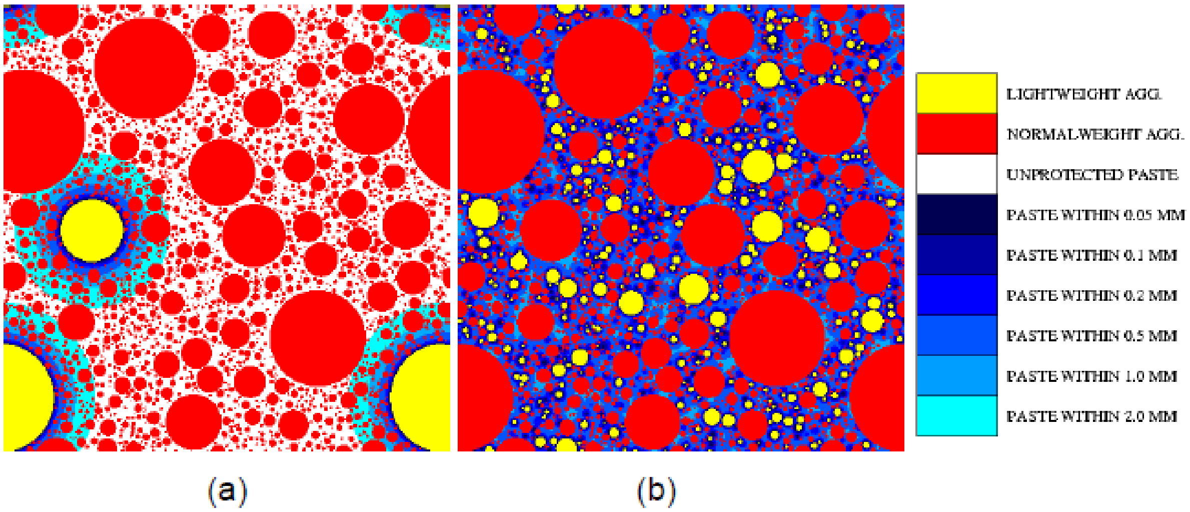

Figure A.2: Spatial distribution of coarse LWA vs. fine LWA (Henkensiefken 2008)

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 normal weight coarse and fine aggregate with saturated lightweight intermediate aggregate (Villareal and Crocker 2007). Traditional lightweight concrete construction has utilized replacement of normal weight 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 available 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 subsequently referred to as Equation A.1.

$$C_f \times CS \times \alpha_{max} = S \times \Phi_{LWA} \times M_{LWA} \qquad \text{(A.1)}$$

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 are shown in Figures A.3 and A.4.

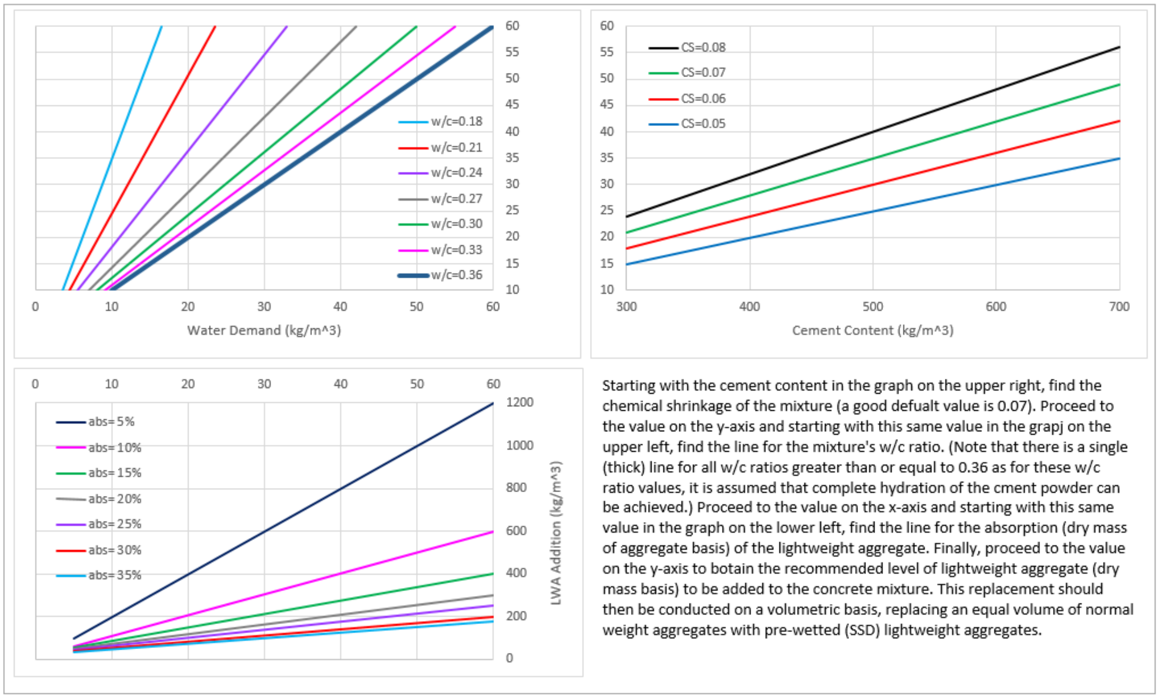

Figure A.3: Nomograph in SI units (Bentz 2009)

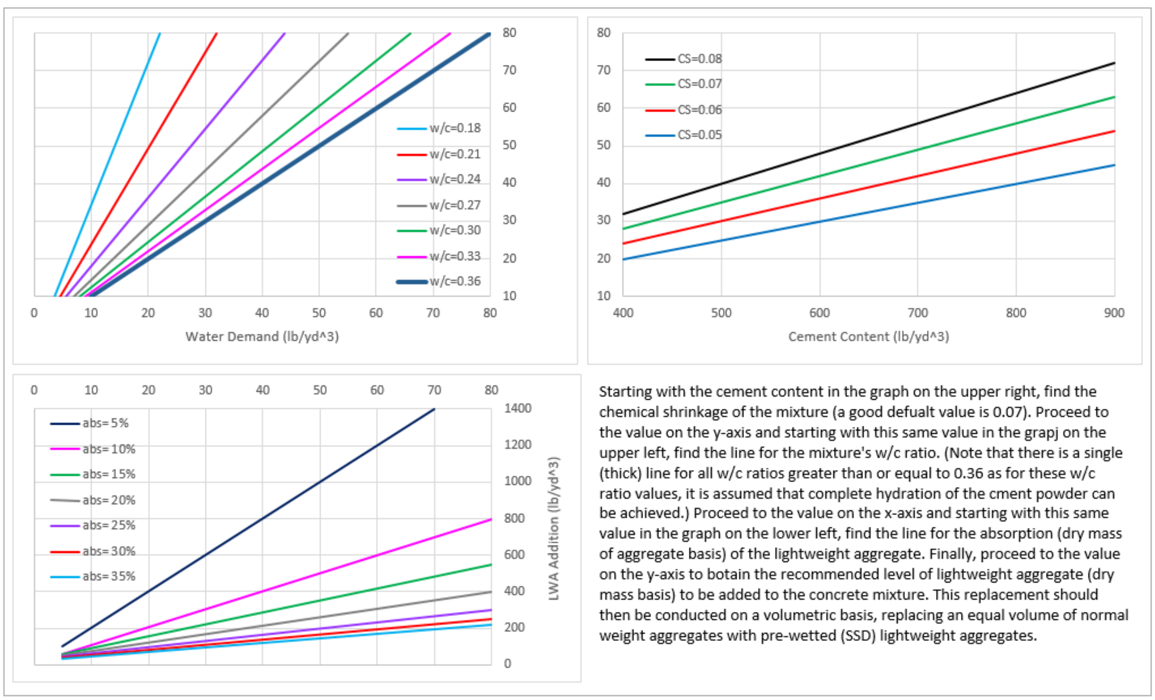

Figure A.4: Nomograph in English units (Bentz 2009)

Equation A.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 A.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. As generally recommended for concrete mixture designs, the importance of trial batching cannot be understated. ACI 208-13 recommends developing a three-point curve to determine adequate LWA replacement rates (2013) for the desired mixture characteristic(s) (ACI 2013).

A.3 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.

A.3.1 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 should be given to the characteristics of absorption and desorption, for the reasons discussed in Section A.2.2 above.

Optimal use of prewetted LWA as an internal curing agent typically requires that the aggregate be in, or close to, a saturated condition prior to mixing. This condition can be achieved in the laboratory by either the paper towel method prescribed in ASTM C1761 (ASTM 2017) or by the centrifuge method described in Miller et al. (2014), the Indiana Testing Method (ITM) 222 (IDOT 2015)), or the New York State DOT Test Method NY 703-19E (NYSDOT 2008). 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 aggregate's desorption characteristics compared to industry standards according to ASTM C1761. Most LWAs possess similar desorption isotherms, 97% or greater but some LWAs do not desorb this magnitude of water at 94% RH and therefore laboratory testing is still recommended (Henkensiefken et al. 2009).

A.3.2 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. 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). Moderate replacement rates of LWA will also ensure that the concrete unit weight will remain above 135 pcf, eliminating the need for additional structural design considerations (ACI 2014a, 2014b, AASHTO 2012).

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 Figures A.3 and A.4 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 normal weight 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 normal weight 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 (Bentz et al. 2005, 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 (ACI 2014a, ACI 2014b, 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. (2016) 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.

A.3.3 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). 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. Researchers 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 $/cy 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).

A.3.4 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). Pavement mixtures also tend to have lower slump and lower compressive and flexural strengths (NCDOT 2012) than mixtures used for other types of structures. 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. 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.” TxDOT also noticed an improvement in compressive strength during testing of specimens of ICC versus conventional concrete. 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).

A.3.5 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 very high early strength LMC was not identified.

A.4 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 previously, 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 adhere 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 A.5 is a schematic showing 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.

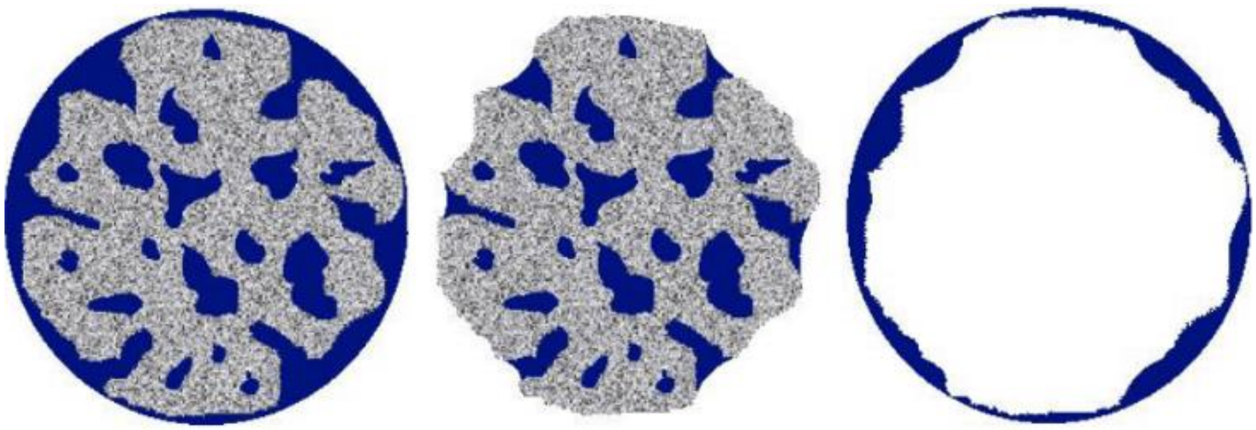

Figure A.5: Schematic explaining surface moisture of aggregate (Barrett et al. 2015)

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).

Many state agencies report no differences in placing and finishing of internally cured concrete, compared to placing and finishing of similar conventional mixtures (Villareal and Crocker 2007, Streeter et al. 2012). Some agencies reported a slight increase in bleed water which made placement easier (Villareal and Crocker 2007, Bentz and Weiss 2011). Researchers, state agencies, and other guidance documents stress that traditional external 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).

A.4.1 Specification Provisions

The literature indicates 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).

Many state agencies tend to simplify the Bentz and Snyder approach (equation A.1) 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, NYSDOT and Illinois DOT specify that a 30% replacement of LWA for normalweight fines be utilized, regardless of the absorptive capacity of the LWA or the chosen cement content (Speck 2018). However, NYSDOT also specifies the use of a LWA with a minimum absorption capacity of 15%. Indiana DOT utilizes the Bentz and Snyder approach (Equation 1.1) to compute the LWA replacement, but indicates that in no cases should it be less than 30% of sand by volume (Speck 2018). West Virginia DOT also provides an approach to compute the amount of LWA to be included in the mixture, but states that in no case shall the volume of SSD LWA be less than 25% of the total fine aggregate SSD volume of the entire mix (WVDOT 2016). Louisiana DOTD specifies 225 to 275 pounds of prewetted LWA be used per cubic yard of concrete, with the normalweight sand reduced by the volume of the prewetted LWA added (Speck 2018) 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.

The benefits of laboratory production of concrete (such as controlled environments) 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 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.

A.4.2 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 (Nantung 2016).

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.

A.4.3 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, and
- Justifying increased initial costs due to addition of manufactured LWA.

A.5 Research Needs

Extensive research to identify, quantify, and evaluate the benefits of use of ICC have been performed. 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.

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APPENDIX B – SUPPORTING MATERIAL FOR LABORATORY TESTING (Chapter 3)

Version 6.42

Material Certification Report

Portland Cement
I-II(MH)

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

Certification

This Holcim cement meets the specifications of ASTM C150 for Type I-II(MH) cement, and complies with AASHTO M85 specifications for Type I-II(MH) cement.

General Information

Supplier:		Source Location:	Holly Hill Plant
Address:			2173 Gardner Boulevard Holly Hill, SC 29059
Telephone:		Contact:	Scott Poaps
Date Issued:			

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 Scott Poaps, Quality Manager

Figure B.1: Mill report for portland cement

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	
	Results	**Specification (Class F)**	
Chemical Analysis**	**(wt%)**	**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.2: Fly ash test report

Table B.1: Sieve analysis for coarse aggregate

Sieve Size	Percent Passing
1”	100
¾”	96
½”	55
3/8”	33
No. 4	5
No. 8	2
No 200 Decant (%)	0.3

Table B.2: Sieve analysis results for fine aggregates

	Percent Passing		
Sieve Size	LWA1	LWA2	Normalweight C33
No. 4	99	99.8	100
No. 8	92	82	100
No. 16	56	51	87
No. 30	34	29	25
No. 50	20	15	13
No. 100	11	8	3
Pan	0	0	0

Figure B.3: Gradation curves for fine aggregates

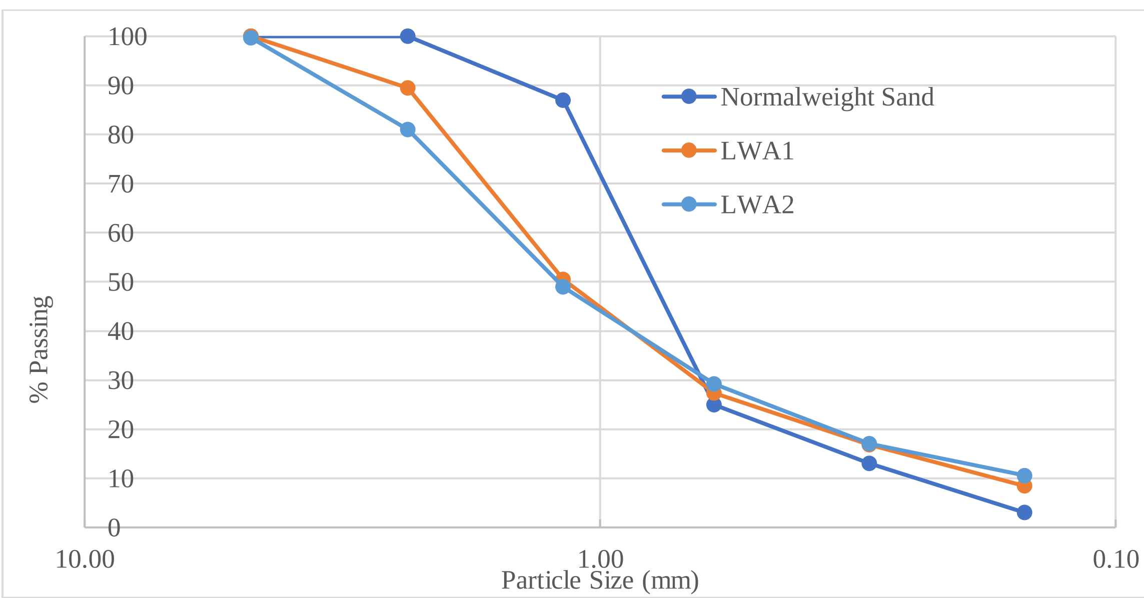


Table B.3: Fresh property test results

Type of Mixture	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.
Conventional Bridge Deck	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
Latex-Modified Concrete (LMC)	CLMC	10.25	-	-	10.25	7.5%	-	7.5%	134.2	-	-	134.2
	ILMC	9.75	-	-	9.75	5.6%	-	5.6%	132.3	-	-	132.3
	CL	4.50	-	-	4.50	6.6%	-	6.6%	139.6	-	-	139.6
	ILA	11.00	-	-	11.0	10.0%	-	10.0%	126.2	-	-	126.2
	ILB	3.50	-	-	3.50	6.2%	-	6.2%	138.9	-	-	138.9
VHES LMC	RSCL	3.50	-	-	3.50	5.7%	-	5.7%	139.5	-	-	139.5
	RSIL	3.75	-	-	3.75	4.4%	-	4.4%	137.0	-	-	137.0
Pavement	P.B.N.N*	3.30	-	-	3.30	5.4%	-	5.4%	142.0	-	-	142.0
	IP	7.50	-	-	7.50	5.2%	-	5.2%	135.4	-	-	135.4

* mixture from NCDOT RP 2015-03

Table B.4: 28-day compressive strength results for each batch

Mixture ID	28-day Compressive Strength (psi)			Average Compressive Strength (psi)	Standard Deviation
	1	2	3		
CC	7842	7604	7913	7786	162
I1M	5356	5290	5107	5251	129
I2M	7139	7003	6650	6931	252
I1H	5036	5667	5613	5439	350
I2H	5309	4657	4847	4938	335
CF	4956	4625	5248	4943	312
I1MF	5111	5217	5413	5247	153
I2MF	5391	5382	5904	5559	299
I1HF	5518	5007	5148	5224	264
I2HF	5545	5543	5415	5501	74
CLMC	7023	7436	-	7230	292
ILMC	5008	5426	-	5217	296
CL	5304	-	-	5304	-
ILA	4154	-	-	4154	-
ILB	5275	-	-	5275	-
RSCL	6749	-	-	6749	-
RSIL	7397	-	-	7397	-
P.B.N.N*	4220	4458	4484	4387	145
IP	4784	4310	4867	4654	301

* mixture from NCDOT RP 2015-03

Table B.5: 28-day modulus of elasticity test results

Mixture	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
CLMC	3,528,130	3,773,139	3,650,635	173,248
ILMC	3,940,835	3,670,950	3,805,893	190,838
CL	2,967,566	3,462,924	3,215,245	350,271
ILA	2,220,769	2,466,633	2,343,701	173,852
ILB	2,837,427	3,086,956	2,962,192	176,444
RSCL	4,121,585	3,809,167	3,965,376	220,913
RSIL	3,045,383	3,866,555	3,455,969	580,656
P.B.N.N*	2,219,808	4,109,804	3,514,806	841,454
IP	2,218,806	2,398,764	2,308,785	127,250

* mixture from NCDOT RP 2015-03

Table B.6: Poisson's ratio test results

Mixture	28-day Poisson's Ratio		Average Poisson's Ratio	Standard Deviation
	Specimen 1	Specimen 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	0.04
I1H	0.19	0.24	0.22	0.01
I2H	0.21	0.22	0.22	0.00
CF	0.23	0.23	0.23	0.01
I1MF	0.23	0.24	0.24	0.03
I2MF	0.22	0.26	0.24	0.01
I1HF	0.23	0.21	0.22	0.01
I2HF	0.21	0.22	0.22	0.01
CLMC	0.23	0.22	0.23	0.01
ILMC	0.25	0.24	0.25	0.01
CL	0.21	0.21	0.21	0.00
ILA	0.21	0.23	0.22	0.01
ILB	0.21	0.26	0.24	0.03
RSCL	-	-	-	-
RSIL	-	-	-	-
P.B.N.N*	0.18	0.20	0.19	0.01
IP	0.21	0.20	0.21	0.01

* mixture from NCDOT RP 2015-03

Table B.7: CTE test results

Mixture	28-day CTE (x10^{-6} in/in^{0}F)			Average CTE (x10^{-6} in/in^{0}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
CLMC	6.65	6.71	6.82	6.73	0.09
ILMC	5.86	5.97	6.06	5.96	0.10
CL	-	-	-	-	-
ILA	-	-	-	-	-
ILB	-	-	-	-	-
RSCL	-	-	-	-	-
RSIL	-	-	-	-	-
P.B.N.N*	5.312	5.29	5.33	5.31	0.02
IP	4.95	5.07	5.00	5.01	0.06

* mixture from NCDOT RP 2015-03

Table B.8: Surface resistivity test results (average of three specimens)

Mixture	3-day	7-day	28-day	90-day
CC	13.3	12.7	16.3	17.9
I1M	9.9	10.6	13.7	14.7
I2M	9.3	10.0	13.3	16.7
I1H	9.3	10.7	13.6	14.7
I2H	8.1	9.1	12.5	14.4
CF	10.4	11.1	18.1	46.1
I1MF	8.8	10.3	17.5	45.7
I2MF	7.4	8.1	17.0	36.9
I1HF	7.1	9.2	17.0	43.3
I2HF	6.9	7.6	16.5	43.6
CLMC	18.7	22.0	-	-
ILMC	12.6	16.7	-	-
CL	18.7	24.5	33.8	39.8
ILA	14.4	17.5	24.1	28.6
ILB	22.5	31.1	42.2	51.5
RSCL	263.7	201.2	127.3	130.4
RSIL	119.0	78.5	49.4	76.3
P.B.N.N*	8.0	8.7	10.7	10.8
IP	5.3	6.4	8.2	9.1

* mixture from NCDOT RP 2015-03

Table B.9: Bulk conductivity test results (average of three specimens)

Mixture	28-day (mS/m)	90-day (mS/m)
CC	9.95	6.1
I1M	13.94	6.34
I2M	12.54	9.13
I1H	10.23	6.22
I2H	11.57	5.23
CF	9.93	3.68
I1MF	9.50	3.34
I2MF	9.27	4.12
I1HF	11.65	3.6
I2HF	10.17	3.42
CLMC	2,170	1,290
ILMC	3,250	2,740
CL	-	-
ILA	-	-
ILB	-	-
RSCL	120	260
RSIL	425	310
P.B.N.N*	4,390	1,320
IP	6,190	2,800

* mixture from NCDOT RP 2015-03

Table B.10: Rapid chloride permeability test results (average of three specimens)

Mixture	28-day (coulombs)	90-day (coulombs)
CC	2,962	2,190
I1M	3,382	2,620
I2M	3,184	2,550
I1H	3,249	2,890
I2H	2,920	2,790
CF	2,444	590
I1MF	2,143	590
I2MF	2,658	960
I1HF	2,475	810
I2HF	2,318	750
CLMC	-	-
ILMC	-	-
CL	28.50	23.95
ILA	15.22	17.84
ILB	26.04	32.33
RSCL	83.47	96.84
RSIL	41.28	63.25
P.B.N.N*	-	-
IP	18.44	7.73

* mixture from NCDOT RP 2015-03

Table B.11: Freeze-thaw durability test results

Specimen ID	Average Post Test P_c	Average Post Test DF	% Mass Chance	Cycles Completed
CC	85.22	85.22	0.39	300
I1M	88.39	88.39	0.17	300
I2M	100.96	100.96	-0.23	300
I1H	91.67	91.67	0.20	300
I2H	83.80	83.80	0.48	300
CF	80.15	80.15	0.29	300
I1MF	87.15	87.15	0.23	300
I2MF	85.98	53.84	-1.57	300
I1HF	83.19	83.19	-0.38	300
I2HF	42.23	12.67	1.45	90

Table B.12: ASTM C157 shrinkage test results

	Length change due to shrinkage (%)				
Mixture ID	2 weeks	4 weeks	8 weeks	16 weeks	64 weeks
CC	0.0215	0.0318	0.0409	0.0509	0.063
I1M	0.0227	0.0327	0.043	0.0561	0.0683
I2M	0.0179	0.0315	0.0442	0.0539	0.069
I1H	0.0252	0.0333	0.043	0.0479	0.0683
I2H	0.0173	0.0294	0.0442	0.0579	0.0783
CF	0.0248	0.0348	0.0427	0.0539	0.0637
I1MF	0.0197	0.0282	0.0382	0.0476	0.061
I2MF	0.0185	0.0279	0.04	0.0445	0.0603
I1HF	0.0255	0.0291	0.0403	0.0488	0.063
I2HF	0.0158	0.0255	0.0382	0.0576	0.0703
CLMC	0.0327	0.0353	0.053	0.0623	0.08
ILMC	0.017	0.0287	0.0497	0.0583	0.071
CL	-	-	-	-	-
ILA	-	-	-	-	-
ILB	-	-	-	-	-
RSCL	0.0005	0.002	0.0055	0.0105	0.0225
RSIL	0.001	0.0005	0.0005	0.0025	0.0165
P.B.N.N*	0.0001	0.02	0.027	0.0353	0.0493
IP	0.044	0.051	0.062	0.079	0.098

* mixture from NCDOT RP 2015-03

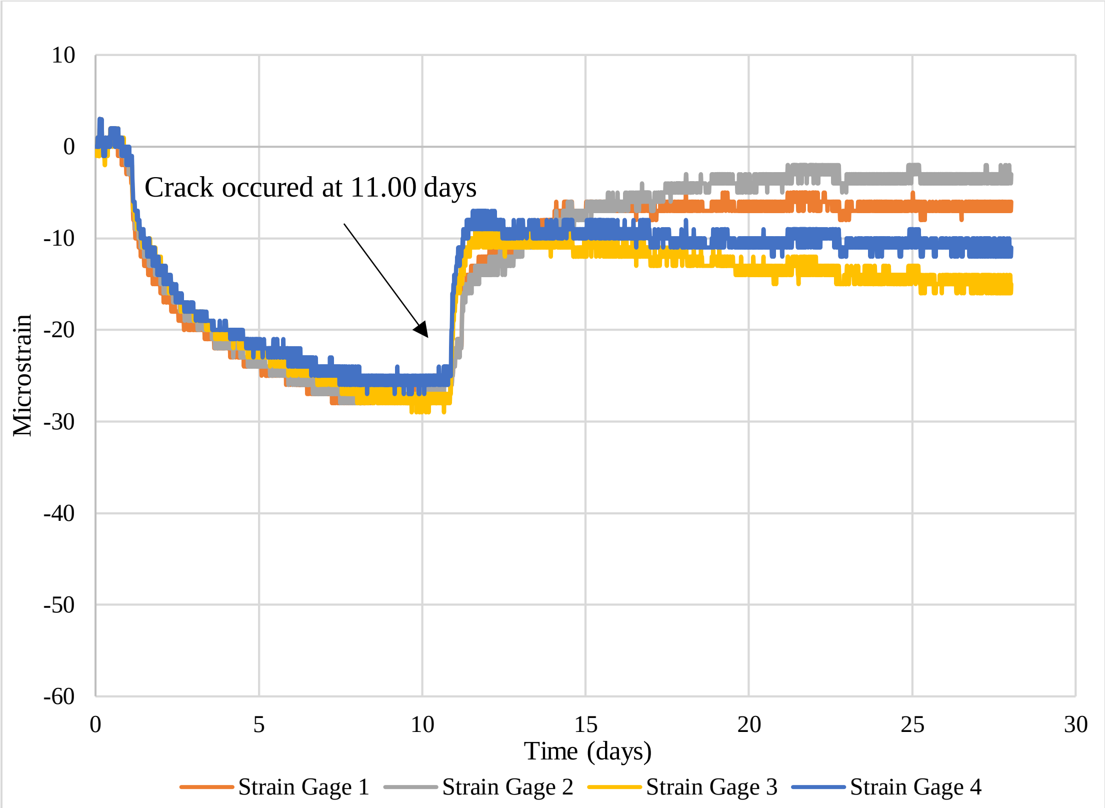


Figure B.4: Example plot of microstrain vs. time for a typical test specimen, showing identification of time to cracking

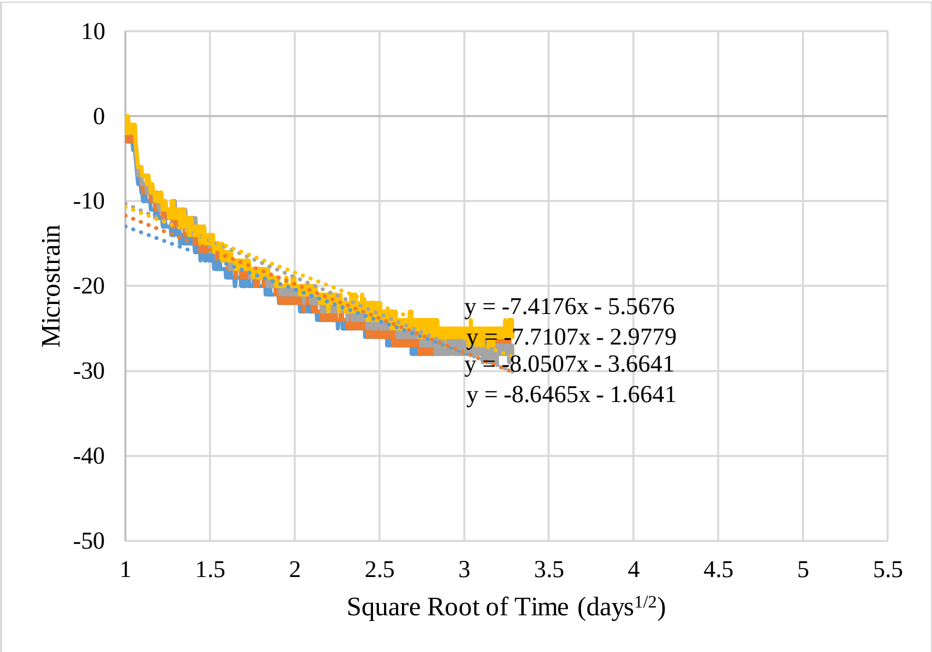


Figure B.5: Example plot of strain vs. square root of time and linear regression, with average slope of lines computed as strain rate factor

Table B.13: ASTM C1581 cracking potential test results

Mixture ID / Specimen #		Precrack rate of strain				Avg. Precrack Rate of Strain	Avg. Rate of Strain (in./in./day)	Stress rate, q (psi/day)	Average stress rate (psi/day)	Cracked ? (Y/N)	Time to crack, Tc (days)*
		Gage #1	Gage #2	Gage #3	Gage #4						
CC	1	3.087	7.277	10.770	9.770	7.73E-06	7.42E-06	11.218	11.023	N	13
	2	7.418	7.711	8.051	8.647	7.96E-06		12.558		Y	11
	3	3.867	7.928	7.952	-	6.58E-06		9.292		Y	13.75
I1M	1	3.151	5.688	11.281	13.967	8.52E-06	7.73E-06	9.853	9.957	Y	20.5
	2	6.987	6.820	7.010	8.860	7.42E-06		9.285		Y	17.5
	3	5.491	5.430	10.828	-	7.25E-06		10.734		Y	12.5
I2M	1	3.126	4.871	11.315	-	6.44E-06	6.99E-06	8.701	8.580	Y	15
	2	-	9.687	10.686	-	1.02E-05		12.747		Y	17.5
	3	2.536	6.138	-	-	4.34E-06		4.290		N	28
I1H	1	6.422	12.333	11.653	10.439	1.02E-05	9.93E-06	10.241	10.562	Y	27.25
	2	6.178	7.221	8.667	15.893	9.49E-06		11.473		Y	18.75
	3	9.590	9.494	9.394	11.847	1.01E-05		9.974		N	28
I2H	1	0.281	0.458	0.426	11.529	3.17E-06	4.56E-06	3.140	4.637	N	28
	2	4.446	6.651	6.659	-	5.92E-06		5.855		N	28
	3	3.001	4.647	6.082	-	4.58E-06		4.916		Y	23.75
CF	1	4.391	3.219	3.559	19.029	7.55E-06	6.27E-06	12.344	7.937	Y	10.25
	2	1.815	3.762	5.075	6.011	4.17E-06		4.121		N	28
	3	8.535	1.647	10.574	7.582	7.08E-06		7.344		Y	25.5
I1MF	1	6.765	9.402	12.230	-	9.47E-06	8.59E-06	11.443	9.344	Y	18.75
	2	7.766	9.077	10.693	12.165	9.93E-06		10.289		Y	25.5
	3	2.373	5.515	5.907	11.671	6.37E-06		6.299		N	28
I2MF	1	-	3.913	-	7.533	5.72E-06	6.19E-06	5.662	7.605	N	28
	2	4.403	7.546	-	10.451	7.47E-06		10.354		Y	14.25
	3	1.485	4.471	10.225	-	5.39E-06		6.798		Y	17.25
I1HF	1	4.830	7.343	8.307	9.531	7.50E-06	6.05E-06	7.703	6.077	Y	26
	2	3.540	5.894	4.574	4.460	4.62E-06		4.567		N	28
	3	4.196	5.402	6.633	7.863	6.02E-06		5.959		N	28
I2HF	1	4.576	5.558	10.788	-	6.97E-06	7.56E-06	8.268	9.062	y	19.5
	2	7.823	5.766	8.107	10.402	8.02E-06		10.846		Y	15
	3	5.768	-	7.621	9.629	7.67E-06		8.073		Y	24.75
IP	1	5.652	6.205	9.124	-	6.99E-06	7.02E-06	8.752	7.552	Y	17.5
	2	8.753	9.158	8.242	12.769	9.73E-06		9.626		N	28
	3	7.956	2.012	3.006	-	4.325E-06		4.278		N	28

* A time to cracking of 28 days with "N" for "Cracked?" indicates no crack was observed during 28 day testing period

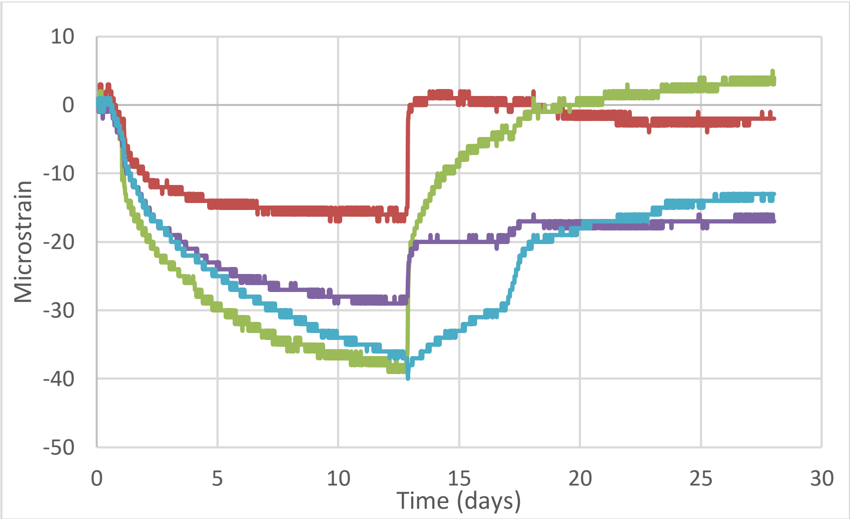


Figure B.6: ASTM C1581 cracking test mixture CC, specimen 1

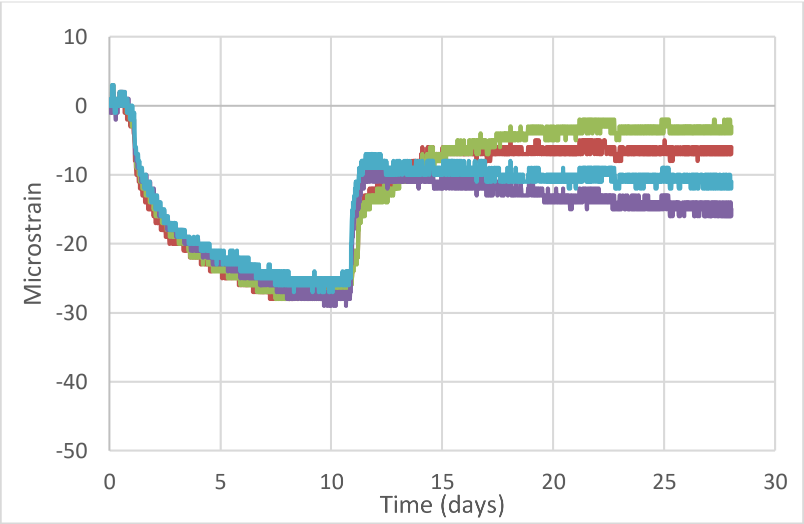


Figure B.7: ASTM C1581 cracking test mixture CC, specimen 2

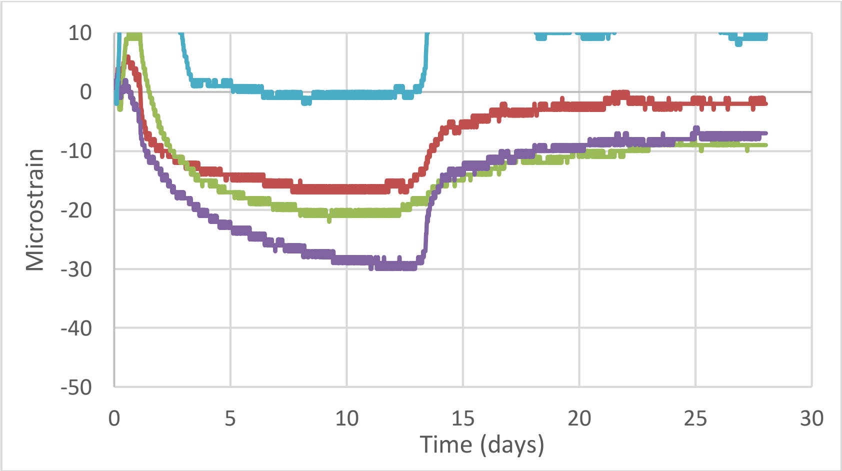


Figure B.8: ASTM C1581 cracking test mixture CC, specimen 3

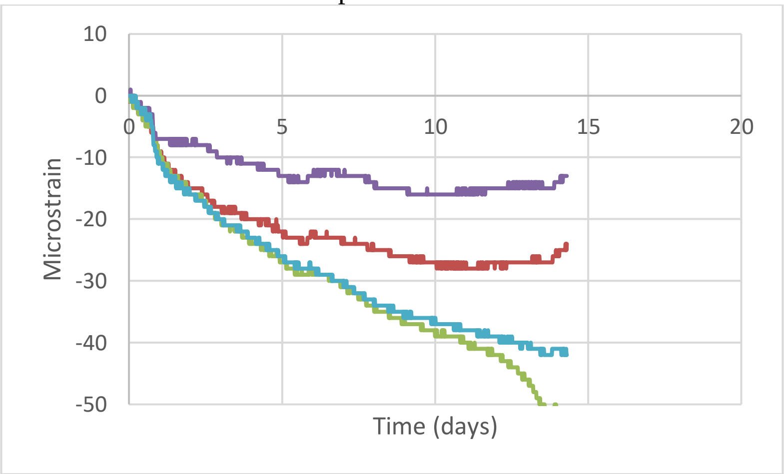


Figure B.9: ASTM C1581 cracking test mixture I1M, specimen 1

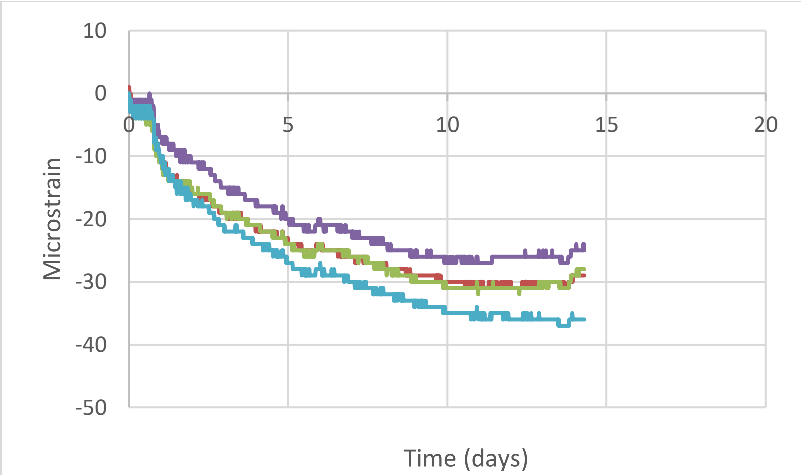


Figure B.10: ASTM C1581 cracking test mixture I1M, specimen 2

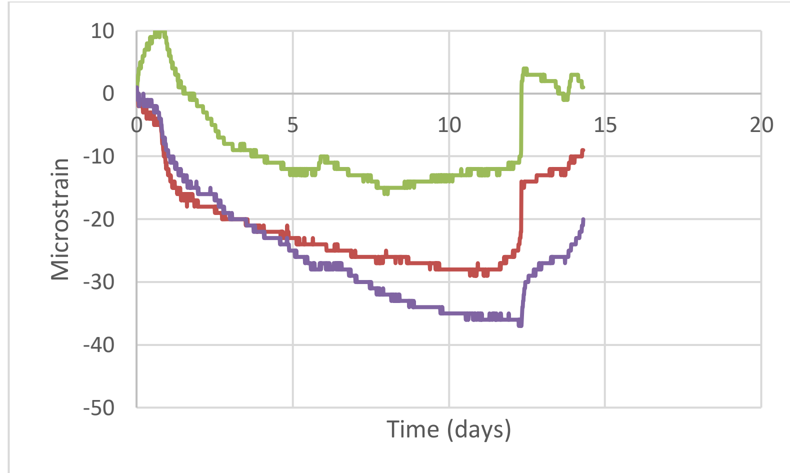


Figure B.11: ASTM C1581 cracking test mixture I1M, specimen 3

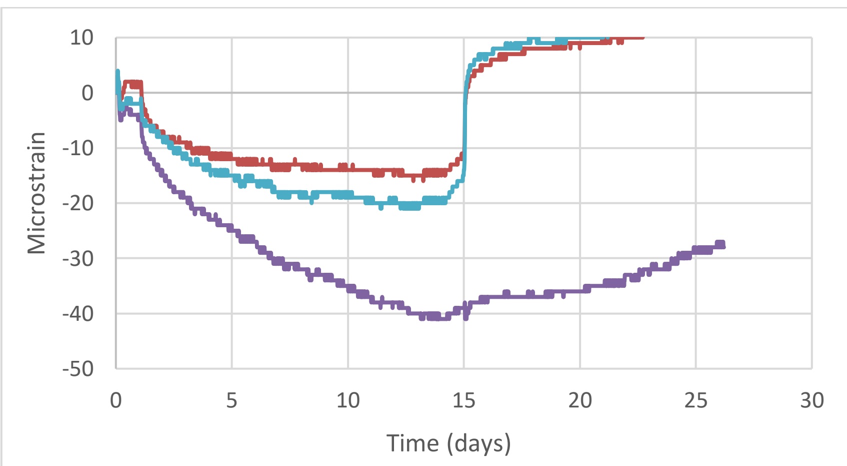


Figure B.12: ASTM C1581 cracking test mixture I2M, specimen 1

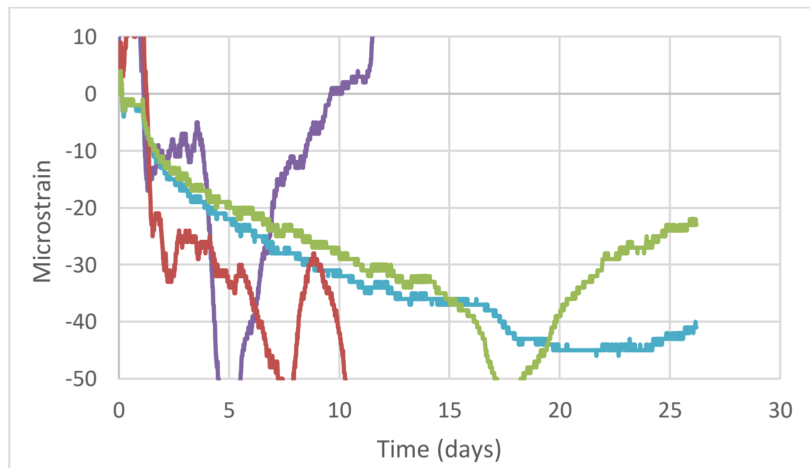


Figure B.13: ASTM C1581 cracking test mixture I2M, specimen 2

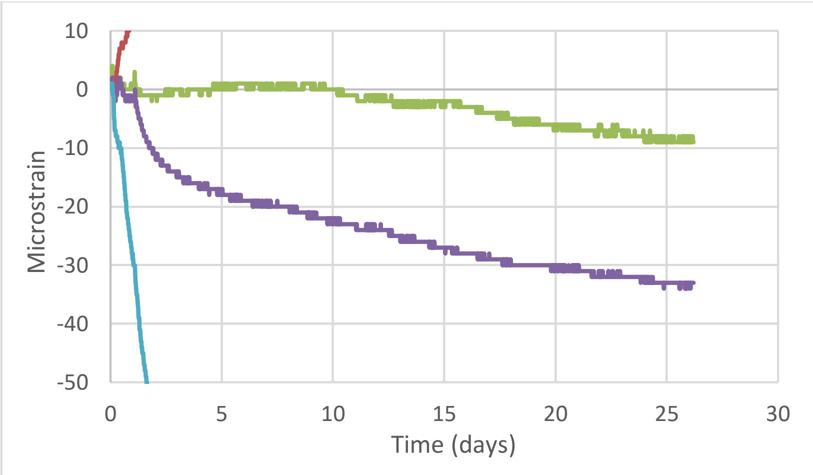


Figure B.14: ASTM C1581 cracking test mixture I2M, specimen 3

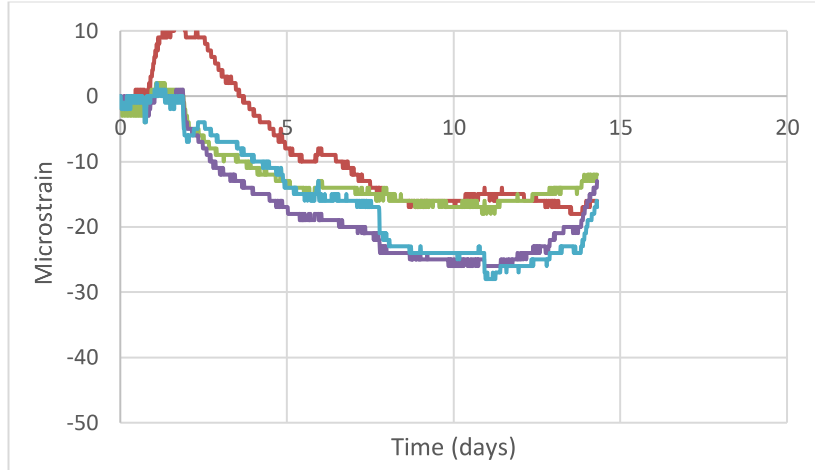


Figure B.15: ASTM C1581 cracking test mixture I1H, specimen 1

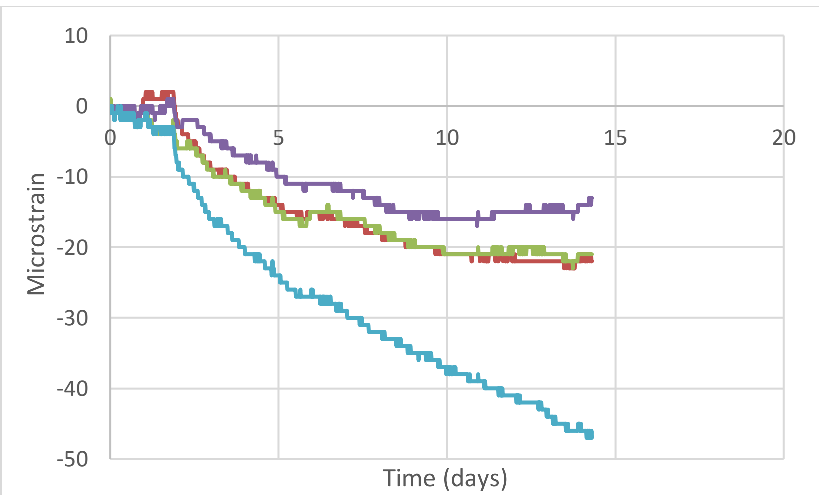


Figure B.16: ASTM C1581 cracking test mixture I1H, specimen 2

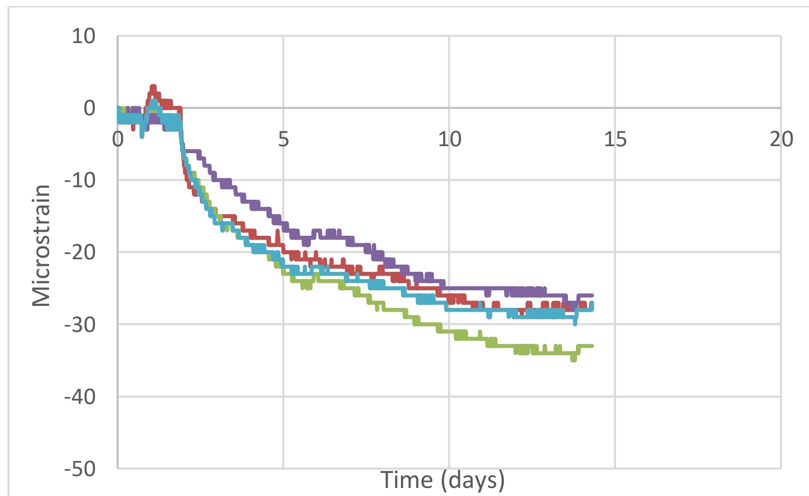


Figure B.17: ASTM C1581 cracking test mixture I1H, specimen 3

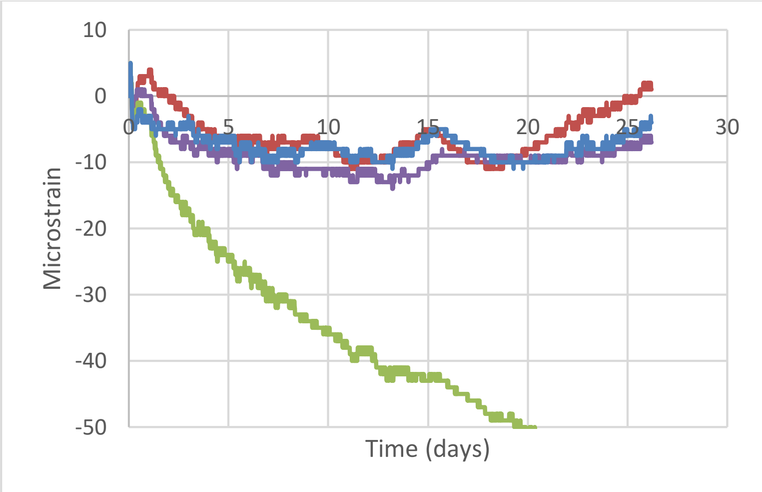


Figure B.18: ASTM C1581 cracking test mixture I2H, specimen 1

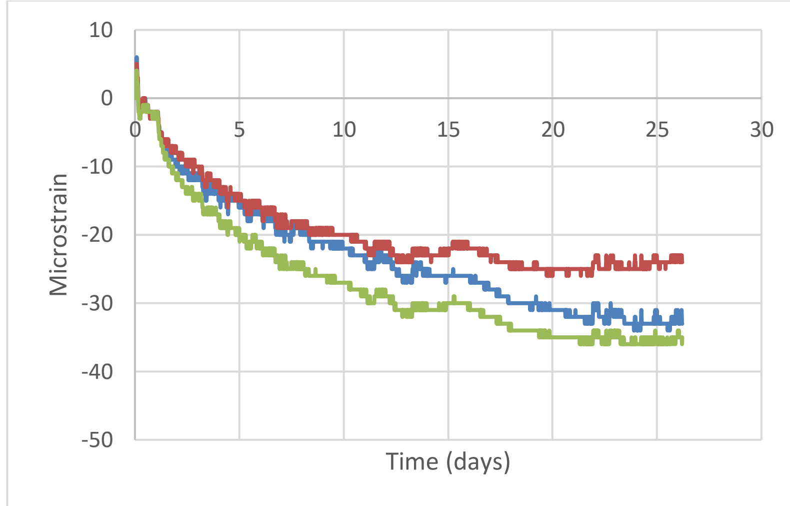


Figure B.19: ASTM C1581 cracking test mixture I2H, specimen 2

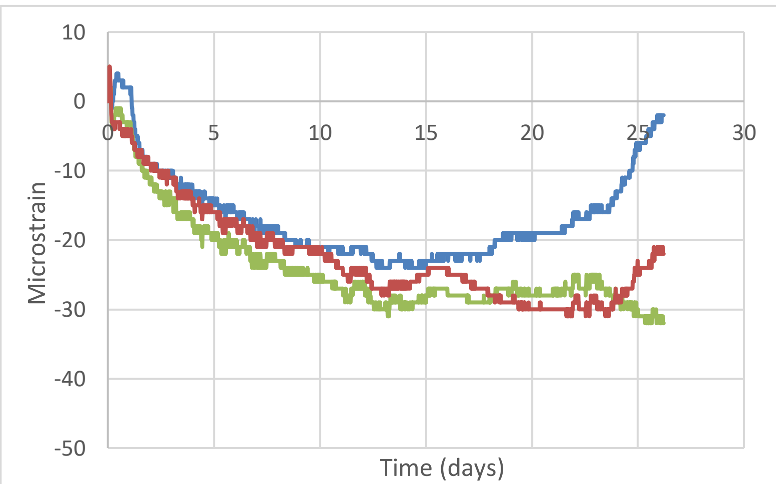


Figure B.20: ASTM C1581 cracking test mixture I2H, specimen 3

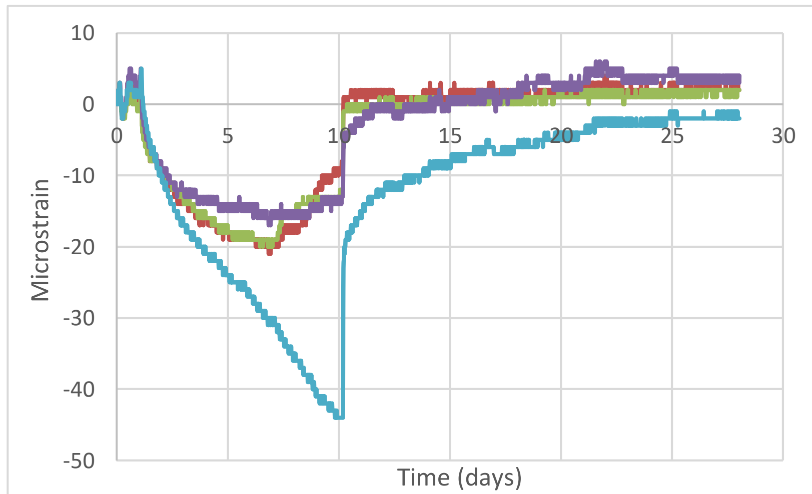


Figure B.21: ASTM C1581 cracking test mixture CF, specimen 1

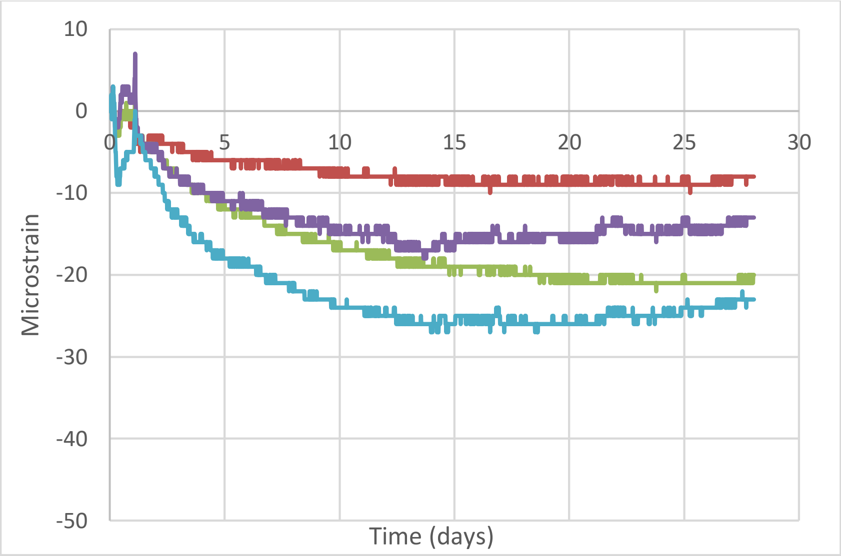


Figure B.22: ASTM C1581 cracking test mixture CF, specimen 2

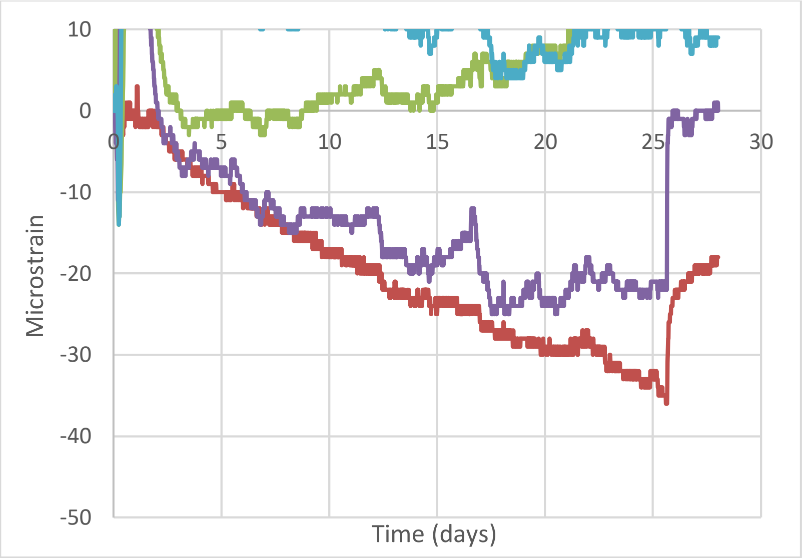


Figure B.23: ASTM C1581 cracking test mixture CF, specimen 3

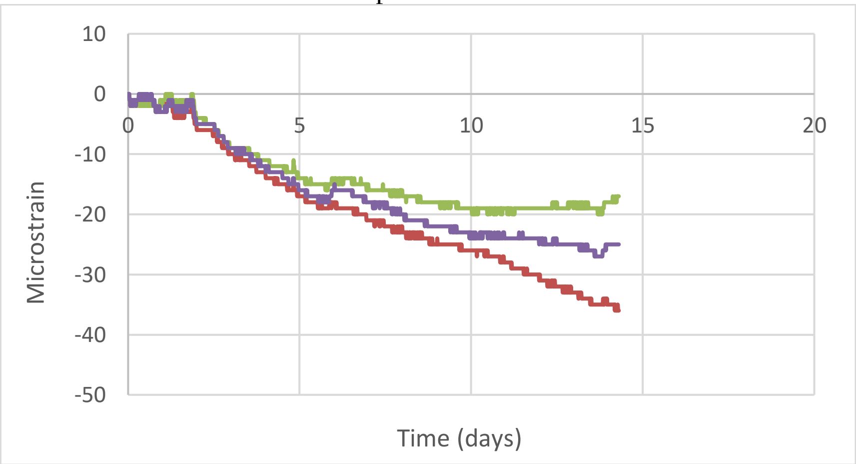


Figure B.24: ASTM C1581 cracking test mixture I1MF, specimen 1

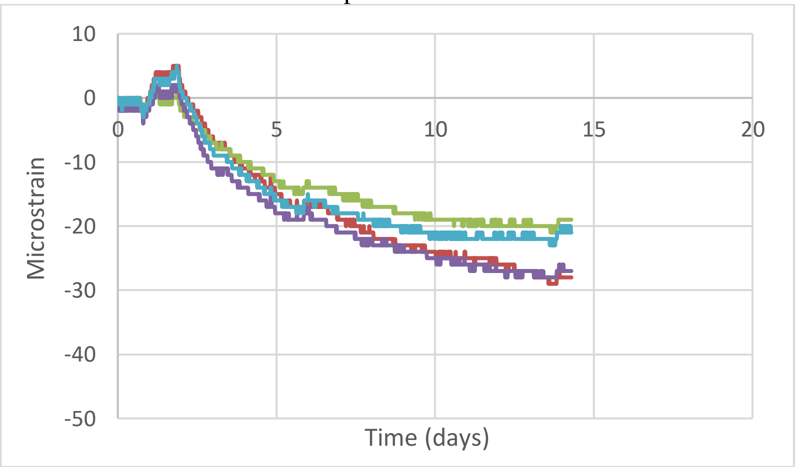


Figure B.25: ASTM C1581 cracking test mixture I1MF, specimen 2

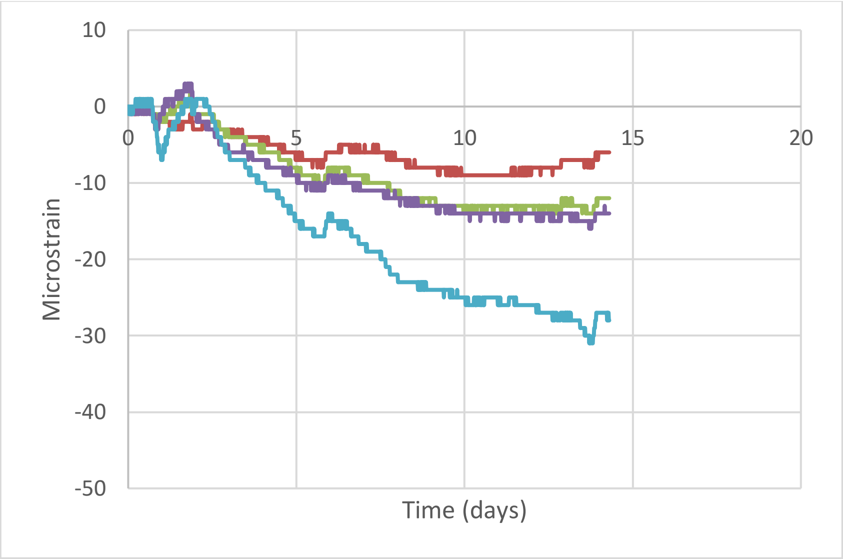


Figure B.26: ASTM C1581 cracking test mixture I1MF, specimen 3

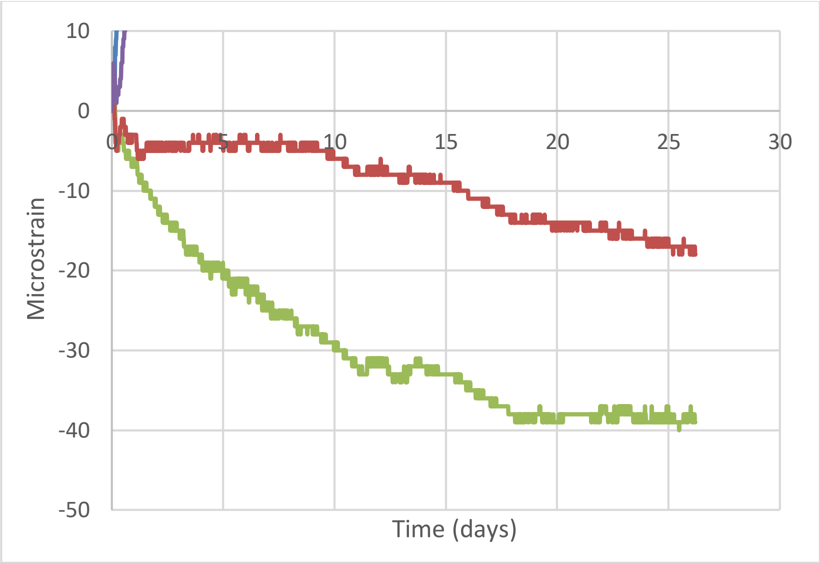


Figure B.27: ASTM C1581 cracking test mixture I2MF, specimen 1

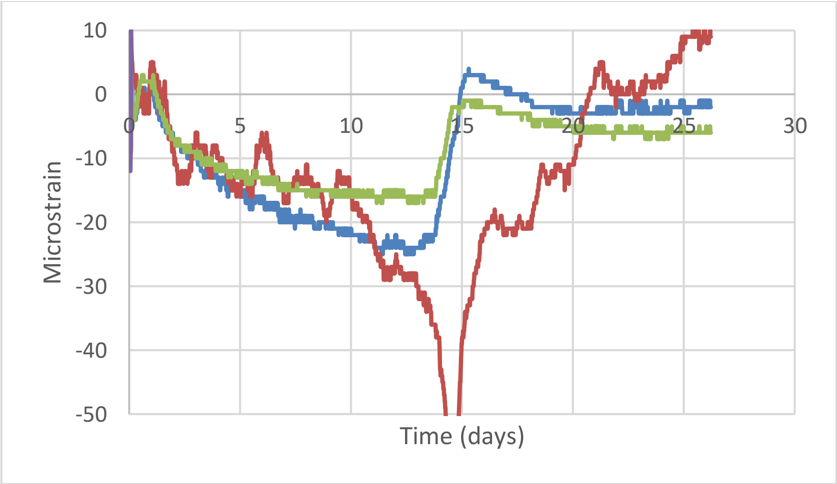


Figure B.28: ASTM C1581 cracking test mixture I2MF, specimen 2

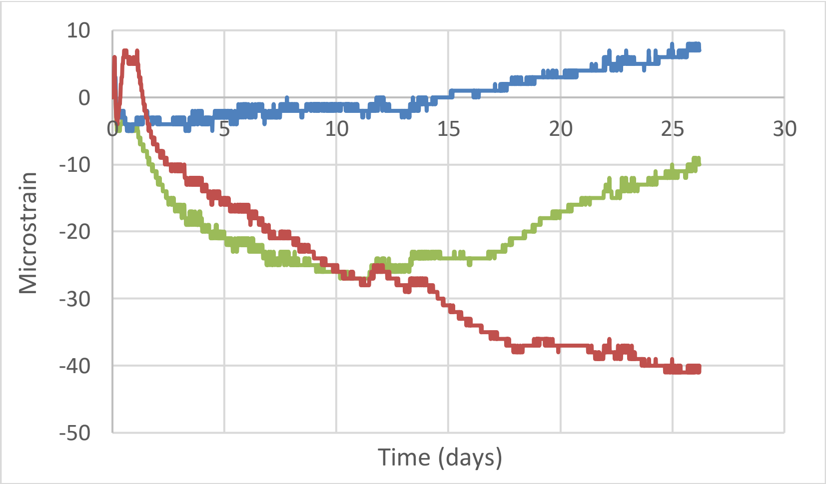


Figure B.29: ASTM C1581 cracking test mixture I2MF, specimen 3

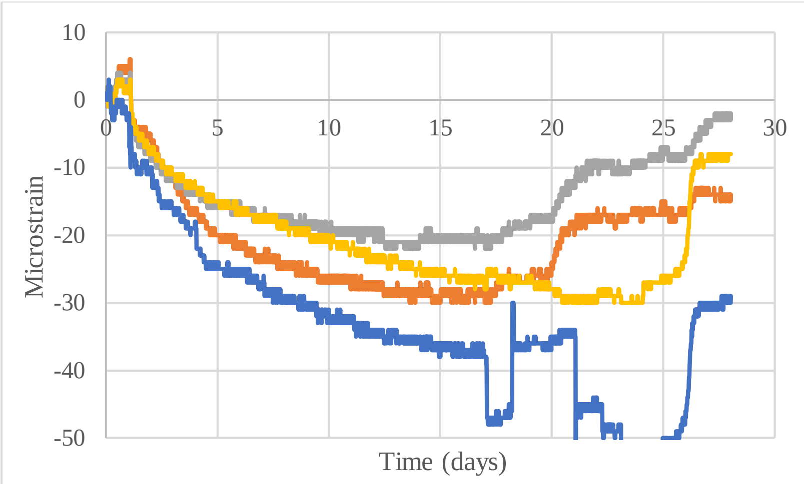


Figure B.30: ASTM C1581 cracking test mixture I1HF, specimen 1

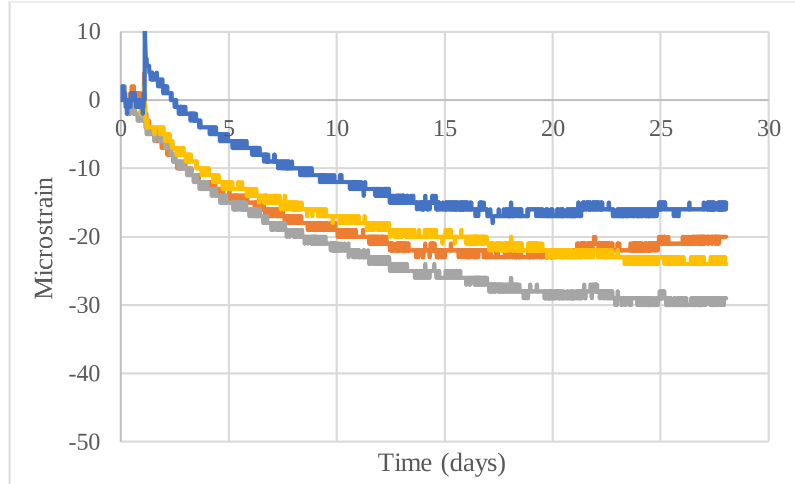


Figure B.31: ASTM C1581 cracking test mixture I1HF, specimen 2

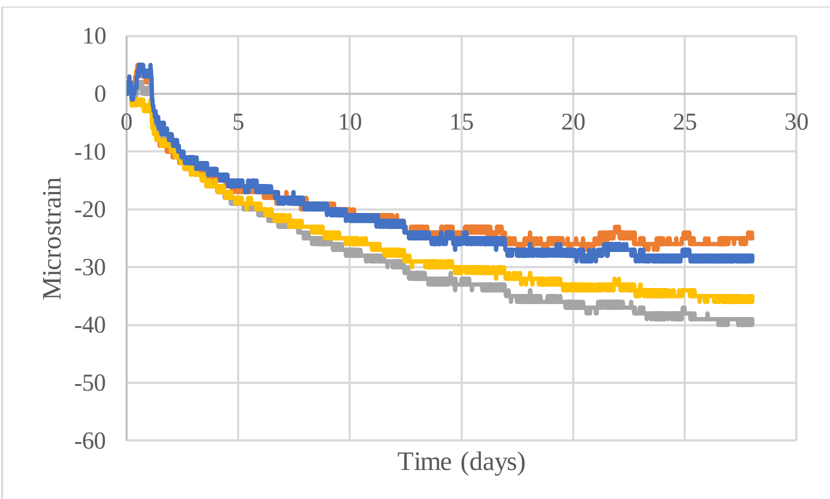


Figure B.32: ASTM C1581 cracking test mixture I1HF, specimen 3

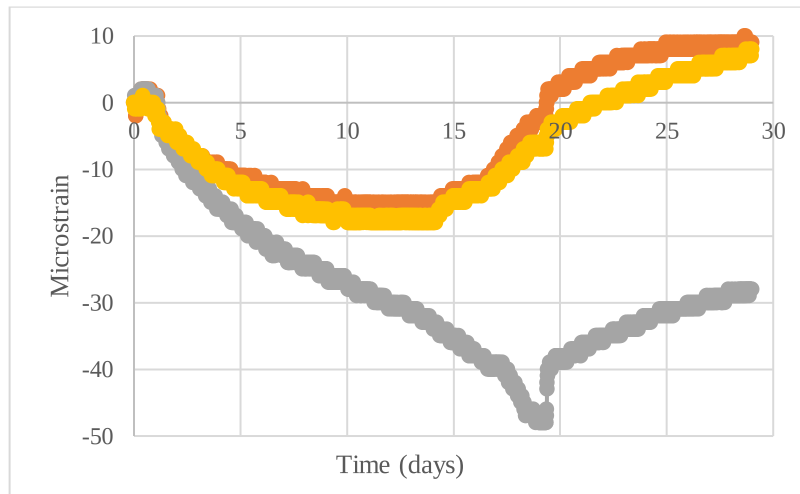


Figure B.33: ASTM C1581 cracking test mixture I2HF, specimen 1

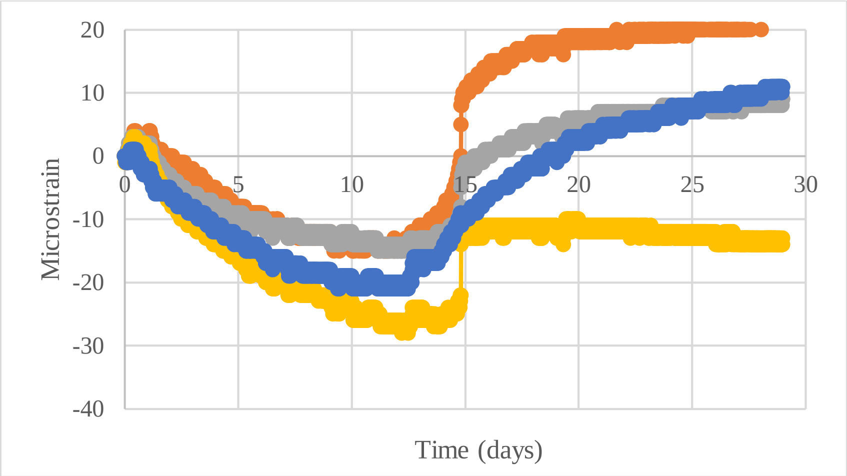


Figure B.34: ASTM C1581 cracking test mixture I2HF, specimen 2

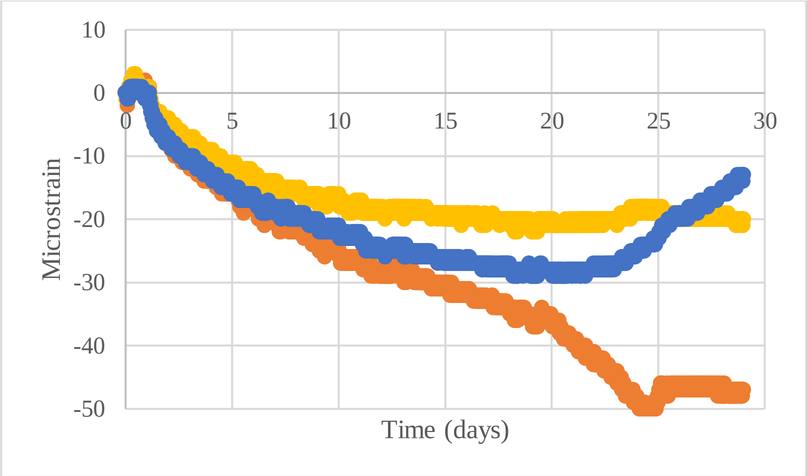


Figure B.35: ASTM C1581 cracking test mixture I2HF, specimen 3

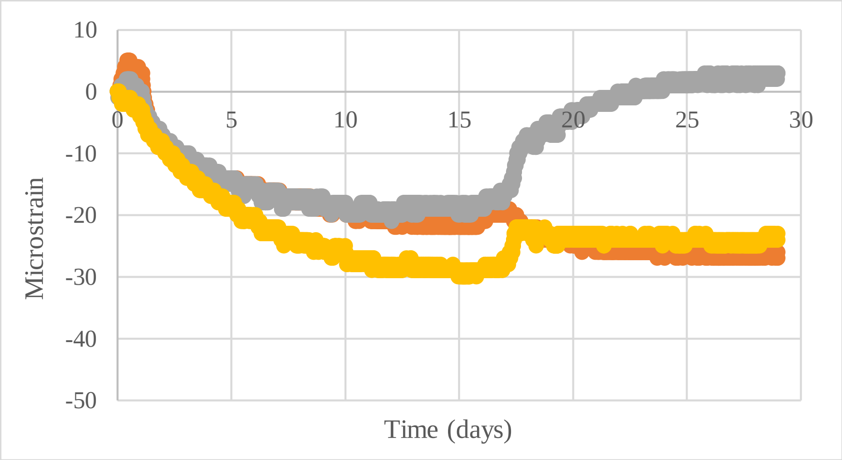


Figure B.36: ASTM C1581 cracking test mixture IP, specimen 1

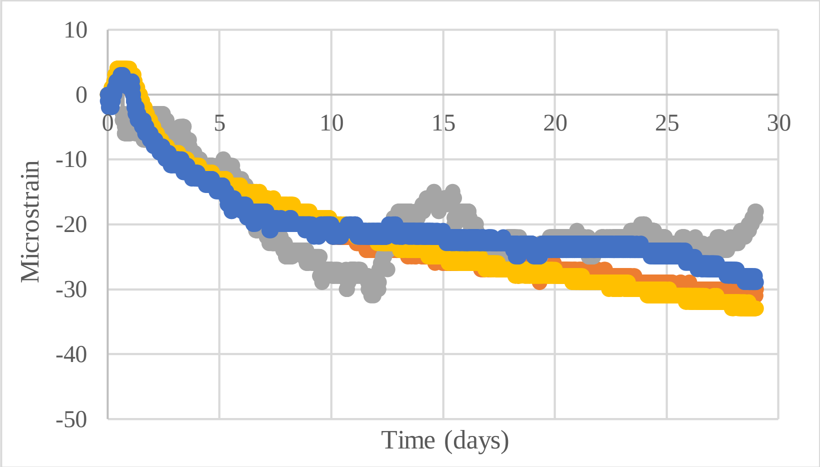


Figure B.37: ASTM C1581 cracking test mixture IP, specimen 2

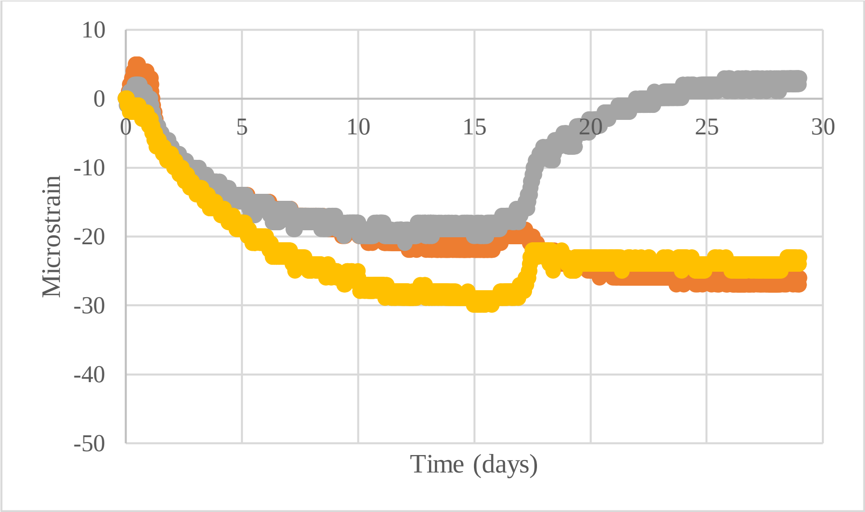


Figure B.38: ASTM C1581 cracking test mixture IP, specimen 3

Figure B.39: Autogenous shrinkage testing apparatus (from ASTM C1698)

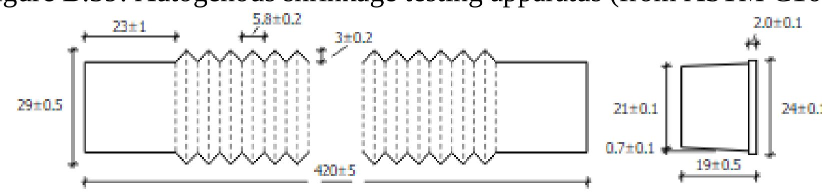


Figure B.40: Geometry of autogenous shrinkage specimens (from ASTM C1698)

Table B.14: Autogenous shrinkage test results

Mixture ID	Autogenous Strain (μm/m) 28 Day Average
CC	433.6
I1M	263.8
I2M	272.8
I1H	298.9
I2H	173.3
CF	384.7
I1MF	261.5
I2MF	227.1
I1HF	271.5
I2HF	185.2
CLMC	139.5
ILMC	70.6
CL	273.6
ILA	70.0
ILB	45.5
RSCL	193.3
RSIL	146.0
P.B.N.N.*	-
IP	-

APPENDIX C – PROJECT SPECIAL PROVISION USED FOR PILOT PROJECT

Internally Cured Concrete Field Study (SPECIAL)

1.0 Description and Overview

Internal curing concrete is a modified conventional concrete mixture in which prewetted lightweight fine aggregate is substituted for a portion of the standard fine aggregate to aid the curing process. Research has demonstrated that internal curing concrete using prewetted fine aggregates provides benefits including increased cement hydration, improved strength development, increased durability, reduced permeability and reduced shrinkage and cracking. UNC Charlotte is working with NCDOT to evaluate internally cured concrete mixtures using materials available for construction in North Carolina as part of NCDOT research project RP 2016-06, "Internally Cured Concrete Using Lightweight Aggregate." The bridge deck for the subject project is included in the research project as the demonstration project for field study.

The concrete deck for the subject bridge will be constructed using two concrete mixtures: one that is a conventional normal weight concrete mixture, and a second concrete mixture designed to provide internal curing. The deck in Stage I of the project (southbound lanes) will be constructed to include one span of conventional normal weight 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 1, below. If desired, the contractor could elect to switch the spans for placement of each type of mixture. However, alternate spans will be utilized for each mixture.

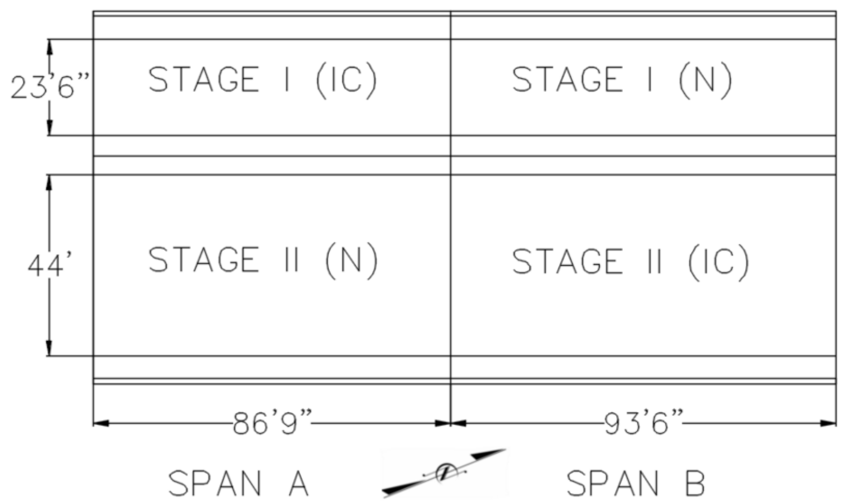


Figure 1: Schematic of bridge deck showing construction stages and denoting conventional normal weight (N) and internally cured (IC) concrete sections.

2.0 Materials, Mixture Proportioning, and Batching

The conventional normal weight concrete mixture for this project will be Class AA concrete meeting NCDOT Standard Specifications Section 1000-4. Cementitious materials utilized for the bridge deck should include fly ash using substitution rates allowable per NCDOT Specifications. The internally cured concrete mixture shall contain the same materials and mixture proportions utilized for the Class AA concrete mixture, but should be modified by substituting 30% lightweight fine aggregate (meeting AASHTO M195) by volume for the standard fine aggregate. Both mixtures should include air entraining admixture, water-reducing admixture, and other admixtures as required to meet NCDOT specifications for Class AA concrete and project specifications. Additional guidance for developing the internally cured concrete are provided in American Concrete Institute (ACI) 308-213 R-13. The UNC Charlotte research team is available to work with the concrete producer to assist in development of the mixtures, if requested.

Submit proposed concrete mix 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. The following information should be provided for both the conventional normal weight concrete and internal curing concrete mixtures:

- Cementitious materials content (lb/yd^3)
- Water content (lb/yd^3)
- Coarse aggregate content (lb/yd^3)
- Fine aggregate content (lb/yd^3) based on saturated surface dry (SSD) condition of aggregates
- Admixture dosages
- Compressive strength test results per NCDOT submittal requirements (psi)

During batching, corrections must be made to account for additional free (surface) moisture contained on the prewetted fine aggregate. Additional adjustments should be made to account for changes in the fine aggregate fineness modulus and aggregate moisture contents. The moisture content of the prewetted lightweight fine aggregate should be determined using New York State DOT test method NY 703-19E "Moisture Content of Lightweight Fine Aggregate." Determine the free moisture content of the prewetted lightweight fine aggregate immediately prior to batching, and adjust batch weights accordingly. Do not adjust the mix water for absorbed water in the prewetted fine aggregate, as it is retained within the fine aggregate and does not affect the mix water. The entrained air content may be tested per ASTM C231 using a Type B pressure meter, as the prewetted lightweight aggregate is saturated prior to use.

3.0 Stockpile Management for Prewetted Lightweight Fine Aggregate

At the concrete production facility, establish a stockpile area and moisture delivery system for the lightweight fine aggregate. Sprinklers have been successfully utilized to pre-wet stockpiles of fine aggregate for internal curing concrete. Maintain the stockpile so that delivery of adequate moisture to prewet the material is ensured, and the stockpile does not become contaminated with other materials. Contaminated aggregate must be discarded.

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 shall 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.

4.0 Placement

Placement of the conventional normal weight 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 lightweight aggregate pores. During placement, the contractor shall periodically (as requested) provide the UNC Charlotte research team a small sample (~2 cubic feet) for the purpose of field testing and preparing specimens for laboratory testing. Concrete required by UNC Charlotte personnel will be sampled from the concrete delivered for placement on the bridge deck. No additional sampling or testing for this research will be required for the closure pours or sidewalk.

Additionally, to facilitate the research objectives, one small test slab (22"x38"x8") will be cast by the contractor for each poured section. UNC Charlotte personnel will fabricate the formwork for the small slabs and transport the formwork to the site. The location of these specimens, as well as temporary storage of additional field-cured specimens (cylinders, beams, etc.) will be identified by the Contractor and approved by NCDOT prior to the start of construction. Finishing operations may be performed by hand, including burlap drag finish. Curing measures for the small slabs should be the same as those utilized for the bridge deck (wet cured for 7 days as required by specifications). Care shall be taken by the contractor to ensure that the placement and curing of the test slabs is as close as possible to methods used during construction of the bridge deck. It is anticipated that during the course of the project, no more than 3 CY of concrete would be requested by UNC Charlotte for samples or slabs.

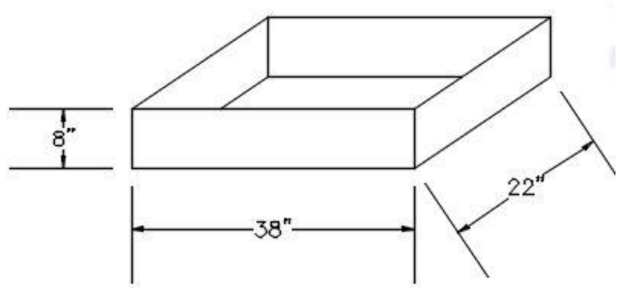


Figure 2: Small slab to be placed and finished by Contractor for each portion of concrete deck placement (four total).

5.0 <u>Instrumentation</u>

To evaluate the internal curing concrete mixture, the subject bridge deck will be instrumented with sensors to measure strain and internal moisture/humidity. Some of the sensors will be attached to the reinforcing steel, and concrete will be placed around the sensor. Other sensors may be installed after placement of the concrete deck. Instrumentation equipment shall be purchased and installed by the UNC Charlotte Department research team. At this time, the instrumentation required for measurement of strain has been identified. However, the specific sensors and installation protocol for measurement of internal moisture/humidity are still being evaluated by UNC Charlotte personnel. UNC Charlotte will provide further details on the instrumentation prior to the start of construction.

The Contractor will work with UNC Charlotte personnel to facilitate installation of the instrumentation equipment prior to and after placement of the concrete. The contractor shall ensure that reasonable care is taken by all personnel to avoid stepping on, striking, or otherwise damaging instrumentation, wiring, and other supplementary equipment during and after placement of the concrete. The contractor will assist the UNC Charlotte research team in identifying potential preventable hazards to instrumentation equipment, and implementation of solutions to ensure survivability of the sensors and other equipment.

Sensors installed prior to concrete placement will be used to measure strain in the bridge deck. UNC Charlotte personnel will tie these sensors (approximately 16 total) to the reinforcing steel prior to placement of the concrete in positions roughly indicated in Figure 3. Each gauge will have wires running from the gauge to an instrumentation box that will be mounted near the bridge abutment. The mounting location for the enclosure box will be determined by the project team and approved by NCDOT prior to construction.

To monitor internal moisture/humidity, other sensors will be installed in the bridge deck. Typically, these sensors can be installed five (5) days after concrete placement. For ease of installation of these sensors and to avoid drilling, greased dowels or other "plugs" will be prepared by UNC Charlotte and installed at the desired sensor location prior to placement of the concrete. After initial setting of the concrete, the plugs will be removed by UNC Charlotte, sensors shall be installed into the holes, and holes will be filled with repair materials by UNC Charlotte.

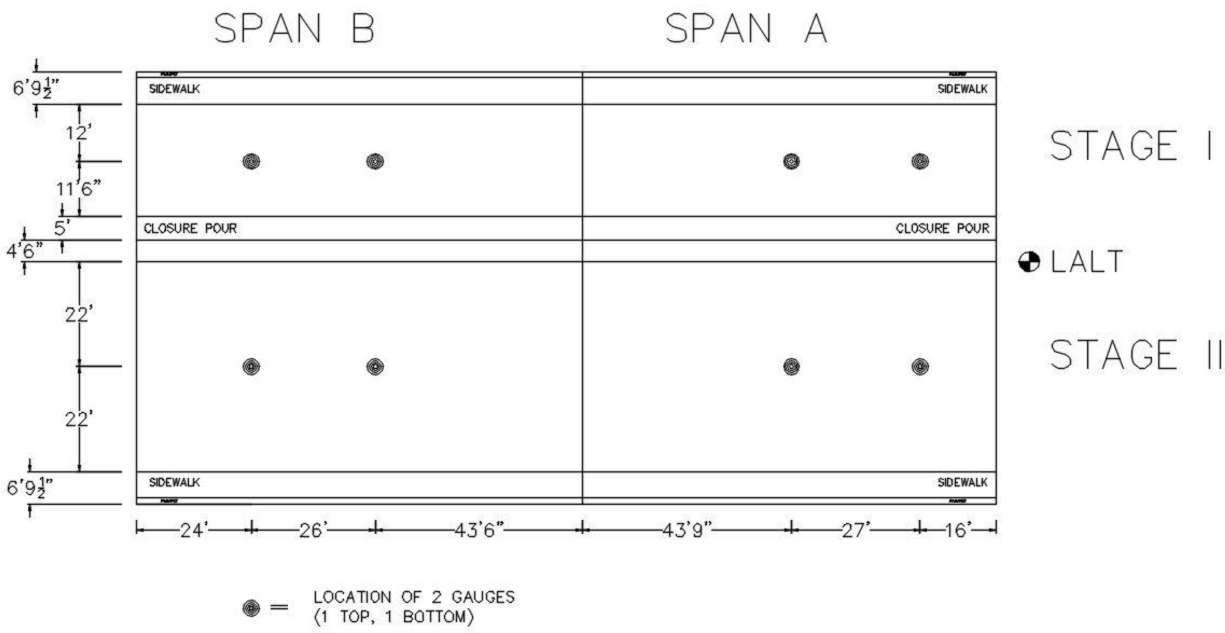


Figure 3: Approximate locations of sensors installed prior to concrete placement.

6.0 Access

The Contractor shall provide UNC Charlotte personnel with access to the bridge and concrete batch plant for the purposes of:

- monitoring the processes used to prepare aggregate stockpiles
- observing concrete batching and placement
- performing field testing such as slump, air content, and other tests
- preparing and curing test specimens to be returned to UNC Charlotte's laboratory
- preparing, curing, and testing small slab specimens in the vicinity of the bridge deck
- installing the instrumentation and the monitoring the equipment in the bridge deck (as described above)
- performing visual observations of the bridge deck to observe early age performance

Adequate notice of the anticipated start date of construction tasks shall be provided to UNC Charlotte, and the Contractor shall regularly update UNC Charlotte personnel on the project schedule in a timely manner. Correspondence with UNC Charlotte should be directed to the Research Project 2016-06 Principal Investigator, Tara Cavalline (704-201-3468, tcavalline@uncc.edu). Contact information for other UNC Charlotte personnel will be provided after the contract is awarded.

It is anticipated that visual observations of the bridge deck will be performed at 1 month, 2 months, 5 months, 8 months, and 1 year after placement. If on-site, the Contractor will work with UNC Charlotte personnel to ensure the visual survey can be performed. The Contractor will also ensure that UNC Charlotte personnel has access to the instrumentation box and field test specimens for the duration of construction at the site. Additionally, UNC Charlotte team requests input from the Contractor and concrete producer regarding their experience with internal curing concrete to supplement the research findings.

7.0 Measurement and Payment

The entire cost of this work with the exception of instrumentation and instrumentation installation shall be included in the unit contract price bid for Reinforced Concrete Deck Slab.

APPENDIX D - SUPPLEMENTAL INFORMATION FOR EVALUATION OF FIELD RESULTS (Chapter 4)

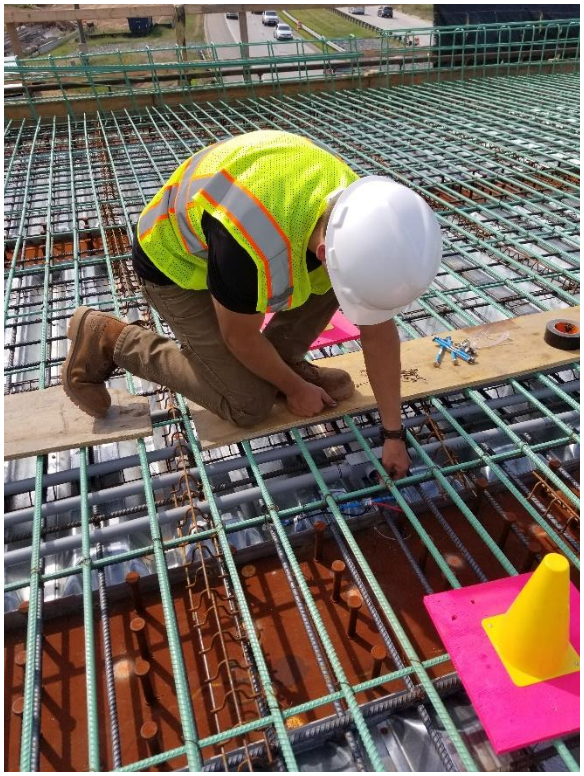


Figure D.1: Installing sensors in bridge deck

Figure D.2: Data acquisition equipment beneath deck

Figure D.3: Conduit installed within reinforcing steel of bridge deck (gray PVC). Lead wires to sensors are shown. Humidity sensor is shown at top (attached to black wire) and vibrating wire strain gage is shown at bottom (attached to red wire)

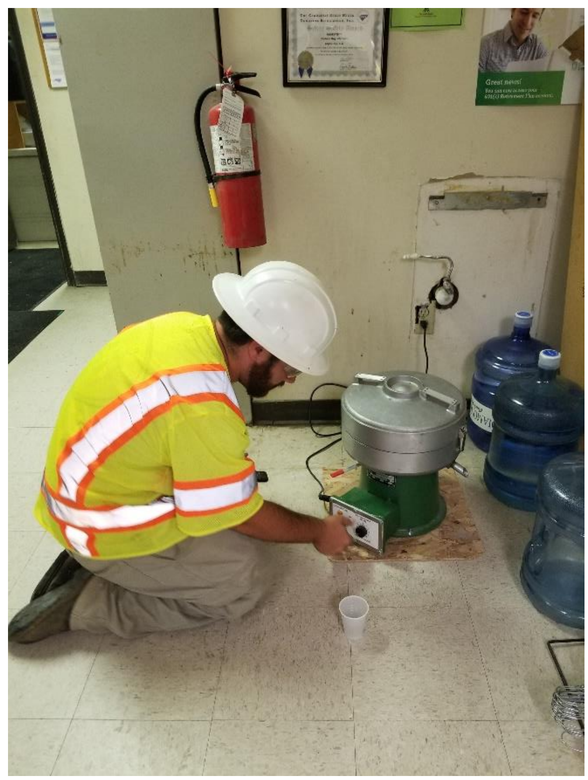
Figure D.4: Companion slabs prior to concrete placement. Note separate instrumentation system identical to the one installed on bridge deck. Companion slabs were cast on trailer for easy transport to UNC Charlotte after initial curing.

Figure D.5: Placement of control section of deck on July 26, 2017. Yellow cones marks location of two sets of sensors. Concrete placed prior to photo has encapsulated two other sets of sensors.

Figure D.6: Placement of concrete around sensors in one companion test slab (control mixture). Note data acquisition box serving both companion slabs.

Figure D.7: Stockpile of prewetted lightweight aggregate the day before placement of the internally cured bridge deck (Stage 1). Note location of sprinklers. Drainage of the stockpile was directed to the far side of the pile (slab sloped to drain).

Figure D.8: Preparing second companion slab form, sensors, and data acquisition system for placement of ICC mixture.



Figures D.9 and D.10: Moisture content testing at ready mix plant prior to batching ICC mixture, using paper towel method (left) and centrifuge method (right).

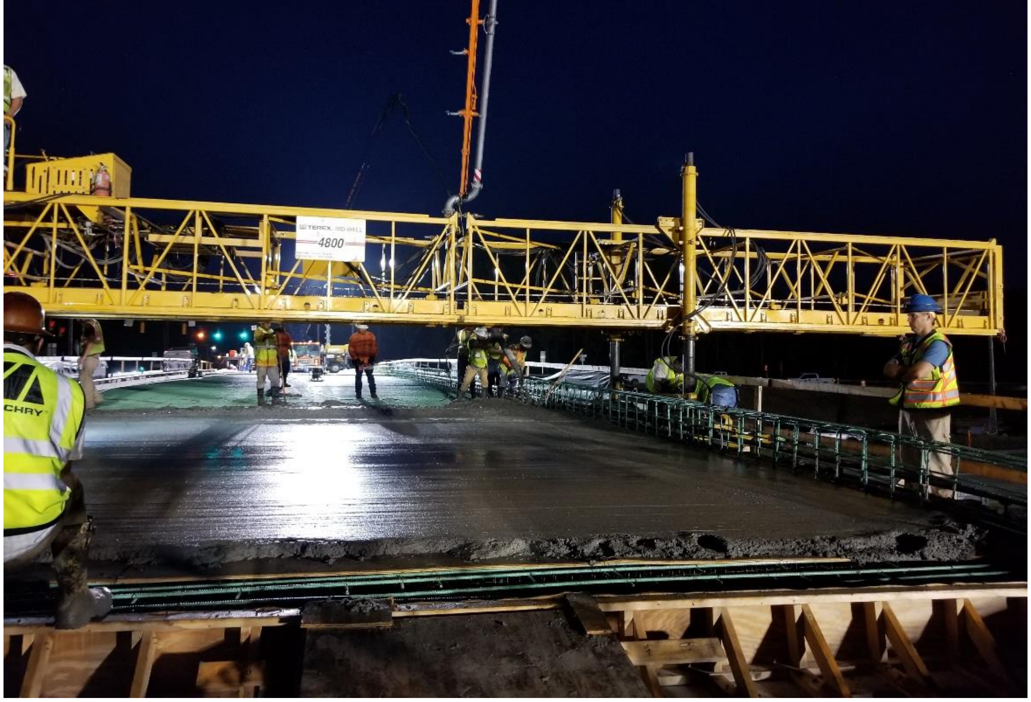


Figure D.11: Placement of ICC mixture at pilot project.

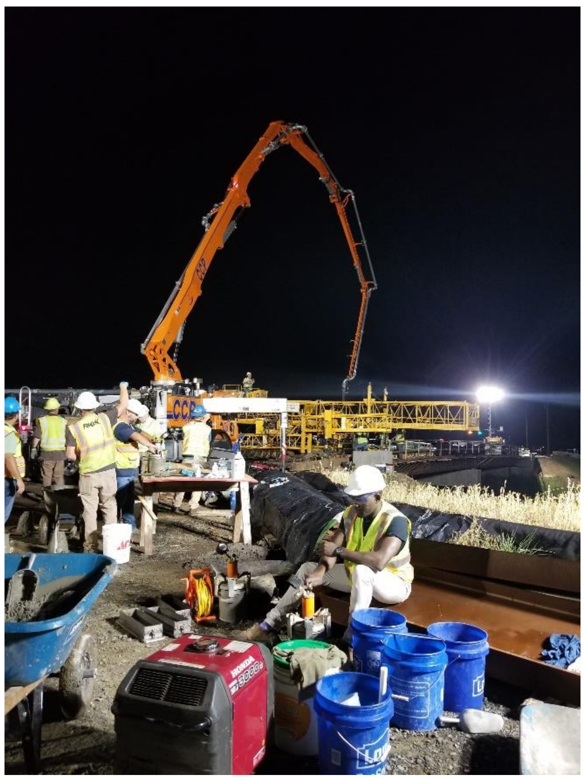
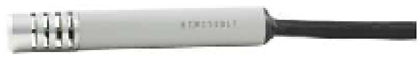


Figure D.12: Testing of fresh properties of ICC mixture using SAM while concrete is being placed on deck

APPENDIX E – SUPPORTING INFORMATION FOR FIELD INSTRUMENTATION AND DATA COLLECTION (CHAPTER 4)

measurement SPECIALTIES™

HTM2500LF – Temperature and Relative Humidity Module



√RoHS

- **Hermetic Housing**
- **Humidity calibrated within +/-2% @55%RH**
- **Temperature measurement through NTC 10kOhms +/-1% direct output**
- **Small size product**
- **Typical 1 to 4 Volt DC output for 0 to 100%RH at 5Vdc**

DESCRIPTION

Based on the rugged HTS2035SMD humidity / temperature sensor, HTM2500LF is a dedicated humidity and temperature transducer designed for OEM applications where a reliable and accurate measurement is needed. Direct interface with a micro-controller is made possible with the module's humidity linear voltage output.

FEATURES

- Full interchangeability
- High reliability and long term stability
- Not affected by water immersion
- Ratiometric to voltage supply
- Suitable for 3 to 10 Vdc supply voltage

Humidity Sensor Specific Features

- Instantaneous de-saturation after long periods in saturation phase
- Fast response time
- High resistance to chemicals
- Patented solid polymer structure

Temperature Sensor Specific Features

- Stable
- High sensitivity

APPLICATIONS

- Industrial
- Process control
- Hygrostat
- Data logger

...

HTM2500LF HPC169 Rev C | www.meas-spec.com 1/9 | October 2012

Figure E.1: Relative humidity sensors used for instrumented slabs

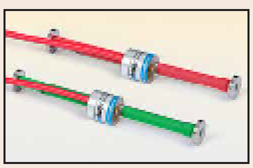
4200 Series

Concrete Embedment Strain Gages

Applications

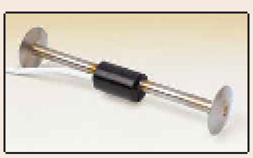
The Model 4200, 4202 and 4210 are designed to measure strains in or on…

- **Foundations**
- **Piles**
- **Bridges**
- **Dams**
- **Containment vessels**
- **Tunnel liners**
- **Mass concrete with coarse aggregates**
- **Laboratories and/or where space limitations exist (Model 4202)**



Model 4200ER (5,000 μstrain) and 4200ER (10,000 μstrain) Strain Gages.

Model 4200L (low modulus) Strain Gage.



Model 4200HT-T High Temperature Strain Gage.

Model 4202 (front), Model 4200 (center) and Model 4210 (rear) Concrete Embedment Strain Gages.

Operating Principle

The Model 4200 Series Vibrating Wire Embedment Strain Gages are designed for direct embedment in concrete. The Model 4200 (standard model) has a 153 mm gage length and is commonly used for strain measurements in foundations, piles, bridges, dams, containment vessels, tunnel liners, etc. The Model 4210 has a 250 mm gage length and is designed for use in mass concrete with coarse aggregates. It is extra rugged to resist bending and has large flanges to provide greater engagement area. The 4202 and 4204 (51 mm and 102 mm gage lengths, respectively) are designed for laboratory use and/or where there are space limitations.

Strains are measured using the vibrating wire principle: a length of steel wire is tensioned between two end blocks that are embedded directly in concrete. Deformations (i.e. strain changes) of the concrete mass, will cause the two end blocks to move relative to one another, thus altering the tension in the steel wire. The tension is measured by plucking the wire and measuring its resonant frequency of vibration using an electromagnetic coil.

Advantages and Limitations

The Model 4200 Series Strain Gages enjoy all the advantages of vibrating wire sensors, which includes excellent long term stability, maximum resistance to the effects of water, and a frequency output suitable for transmission over very long cables.

All components are made from stainless steel for corrosion protection and the gages are fully waterproof. The Model 4210 is very rugged and designed to withstand the rigors of concrete placement.

Each gage incorporates a thermistor so that the temperature can be read and displayed by the readout.

Extended range, low modulus and high temperature versions are also available. The Model 4200ER is designed for measuring large strains up to 10,000 μstrain. The Model 4200L (low modulus version) is particularily suitable for measuring curing strains in concrete. The Model 4200HT is designed for short-term use at temperatures up to 200°C while the Model 4200HT-T is designed for long-term use at temperatures up to 220°C, making it particularly suitable for installation in steam-cured spun concrete piles.

Geotechnical and Structural **Instrumentation**

Figure E.2: Vibrating wire strain gages used in instrumented slabs

APPENDIX F - DOCUMENTATION OF DISTRESS AT PILOT PROJECT STAGE 1 – APRIL 13, 2018

Crack #	Area Photo	Detail Photo	Location and Description
1			Width: 0.005 inches Location: 32 ft 10 in from Northwest corner (Internally Cured)
2			Width: 0.005 inches Location: 63 ft 2 in from Northwest corner (Internally Cured)
3			Width: 0.007 inches Location: 91 ft 11 in from Northwest corner (Internally Cured)

4			Width: 0.010 inches Location: 25 ft 0 in from Southeast corner (Conventional concrete)
5			Width: 0.005 inches Location: 119 ft 11 in from Northwest corner (Conventional concrete)
6			Width: 0.005 inches Location: 68 ft 6 in from Southeast corner (Conventional concrete)

7			Width: 0.005 inches Location: 69 ft 6 in from Southeast corner (Conventional concrete)

APPENDIX G – SPECIAL PROVISION FOR INTERNALLY CURED CONCRETE FOR CLASS AA CONCRETE

Internally Cured Concrete (SPECIAL)

1.0 Description and Overview

Internally cured concrete is a modified conventional concrete mixture in which prewetted lightweight fine aggregate is substituted for a portion of the standard fine aggregate enhance the hydration of cementitious materials. Research has demonstrated that internally cured concrete using prewetted fine aggregates provides benefits including increased cement hydration, reduced permeability, and reduced shrinkage and cracking.

2.0 Materials, Mixture Proportioning, and Batching

Cementitious materials utilized should include fly ash using substitution rates allowable per NCDOT Specifications. The internally cured concrete mixture shall contain the same materials and mixture proportions utilized for the Class AA concrete mixture in accordance with Section 1000-4, but should be modified by substituting lightweight fine aggregate meeting AASHTO M195 by volume for a portion of the standard fine aggregate. The amount of lightweight fine aggregate used in the mixture should be determined using mixture proportioning guidance provided in ACI (308-123)R-13 and ASTM C1761/C1761M-17. The internally cured concrete mixtures should include air entraining admixture, water-reducing admixture, and other admixtures as required to meet NCDOT specifications for Class AA concrete and project specifications. Additional guidance is provided in ACI (308-213)R-13.

Submit proposed concrete mix 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. The following information should be provided for the internal curing concrete mixtures:

- Cementitious materials content (lb/yd^3)
- Water content (lb/yd^3)
- Coarse aggregate content (lb/yd^3)
- Fine aggregate content (lb/yd^3) based on saturated surface dry (SSD) condition of aggregates
- Admixture dosages
- Compressive strength test results per NCDOT submittal requirements (psi)

During batching, corrections must be made to account for additional free (surface) moisture contained on the prewetted fine aggregate. Additional adjustments should be made to account for changes in the fine aggregate fineness modulus and aggregate moisture contents. The moisture content of the prewetted lightweight fine aggregate should be determined using ASTM C1761/C1761M-17 or the centrifuge method as described in Indiana DOT Test Method ITM 222-15T, “Specific Gravity Factor and Absorption of Lightweight Fine Aggregate.” Determine the free moisture content of the prewetted lightweight fine aggregate immediately prior to batching, and adjust batch weights accordingly. Do not adjust the mix water for absorbed water in the prewetted fine aggregate, as it is retained within the fine aggregate and does not affect the mix water. The entrained air content may be tested per ASTM C231 using a Type B pressure meter, as the prewetted lightweight aggregate is saturated prior to use.

3.0 Stockpile Management for Prewetted Lightweight Fine Aggregate

At the concrete production facility, establish a stockpile area and moisture delivery system for the lightweight fine aggregate. Sprinklers have been successfully utilized to pre-wet stockpiles of fine aggregate for internal curing concrete. Maintain the stockpile so that delivery of adequate moisture to prewet the material is ensured, and the stockpile does not become contaminated with other materials. Contaminated aggregate must be discarded.

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 shall 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 to 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.

4.0 Placement

Placement of the conventional normal weight 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 lightweight aggregate pores.