MITIGATION OF EARLY AGE CRACKING OF CONCRETE FOR BRIDGE DECKS

BY

WILLIAM OLIVER WILSON

THESIS
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Adviser:

Professor David A. Lange
ABSTRACT

In order to reduce reconstruction time and costs in order to better serve the general public a need has arisen for longer service life when considering concrete bridge decks for highway applications. Service life of concrete bridge decks is often cut short indirectly, due to deterioration mechanisms that exist at early ages of the concrete. Early age deterioration mechanisms such as early age cracking cause the reinforcing steel rebar to be exposed prematurely to the elements of the environment, which causes for lower service life of the bridge deck through corrosion of the reinforcing steel in the bridge deck. In harsh environments, such as the Midwestern United States, the premature exposure can cause devastating effects to service life. In order to prevent early deterioration, it is imperative to examine early age hardened properties of the concrete and fresh properties of the concrete in order to produce the best defect-free bridge deck available which will promote long service life.

In order to examine early age hardened properties and fresh properties of concrete to be used for bridge decks on the Illinois Tollway, laboratory testing was performed by CTL Group and the University of Illinois at Urbana-Champaign. Through extensive laboratory testing, results indicate that many mitigation strategies are able to improve early age cracking in concrete for bridge decks. Through early age mitigation it is possible to decrease the probability of early age deterioration in concrete bridge decks and promote service life.
Acknowledgements

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Thank you to Professor John Popovics for extending an opportunity for a position as a teaching assistant for CEE 300. Without this opportunity I would have never been able to attend UIUC and therefore am forever thankful.
Thank you to all my friends that I have made along the way. There are too many to list here, but I know that my time spent here would not have been nearly as manageable without all of you.

I would like to thank my family as well, Ben, Mom, Dad, Grandpa, and Samantha. Without your constant support I would have never been able to accomplish everything I have done. I will always be grateful to have all of you in my life.

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1. Introduction

Increasing service life and durability is a main focus in the civil engineering industry today. No longer do we only seek to build bigger and stronger, we as a civil engineering industry are trying to build “longer” as well. With the interest in a longer service life, engineers are looking at new and interesting ways to outperform the infrastructure of yesteryear. In order to outperform existing infrastructure, engineers are forced to examine new properties and parameters when considering performance of all parts of the civil engineer structure, including concrete.

When considering concrete one must consider how the concrete is able to protect the reinforcing steel inside a reinforced concrete structure, such as a concrete bridge deck. It is well known in the literature and the community that one of the main deterioration mechanisms in a reinforced concrete structure is corrosion of the reinforcing steel; therefore it is imperative to prevent early corrosion of the reinforcing steel (Van Dam, D'Ambrosia, & Birch, 2012).

1.1 Background

The Illinois State Toll Highway Authority (Tollway) has put in motion a $12B renovation and reconstruction project that is to take place over a span of fifteen years. This project includes the construction and reconstruction of over one hundred bridge decks in the greater Chicago-land area (Van Dam, D'Ambrosia, & Birch, 2012). The objective is to provide an improved tolled roadway in and around the greater Chicagoland area that will be able to service the cities needs for many years to come. The Tollway is expecting a long life span for the bridge decks that are being constructed and reconstructed with a projected service life of 75-100 years (Van Dam, D'Ambrosia, & Birch, 2012). Due to the ambitious goals related to service life of the steel
reinforced concrete bridge decks, the Tollway requires an improved performance based bridge
dock specification for their concrete used to constructed bridge decks. To provide an improved
performance based bridge deck specification, the Tollway sought out materials consulting to
provide research information to address deterioration concerns commonly seen in concrete
bridge deck materials. The Tollway approached CTL Group (CTL) for help in determining the
testing needed in order to achieve the need for a longer service life of their concrete bridge decks
and to write an improved bridge deck specification that addresses performance based testing for
the concrete mixtures to be used. Through this project the University of Illinois at Urbana-
Champaign (UIUC) was able to collaborate with CTL to provide research testing for the Tollway
that is capable of better understanding the desired performance aspects of several different early
age cracking mitigation strategies commonly used in concrete materials today.

1.2 Deterioration Mechanisms

Early age cracking is a leading threat to the long term service life of concrete materials. Early age
 cracking is generally defined as cracking in concrete that occurs within the first several
days after placement (Bentur, 2002). Early age cracking is seen to reduce service life
tremendously due to the fact that cracks in a young concrete material reduce the concrete’s
ability to resist both water and chloride penetration. When chlorides are allowed to penetrate
into a concrete material to the reinforcing steel of the concrete bridge deck the service life is
dropped due to corrosion of the reinforcing steel inside the concrete matrix. When concrete is
initially placed, it provides a high pH passive layer around the reinforcing steel which protects
the steel from corrosion. If chlorides are allowed to penetrate to the reinforcing steel the chemistry
of the protective concrete around the reinforcing steel changes which allows for
deterioration of the reinforcing steel of the bridge deck to initiate (Mindess, Young, & Darwin, 2003). The deterioration of the reinforcing steel is known commonly as corrosion.

1.2.1 Corrosion

Corrosion is a major problem in concrete bridge decks for two reasons. The first reason is that the products created by corrosion of steel are expansive, which causes a tensile stress in the concrete surrounding the steel. The concrete surrounding the reinforcing steel is commonly known to be weak in tension; therefore the concrete begins to crack and break away from the steel reinforcement. Concrete cracking and break down around the steel reinforcement causes higher exposure of the steel reinforcement which causes for a higher amount of deterioration to occur in the steel (Emmons, 1993). The second reason corrosion is a problem when considering bridge decks is that when steel corrodes it no longer is able to hold the amount of tensile force initially designed in the bridge decks; meaning that a loss of steel section causes for a loss of capacity in the bridge deck which can lead to unsafe conditions for the users of the bridge deck in question. Because of these two reasons, it is imperative to increase the time before the onset of corrosion in concrete bridge decks if a greater service life is desired.

1.2.2 Early Age Cracking

Early age cracking is best understood when considering two competing mechanisms—early age shrinkage and creep. Through the understanding of both shrinkage and creep, one is able to understand what causes a restrained concrete material to crack at early ages (Altoubat & Lange, 2001). The general way to think about a restrained concrete is to know that concrete will shrink chemically, known as autogenous shrinkage, and through drying shrinkage at early ages. When the concrete shrinks without restraint, the concrete is free to reduce in length and no stress is induced into the material. Differently when concrete is restrained, for instance in a concrete
bridge deck, the concrete is not allowed to reduce in length and a stress is induced inside the concrete matrix. When a stress is induced inside the concrete matrix, the concrete matrix diminishes the stress through a process known as creep. Creep is defined as a process where a material with a constant load will reduce this load through slow deformation of the material. Initially there is an elastic component to the creep followed by a slow time-dependent deformation. If the shrinkage in the restrained concrete matrix is minimal, a smaller stress will be induced in the concrete matrix and the concrete will be able to account for the stress through the deformation process of creep. Therefore if the concrete exhibits a high enough creep capacity the concrete will “relax away” the stress produced by shrinkage and will avoid early age cracking.

1.2.2.1 Early Age Shrinkage

Concrete shrinks due to loss of moisture externally and internally. Moisture is lost externally due to an imbalance of external relative humidity and internal relative humidity of the concrete matrix. Moisture loss causes capillary stress inside the concrete matrix which leads to shrinkage in the material. Capillary stress is described below in Equation 1 where $P_{\text{cap}}$ is the hydrostatic tension of the water in the capillary pore, $\gamma$ is the surface tension of the water, and $r$ is the radius of the water meniscus inside the pore.

$$P_{\text{cap}} = \frac{2\gamma}{r} \quad (1)$$

When water evaporates the radius of the meniscus reduces and causes pressure in the capillary pores. The pressure brings the walls of the pores closer together which in turn induces shrinkage. The phenomenon described is commonly known as drying shrinkage (Mindess, Young, & Darwin, 2003).
Autogenous shrinkage is shrinkage that occurs naturally inside concrete under sealed conditions. In other words, autogenous shrinkage occurs without outside influence of temperature or relative humidity. Autogenous shrinkage can be described best as occurring due to self-desiccation of the cement paste which is caused by loss of water inside the cement paste matrix due to cement hydration. Along with the mechanism for capillary stress discussed before, disjoining pressure is present during self-desiccation of the cement paste. Disjoining pressures are caused by water being used in the cement hydration process in order to create calcium silicate hydrate, CSH. When disjoining pressures are reduced the pores in the cement paste are drawn together, attributing to shrinkage (Mindess, Young, & Darwin, 2003).

1.2.2.2 Creep

Creep is a phenomenon that occurs when a load is applied to a material that causes displacement over time. The displacement overtime is known as creep. Creep is generally found through the relationship shown in the equation below in Equation 2.

\[ \varepsilon_{cr} = C \times sinh \left( \frac{V \sigma}{RT} \right) \]  

(2)

In the equation, \( \varepsilon_{cr} \) represents creep strain, \( C \) is a constant, \( V \) is the activation volume, \( R \) is the gas constant, \( T \) is absolute temperature, and \( \sigma \) is the applied stress (Mindess, Young, & Darwin, 2003). \( \varepsilon_{cr} \) is increased when stress increases in the specimen. Creep is often represented using specific creep and creep coefficient. The equation for specific creep is displayed below in Equation 3 (Mindess, Young, & Darwin, 2003).

\[ \text{specific creep} = \frac{\varepsilon_{cr}}{\sigma} \]  

(3)

Creep coefficient is represented by Equation 4 below, where \( \varepsilon_c \) represents the instantaneous strain upper the applied load (Mindess, Young, & Darwin, 2003).
Creep reduces stress in a concrete because the energy from stress is displaced due to the deformation of the material. Creep can be helpful when considering early age shrinkage properties in a restrained concrete specimen. Creep has the ability to reduce the amount of stress due to shrinkage in restrained specimens to levels that the concrete is able to manage before cracking occurs at early ages. When considering restrained concrete, it is crucial to understand that a concrete must be able to respond to the stress due to shrinkage through creep in order to prevent cracking.

1.2.3 Chloride Penetration

The two forms of chloride penetration examined in this study are diffusion of chlorides through the concrete matrix and early age cracking of the concrete material. Chlorides will naturally penetrate through a concrete matrix due to the porous nature of concrete materials. Concrete is known to have a highly interconnected pore structure that allows moisture movement into and out of the concrete matrix. Since water is allowed to penetrate into the concrete through the concrete’s pore structure, chlorides in the water are allowed to penetrate into the concrete as well. When the chlorides are able to penetrate deep enough into the concrete material, to the reinforcing steel of the concrete bridge deck, they are able to change the chemical make-up, drop the local pH, around the reinforcing steel and allow corrosion as stated before. The process described is shown below in the Figure 1.
Figure 1: Image showing the process of chloride penetration in concrete (Emmons, 1993).

Generally chloride penetration is known to be a slow process for high performance concretes due to lower water to cementitious ratios, w/cm, and use of supplementary cementitious materials, SCMs. Reducing the w/cm causes for a denser concrete microstructure which reduces the capillary porosity of the concrete matrix. Capillary porosity is known as the pores left in a concrete material due to excess water used in the production of concrete. SCMs provide a more efficient use of the cement product known as calcium hydroxide, CH, which is a product of the water and cement chemical reaction that does not provide an efficient strength to the concrete
mixture and is leachable, meaning that it can easily be removed from the concrete matrix through water permeability. SCMs are able to combine with the CH to provide CSH which is the desired product in the water and cement chemical reaction. CSH provides a much stronger bond at the microstructural level compared to CH and is non-leachable (Taylor, 1997). Knowing that CSH provides a denser, less permeable concrete allows the idea that having a higher percentage of CSH will provide a higher performing concrete material when considering durability and service life. Due to this knowledge, SCMs are commonly used today to provide high performance concrete for many applications, including concrete bridge decks.

Due to the slow nature of chloride penetration in concrete materials, the major deterioration mechanism examined in this study is that of early age cracking. Early age cracking in many cases causes for direct penetration to the reinforcing steel which greatly reduces service life in concrete bridge decks, especially in areas of the United States that experience harsh winters that require a high use of deicing salts to be used on the roadways throughout the winter. Figure 2 below shows how dramatic cracking can be when considering early age deterioration.
Due to the location of the Chicagoland area in the United States, high amounts of deicing salts are used during the winter which exacerbates the problem of chloride penetration. With this information it is seen that early age cracking must be mitigated when considering concrete bridge decks in the Midwestern United States.
1.3 Early Age Cracking Mitigation Strategies

Several early age cracking mitigation strategies exist in the literature and in practice due to a need for the reduction of early age cracking. The mitigation strategies examined in this study include the use of saturated lightweight aggregates for internal curing, shrinkage reducing admixtures, and using an optimized aggregate gradation.

1.3.1 Internal Curing

Saturated lightweight aggregates, SLA, are beginning to be used in the industry as a way to reduce early age shrinkage in concrete mixtures through a process known as internal curing. Internal curing is an idea that requires a highly absorptive aggregate to be used inside a concrete matrix to more efficiently disperse the mix water in the concrete matrix. Internal curing works by starting with an oven dry highly absorptive aggregate, commonly known as a lightweight aggregate, LWA, which is soaked in water for a given amount of time, generally three days, in order to absorb water that will be used after set in the concrete. The water inside the aggregate that contributes to the concrete matrix is known as free water (Henkensiefken, Bentz, Nantung, & Weiss, 2009). The free water is allowed to freely move in and out of the aggregate after setting of the concrete. Inside the concrete matrix upon set the internal relative humidity, RH, of the concrete drops as the concrete begins to dry. When the relative humidity drops below a threshold value that is dictated by the desorption characteristics of the LWA used, the free water leaves the LWA and is used by the unhydrated cement particles to further hydration of the concrete matrix (Henkensiefken, Bentz, Nantung, & Weiss, 2009). The continuation of the hydration process allows for a more efficient use of cement in the concrete system and more importantly reduces the effects of early age shrinkage. With a denser concrete microstructure, higher early age local strengths are seen in the concrete matrix which is able to diminish the
forces that cause shrinkage in the concrete material. Self-desiccation is also reduced due to the higher abundance of usable water inside the cement paste.

1.3.2 Shrinkage Reducing Admixtures

Shrinkage reducing admixtures, SRA, are used to chemically reduce surface tension of water at the liquid solid interface in the cement matrix of concrete. SRAs are able to reduce surface tension of water due to the use of hexylene glycol. The dosage rates for SRA in concrete are up to ten percent, with dosages above ten percent having no further effects on reducing shrinkage in concrete (Eclipse 4500).

Research has established that SRAs can reduce both drying and autogenous shrinkage. Since both of these shrinkage mechanisms occur due to surface tension of water inside the pores of concrete, it is reasonable that a reduction of the surface tension of the pore water would reduce shrinkage of these kinds. From the literature it is also found that SRA concrete mixtures have a much higher crack resistance in the restrained ring test (Bentur, 2002). Since the early age shrinkage is reduced, the concrete material develops much lower stresses at earlier ages. The reduction in stress at early ages allows the concrete to achieve a higher degree of hydration before higher levels of stress develop which in turn allows for a higher amount of crack resistance.

1.3.3 Optimized Gradation

Optimizing the gradation of the aggregates in a concrete mixture is an effective and inexpensive way to improve the concrete as a whole, including early age shrinkage properties. In recent years concrete plants are beginning to provide additional aggregate bins to allow for better graduations to be used in the concrete; therefore the aggregates are available at the plants for use in optimizing gradation for a mix, making for an inexpensive solution (Van Dam,
D'Ambrosia, & Birch, 2012). An optimized concrete aggregate gradation causes for better particle packing inside the concrete matrix which in turn allows for a more efficient use of the cement paste inside the concrete matrix, allowing for a workable mix with less cement paste. As shown below in Table 1, the optimized mix uses the least amount of cementitious materials compared to all other mixes tested. Since the cement paste portion of the concrete mix is lessened the shrinkage is also lessen since the cement paste is the portion of the concrete that shrinks.

1.4 Concrete Mixture Proportions

The concrete mixes examined throughout testing are presented in Table 1. The mixes include a basic concrete bridge deck mix that follows the current specification from the Tollway, BS, an optimized gradation mix, OPT, a saturated lightweight aggregate mix, SLA, a shrinkage reducing admixture mix, SRA, and a combined approach using all three mitigation strategies, ULT. Two different LWA's are used in testing and denoted by SLA #1 and SLA #2. SLA #1 uses an expanded slag aggregate for internal curing while SLA #2 uses an expanded shale aggregate.

Table 1: Concrete mixes examined throughout testing in lbs/yd$^3$

<table>
<thead>
<tr>
<th>Mix ID:</th>
<th>BS</th>
<th>OPT</th>
<th>SLA#1</th>
<th>SRA</th>
<th>ULT</th>
<th>SLA#2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement</td>
<td>515</td>
<td>375</td>
<td>409</td>
<td>403</td>
<td>313</td>
<td>409</td>
</tr>
<tr>
<td>Fly ash</td>
<td>0</td>
<td>125</td>
<td>0</td>
<td>134</td>
<td>111</td>
<td>0</td>
</tr>
<tr>
<td>Ground slag</td>
<td>110</td>
<td>0</td>
<td>136</td>
<td>0</td>
<td>154</td>
<td>136</td>
</tr>
<tr>
<td>Coarse aggregate: CM-11</td>
<td>1875</td>
<td>1501</td>
<td>1714</td>
<td>1840</td>
<td>1245</td>
<td>1714</td>
</tr>
<tr>
<td>3/8 chip: CM-16</td>
<td>0</td>
<td>391</td>
<td>0</td>
<td>0</td>
<td>325</td>
<td>0</td>
</tr>
<tr>
<td>Fine aggregate: FM-02</td>
<td>1160</td>
<td>1370</td>
<td>986</td>
<td>1323</td>
<td>1039</td>
<td>986</td>
</tr>
<tr>
<td>Expanded shale</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>236</td>
<td>258</td>
</tr>
<tr>
<td>Expanded slag</td>
<td>0</td>
<td>0</td>
<td>364</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Water</td>
<td>263</td>
<td>210</td>
<td>237</td>
<td>226</td>
<td>220</td>
<td>237</td>
</tr>
<tr>
<td>Water to cementitious ratio</td>
<td>0.43</td>
<td>0.43</td>
<td>0.35</td>
<td>0.43</td>
<td>0.39</td>
<td>0.43</td>
</tr>
</tbody>
</table>
Table 1 shows mixture proportions in pounds per cubic yard of concrete including w/cm. Chemical admixtures and fresh properties are shown below in Table 2. Admixtures are displayed in units of fluid ounces per cubic yard.

Table 2: Admixtures used in concrete mixes in fl oz/yd³

<table>
<thead>
<tr>
<th>Mix ID:</th>
<th>BS</th>
<th>OPT</th>
<th>SLA#1</th>
<th>SRA</th>
<th>ULT</th>
<th>SLA#2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chemical Admixture</td>
<td>1.6</td>
<td>2.6</td>
<td>2.6</td>
<td>5.0</td>
<td>3.8</td>
<td>2.6</td>
</tr>
<tr>
<td>Air entrainer</td>
<td>18.8</td>
<td>15.0</td>
<td>16.4</td>
<td>16.1</td>
<td>17.3</td>
<td>16.4</td>
</tr>
<tr>
<td>Mid-range water reducer</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>192.6</td>
<td>191.9</td>
<td>0.0</td>
</tr>
<tr>
<td>Shrinkage reducing admixture</td>
<td>0.0</td>
<td>8.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Slump retention</td>
<td>18.8</td>
<td>15.0</td>
<td>16.4</td>
<td>16.1</td>
<td>17.3</td>
<td>16.4</td>
</tr>
<tr>
<td>Hydration stabilizer</td>
<td>39.6</td>
<td>25.2</td>
<td>29.3</td>
<td>20.4</td>
<td>29.3</td>
<td>29.4</td>
</tr>
<tr>
<td>Superplastisizer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fresh Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump, in.</td>
</tr>
<tr>
<td>Air content</td>
</tr>
<tr>
<td>Temperature, °F</td>
</tr>
<tr>
<td>Fresh Density, lb/ft³</td>
</tr>
</tbody>
</table>
2. Constructability

2.1 Background

Constructability is a parameter that greatly influences the use of concrete technology in the civil engineering field. Generally if a concrete mix has poor constructability it has a much greater chance to perform poorly in practice due to errors in construction. Since constructability can greatly affect the performance and service life of a concrete bridge deck, this issue became one of the major focuses of the project. One believes that constructability should be a major focus of any concrete construction project since uniformity in the placement of a concrete structure greatly affects the ability for the structure to withstand the environmental elements that cause deterioration of a structure.

2.2 Procedure

Constructability is examined by performing a modified version of the fresh property tests for concrete detailed in ASTM C192. The modification to the standard testing included performing standard air content and slump tests for one hour after standard mixing of the concrete material. The standard slump and air content test set-ups are shown below in Figures 3 and 4.
The standard mixing procedure used required one minute of mixing of the fine and coarse concrete aggregates with some of the batch water and air entrainer followed by three minutes of mixing with all contents of the mix, three minutes of rest, and two final minutes of mixing after
the rest period. The batch size of the concrete mixes studied are less than two cubic feet of material which causes for a need to remix the tested concrete material in the mixer between each air content and slump test. The remixing is limited to two revolutions of the concrete mixers since a Lancaster pan mixer capable of good shearing action is used for the mixing of the lab concrete material. The reason for limiting the remixing time is to limit the amount of shearing on the material which causes for air content to effectively be beaten out of the material which greatly affects air content and slump test results. The idea for this type of testing is to roughly simulate the amount of shearing that a concrete mixture would experience in a concrete truck during the placement of a bridge deck. Since the pan mixer used has a higher amount of shear on a smaller amount of material when compared to that of a concrete barrel mixer on a concrete truck, which generally has a capacity of ten cubic yards of concrete, one believes that the procedures used for the fresh property examination in this study are sufficient. After each air content measurement the top layer of water that is used in the testing is removed from the concrete to limit the addition of water to the concrete mixture. With every air content measurement, a fresh temperature and density of the concrete is also taken according to the standard described in ASTM C192.

In addition to the fresh property testing performed on the mixes, four by eight inch concrete cylinders and standard free shrinkage concrete prisms are cast to perform extensive performance based tests. The performance based tests implemented on the prisms and cylinders include strength testing, ASTM C39, petrographic examination, ASTM C457, free shrinkage testing, ASTM C157, freeze thaw testing, ASTM C666, rapid chloride penetration testing, ASTM C1202, standard chloride ponding tests, ASTM C1581, hardened air void analysis, ASTM C457, and elastic modulus testing, ASTM C469. Three replicate samples are used for
each standard test performed in the study of each property. No duplicate testing is performed on
the fresh property testing due to the amount of wasted concrete mix that would be needed in
order to run duplicates on the hour long fresh property testing. The results from the tests
performed are shown below in the results section of this chapter, Chapter 4, and the appendix.

2.3 Results

Below Figures 5 and 6 display every reading for slump and air content for each mix tests.
In the figures provided, one can see that several of the mixtures exhibited large amounts of both
slump and air content loss over one hour of testing. Compression tests for each mix are
performed and shown below in Figure 7. Compression results are an average of three cylinders
and are within the required limits specified by the ASTM standard (ASTM Standard C 39, 2012).
Figure 5: Graph depicting slump taken for each mixture over one hour of testing fresh properties
Figure 6: Graph depicting air content taken for each mixture over one hour of testing fresh properties.
2.4 Discussion and Conclusions

Performing one slump and air content measurement at the beginning of a concrete placement does not represent the behavior of the material near the end of the placement. Shown in Figures 5 and 6 above, the concrete mixtures can behave much less workable at the end of a placement, estimated at one hour, which can lead to workability issues in a field placement. From the fresh property testing it is clear that slump and air content in the concrete can be greatly affected by time before placement. The focus of this testing is placed not on the initial slump or air content but more on the loss of slump and air content over time. From the measurements of slump loss one can see the estimated difference is constructability of a concrete mixture near the end of the placement of a concrete mix. Performing slump loss and air content loss allows for the concrete mix design engineer to better understand the homogeneity of the material and if the material will have desired constructability properties. The loss of slump and air content is seen
as a more appropriate measure of constructability since loss depicts consistency of the material over time.

In addition to the workability tests, compression tests are performed on cylinders from each mix in order to determine the strength. The strengths found through compression testing are more than adequate for bridge deck applications as specified by the current Tollway specification. The minimum compression strength specified by the current Tollway specification is 4,000 psi at 14 days.
3. Creep and Shrinkage

3.1 Background

Understanding creep and shrinkage properties is vital when trying to evaluate a concrete for performance when considering early age cracking in restrained concrete. Shrinkage is known to occur in every concrete material. Different types of shrinkage occur in concrete. The types of shrinkage are discussed fully in Chapter 1 of this document. Upon shrinkage in a restrained concrete, stress occurs inside the concrete matrix since the concrete is not allowed to shrink freely due to the restraint. In concrete bridge decks the restraint is caused by steel reinforcing bars, but restraint can be seen in nearly all concrete applications. When a concrete is restrained, creep occurs in the material which effectively relaxes the stresses caused by shrinkage. In order to avoid early age cracking of a concrete material, tensile creep and shrinkage must be examined.

3.2 Procedures

Creep and shrinkage are examined through the use of the restrained ring test and the use of a restrained shrinkage testing machine (RSTM) developed at UIUC by Dr. Salah Altoubat, Dr. Matthew D’Ambrosia, and Professor David Lange (Altoubat & Lange, 2001). The restrained ring test is performed in accordance with ASTM C1581 at CTL. The RSTM is performed based on specifications written from extensive testing performed by Altoubat and D’Ambrosia (D’Ambrosia, 2011).

3.2.1 Restrained Ring Test

The restrained ring test is performed in accordance with ASTM C1581 which requires the geometry shown below in Figure 8 and a frictionless base for the concrete to rest on in order to reduce restraint not caused by the steel ring itself.
After twenty-four hours of curing, all three duplicate rings are demolded by removing the outer ring mold and testing begins. During the demolding process the ring is covered on top with aluminum tape to prevent the concrete material from drying from the top which would complicate the drying shrinkage seen in the concrete ring. The sides of the concrete are exposed to the environment of the testing room to allowing drying in the material to occur. The rings are stored inside an environmental room maintained at 50 ± 2% RH and 23 ± 0.5°C.

Due to the limited space in the geometry shown, the concrete used to perform the ring tests is wet sieved in order to drop the minimum allowable aggregate size to one third the size of the 1.5 inch allowable space between the steel ring and outer mold of the ring. Reducing the maximum aggregate size to one third the allowable space is standard for reinforced concrete.
structures to allow for proper consolidation of the aggregates and to avoid segregation of the aggregate between the rebar in the structure. Allowing larger aggregate causes for the rebar to act as a sieve and for larger aggregates to be removed, causes for large heterogeneity of the concrete mix placed. In turn, wet sieving the larger aggregates out of the concrete causes for a higher percentage of paste to be present in the mortar, which causes for a higher rate of shrinkage in the material compared to that of the concrete matrix. The ASTM standard ring is chosen for this testing because it allows for a higher amount of restraint on a material with higher shrinkage which in turn causes for shorter testing time to be observed.

An alternative ring test standard exists which allows for a spacing of three inches between the steel ring and outer ring mold, but is not used in this instance due to longer testing time needed to develop the same amount of stress in the material. The alternative restrained ring standard is AASHTO PP34-99, which uses a smaller inner steel ring diameter in order to accommodate the larger aggregate size, shown below in Figure 9.

![Figure 9: AASHTO Ring Dimensions](image)

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The AASHTO standard is not as commonly used since the steel ring is required to be thicker to produce the same amount of restraint as is exhibited in the ASTM restrained ring test. The thicker steel ring causes for additional issues with resolution of the strain gages attached to the ring that measure the strain imparted on the steel ring, which causes for inconsistencies in the AASHTO set up. In order to obtain the appropriate resolution to measure strain properly, the steel ring in the AASHTO set up needs to maintain the same thickness as the ASTM standard which as state before reduces the amount of restraint seen in the concrete matrix and causes for longer testing time. It is also important to note that since the AASHTO specification uses concrete as opposed to a sieved concrete mortar, the concrete experiences a smaller amount of shrinkage due to a lower percentage of paste in the cementitious material matrix which also delays cracking of the restrained concrete ring specimen. Due to all the issues discussed, the ASTM restrained ring standard is chosen to evaluate the concrete mixtures in the testing.

3.2.2 Restrained Shrinkage Testing Machine

3.2.2.1 Technical Background

The RSTM is a dogbone-shaped uniaxial testing frame that provides full restraint to a concrete material through an intricate closed-loop testing process. The dogbone-shape is characterized by a long slender middle portion with flared ends for gripping. The slender middle portion of the specimen is to promote cracking within the specimen’s gage length. The flared ends of the dogbone-shaped specimen in this test are designed to eliminate stress concentrations at the grips of the specimen. The test consists of two identical dogbone-shaped specimens with geometry and photos shown below.
Figure 10: Restrained shrinkage testing dimensions

Figure 11: Image of the restrained concrete specimen with crack
One specimen is allowed to shrink freely, Figure 12, during testing while the second sample is fully restrained, Figure 11, by a hydraulic actuator that periodically places load on the specimen depending on the shrinkage rate of the sample. The shrinkage in both specimens is measured by linear variable displacement transducers, LVDT, with a range of plus or minus 2.54 mm. In the restrained specimen the specimen is allowed to shrink within a specified threshold, .005 mm or with this specimen geometry 8 με. When the specimen is allowed to shrink, the actuator is considered to be in load control, meaning that the load is held constant on the specimen and the position is free to change according to the drying shrinkage exhibited in the concrete specimen. When the restrained specimen shrinks below the threshold value specified, the hydraulic actuator changes to what is known as position control. In position control the actuator applied an additional load that moves the specimen back to its initial strain at the beginning of the test. This additional load is determined by the amount of force required to return the specimen to its initial position. Upon the specimen’s return to the initial position the actuator returns to load control, allowing the restrained specimen to shrink due to drying shrinkage of the concrete. This load control versus position control is shown schematically below in Figure 13.
Early age creep properties can be found from the RSTM testing since both a restrained specimen and companion free shrinkage specimen are measured throughout the testing of the concrete specimens. The restrained specimen exhibits both the free shrinkage properties and the creep response during the duration of the test. Creep can be found in the restrained specimen by simply subtracting the free shrinkage from the total creep plus free shrinkage curve found through testing. Once creep is found, it can be displayed in several different ways that assist the researcher in analysis the performance of the material at early ages.

3.2.2.2 Testing Setup

The concrete is first mixed and placed in the molds and allowed to cure in a sealed condition for twenty-two hours before the test begins. Demolding begins twenty-two hours after placement which allows both specimens to equilibrate thermally for two hours before testing.
The demolding process first involves removing the side pieces of each mold to allow drying shrinkage in the specimen. Next the specimens are covered on the top with aluminum tape in order to eliminate drying from this surface. The LVDT and straight bar for measuring shrinkage are then added to the specimen at the points where hooks have been placed in the concrete specimen. After situating the LVDT the remaining screws that attach the specimen to the Teflon covered superstructure are removed to allow nearly frictionless movement of the specimens being tested. The LVDTs for both specimens are then zeroed manually and the restrained specimen is attached to the hydraulic actuator. A hydraulic actuator capable of 50 kN of force is used in the test setup. Once demolding is completed, the test is ready to begin, twenty-four hours after placement. Approximately one hour after the test begins the threshold value is set for the restrained specimen to ensure that shrinkage has begun in the specimen to avoid a compressive load being placed on the specimen in concretes that show expansion. Testing occurs until the specimen fails or until the end of one week’s time in the testing performed.

3.3 Results

Results are shown below in Figure 14 for the restrained ring tests run on the concrete specimens. The results are shown as an average of three restrained rings made from the same batch of concrete. In the figure, cracking of one ring is displayed as a sharp vertical line. When cracking occurs, the remaining data is an average of the remaining specimen.
The results from RSTM testing are displayed in several figures below. The figures depict stress in restrained specimens (Figure 15), creep (Figure 16), creep coefficient (Figure 17), specific creep (Figure 18), and free shrinkage (Figure 19) for each material tested.

Figure 14: Results from the restrained ring tests made on the proposed concrete mixes (CTL)
Figure 15: Stress implemented during load cycles

Figure 16: Creep experienced in the samples during testing
Figure 17: Creep coefficient found from creep

Figure 18: Specific creep for the mixes tested
3.4 Discussion and Conclusions

The results of the restrained ring tests and RSTMs indicate that the early age mitigation strategies examined as a whole perform much better than the standard bridge deck mix. The SRA and ULT mixes perform best when considering free shrinkage and early age properties. Both the SRA and ULT mixes do not experience any cracking in the restrained ring test as shown in Figure 14. All other mixes exhibit at least one ring cracking during the time of testing with the BS mix having all rings crack before the end of testing.

The SRA and ULT mixes experience the smallest amount of free shrinkage in the RSTM testing which is further indication that these two mixes are the best performing mixes when considering the prevention of early age cracking in restrained bridge decks. Besides looking at free shrinkage, the RSTM provides information regarding creep properties for the mixes. As expected by examining the stress in each mix, the BS and SLA#1 mix experience the highest

Figure 19: Free shrinkage for all samples tested
amount of creep, shown in Figure 16. From further examination of creep coefficient and specific creep results, the SLA#1 mix unexpectedly is outperformed by the BS mixture.

The SLA#1 mix is shown to be the worst-performing mix. To further investigate this case, the expanded slag used in the SLA#1 mix is further examined to understand if the issues arise with this mixture due to the internal curing source. It is found that when performing the calculations for water content in the aggregate before mixing that the expanded slag has a moisture percentage content well about the maximum moisture content expected for this particular aggregate, meaning that additional water is being held inside the aggregate that is not considered free water. The additional trapped water is being account as free water when batching of the concrete, but is not available for internal curing purposes inside the cement paste matrix. Since the aggregate is not going to be able to freely release the trapped water, the SLA#1 mix effectively has a w/cm that is much lower than that of the other mixes. The accounted for trapped water effectively drops the w/cm by .07, from a .42 design to a .35 actual. The drop in w/cm causes for the material to have a much higher cracking potential due to higher amounts of early age shrinkage. The lower w/cm in the RSTM explains the discrepancy between the ring results for the mix, with the SLA#1 mix only having one cracked ring, compared to the poor performance in the RSTM.

Trapped water is able to accumulate inside the expanded slag aggregate because of the way the aggregates are stored before mixing. The aggregates are stored in five gallon buckets, constantly submerged in water. This storage method is not a problem when considering aggregates with highly interconnected pore structures that are able to freely release all the water that they hold. The issue of trapped water only arises in aggregates that have a more poorly connected pore structure, such as in the expanded slag aggregate. The expanded slag aggregate,
as shown in Chapter 5, has larger pores inside the aggregate that act as reservoirs when submerged for extended periods of time. When submerged for three days, per ASTM 1761, the aggregate experiences a much lower maximum absorption capacity when compared to longer submersion times. When the expanded slag is submerged for long periods of time, the internal reservoirs that fill much more slowly due to poor connections to outside pores are able to more fully saturate since the absorption of the aggregate is a time dependent variable. The water inside the aggregate is then accounted as free water when moisture contents are found for the aggregate before mixing which is the cause of the lower effective w/cm as mentioned before.

The best way to prevent the improper compensation for moisture content in the LWA is to more closely monitor the pore structure of a given lightweight aggregate to determine the proper storage for the aggregate. As seen in SLA#2, the expanded shale aggregate used, which is stored and prepared for mixing in the exact manner as the expanded slag, performs very well in the RSTM and even experiences some expansion at early ages. The expanded shale has a much better connected pore structure which allows for water to more freely move in and out of the aggregate for internal curing purposes and has less of a chance to trap water. Both aggregates are shown to having internal curing capability, it is just important to store the aggregates properly in order to provide the most efficient use of internal free water inside the cement paste matrix.
4. Chloride Penetration and Freeze-thaw Resistance

4.1 Background

As discussed extensively in Chapter 1, chloride penetration causes many common deterioration mechanisms inside a concrete matrix. Chloride penetration through the bulk concrete material is dependent on the size and percolation of the pore structure in concrete. One common way of reducing chloride penetration in a concrete matrix is to reduce the w/cm which allows for a drop in capillary pore volume which in turn means a denser concrete material. Reducing the capillary porosity is a well-known and trusted way of reducing the penetration of chlorides into the concrete matrix. As discussed in Chapter 2, issues arise when considering constructability when w/cm is dropped, which causes a research engineer to weigh the benefits of a lower w/cm when considering chloride penetration to the negative impact a lower w/cm can have on other properties of the material, including early age shrinkage and constructability.

Freeze-thaw durability is considered for the concretes tested for the Tollway due to the location of Chicago in the Midwestern United States. Environmental freeze-thaw cycles can have a negative impact on concrete if proper air is not entrained inside the cement paste matrix. When water freezes it expanse into ice crystals, inside concrete this expansion causes internal stresses to be produced if there exists no room for the expansion to occur. When air is entrained into the cement paste matrix, the expansion caused by freezing has room to grow into the entrained air pockets which causes minimal internal stresses and higher resistance to freeze-thaw cycling (Mindess, Young, & Darwin, 2003).

4.2 Procedures

ASTM standard C1202 is used to evaluate each concrete’s performance when considering chloride penetration. ASTM C1202 is the standard for rapid chloride penetrability
of a concrete. Rapid chloride penetrability, RCP, is a test that measures electrical conductance between sides of a concrete exposed to a chloride solution. The electrical conductance gives an idea of how permeable a given concrete is to chloride penetration, but does not produce a specific number that directly relates to the amount of chlorides that penetrate into the concrete sample. RCP tests are run using three duplicates on specimen geometry cut from standard four by eight inch concrete cylinders after seven days of accelerated aging.

ASTM standard C666 is used to evaluate the freeze-thaw resistance of the concrete’s being tested. ASTM C666 uses concrete prisms with specimen dimensions 3 inches by 3 inches by 11.5 inches. The prisms are placed inside water in a refrigeration unit that is programmed to cycle the specimens above and below the freezing point of water. The prisms are cycled a minimum of three hundred times at two to five cycles a day. After testing, weight loss of specimens and ultrasonic pulse echo are taken on the specimen to determine percent damage in the specimens. Three prisms are tested in freeze thaw after lime-water curing for fourteen days.

4.3 Results

The results for the RCP testing are shown below in Figure 20 for concrete specimen after twenty-eight days of accelerated curing. The freeze-thaw testing results are shown in Figure 21 below. All samples tested perform at a high level in freeze-thaw, experiencing nearly no damage after over 300 freeze-thaw cycles.
Figure 20: Results for rapid chloride penetrability testing (CTL).

Figure 21: Results from freeze-thaw testing (CTL).
4.4 Discussions and Conclusions

The results for chloride penetration testing for the concrete mixes examined fall inside the accepted range discussed in the ASTM standards. As seen in Figure 20, the ULT mixture is by far the best performing mix in the RCP testing. The results provide evidence that chloride resistance is possible in concrete mixes with w/cm of 0.42. Maintaining chloride resistance with higher w/cm mixes allows for concrete mixtures for the bridge decks to inherently have lower early age cracking potential since higher w/cm concrete mixes are known to produce lower shrinkage values when compared to lower w/cm mixes.

The results in Figure 21 show that the concretes perform very well when considering freeze-thaw resistance. The freeze-thaw testing validates that the amount and geometry of the air entrained in the concrete is adequate to diminish deterioration caused by freeze-thaw.
5. Moisture Movement

5.1 Background

Moisture movement is a property being considered in concrete mixes today due to advances in internal curing and identification of the cause of deterioration mechanisms. The moisture movement in a specimen can be examined as bulk moisture moving through the concrete or can be considered as how moisture moves inside a drying concrete specimen. The way moisture movement is considered in this chapter is not bulk moisture movement through the concrete material, but instead as how LWA affects relative humidity inside the concrete itself and how different types of LWA provide different amounts of moisture that is free to move throughout the concrete matrix. The LWAs used in testing are fully characterized and mixed inside concrete to determine the effects these aggregates have on the concrete matrix as a whole.

5.2 Procedures

5.2.1 Aggregates and production

The LWAs used in testing, as stated in Chapter 1, are an expanded slag and expanded shale aggregate. The expanded slag used is a material known as Trulite which is manufactured by LaFarge in the Chicagoland area. The expanded shale is known as Hydrocure and is produced by Northeast Solite Corporation, which is based out of Kentucky. The expanded slag is produced by a cooling process known as foaming that allows for pores to form in the molten slag that are stable when cooled to room temperature. Foaming is a process that cools the aggregate at an intermediate rate compared to air cooling and quenching by providing water at small dosages in order to provide a mist to occur that causes the porous structure of the
aggregate. The expanded shale is produced by a high temperature kilning process that also produces a stable porous structure at room temperature.

5.2.2 Characterization Tests

Both LWAs are characterized using a modified version of the ASTM standard for LWA, ASTM C1761. The modifications used involve adding additional desorption points to the lightweight aggregates for better characterization at differing relative humidity and performing full submersion absorption tests with oven dry aggregates in addition to the stand absorption test described in ASTM C1761. The instantaneous absorption test developed in this study of LWA is to be called the petri dish test due to the petri dish used for containing the material. Additional evaluations are also run for both aggregates to better understand the pore structure and composition of the aggregates which include computed tomography scanning, secondary electron imaging, and x-ray diffraction on oven dry powder made from each aggregate.

5.2.2.1 Absorption and desorption tests

Absorption and desorption are two properties commonly examined when using LWA for internal curing purposes. Absorption capacity is important to know for a given aggregate in order to account for mix water that is to be used as internal curing water, known commonly as free water. Absorption capacity also plays a role in determining how much lightweight material must be added to a concrete mix in order to achieve the desired internal curing benefits. Absorption capacity for each material is found using ASTM C1761 along with fully submerging the material for three days each. The submersion tests are data logged with a scale and computer, with periodic agitation of the aggregates in the fine wire mesh basket used in order to remove air bubbles that are trapped around the aggregates.
Desorption is the parameter that allows the research to understand how water is released from the aggregates inside the concrete matrix. For a LWA to be efficient for internal curing applications it must be able to desorb, or give up, the free water included inside the aggregate at high relative humidity inside the concrete matrix. The benefits of free water inside the concrete matrix after set are greatly reduced if the aggregates do not release the water at high relative humidity. When the aggregate releases the free water at higher relative humidity, the free water is able to travel further distances and add in the hydration process more efficiently. Since efficiency is key in desorption, ASTM C1761 requires desorption characteristics to be examined at 94% relative humidity. Additional relative humidity points are added in this examination of the aggregate in order to find a curve that better describes the desorption capacity of each aggregate used. Desorption tests are ran on small samples inside a controlled relative humidity. The relative humidity inside each container is controlled by super saturated salt solutions with known relative humidity values. The containers are checked with relative humidity sensors in order to certify that the relative humidity of the supersaturated salt solutions are being maintained. Desorption measurements are taken daily until equilibration.

5.2.2.2 X-ray diffraction

Powdered x-ray diffraction, XRD, is a bulk method used to determine crystalline phases of a material. XRD is commonly used as the first step when performing a characterizing of a given material. The machine used in testing the LWAs is a Siemens Bruckner D5000 with a scintillation detector, with a monochromator. The LWAs are ground after oven drying to a powder passing the #275 sieve. The test range chosen for both LWA specimens is 5-80°. The scan speed chosen is 1.5°/min in order to obtain a scan that will perform the specific range in one
hours-time. The data is taken at an increment of every 0.03°. Cu Kα radiation is used to perform the XRD scan. The radiation is produced at 40 kV and 30 mA.

5.2.2.3 Computed x-ray tomography

Computed x-ray tomography, CT, is performed on both LWA specimens. CT scanning involves taking x-rays around the perimeter of a sample then reconstructing the images from the x-ray in order to achieve a cross-section of the sample being examined. A 180° rotation is required for a full reconstructed image of the sample being scanned. CT scans are taken to better understand the pore structure of the aggregates. Since the CT scans are able to take images of a full aggregate, it is a great way to examine the pore structure of lightweight aggregates when considering their efficiency in internal curing applications.

The sample is prepared by first oven drying the aggregate. After oven drying, the aggregates are cooled and place inside a thin plastic syringe that is filled with epoxy. Once the epoxy cures, the sample is placed on a special mount, shown below in Figure 22.
The sample is then scanned at 50 keV and 6 Watts producing a current of 121 μA for 8 hours. Ten images are taken per degree imaged by the scanner, causing for nearly one thousand cross-section images to be produced after reconstruction. In some applications the thousands of cross-sections can be constructed together in order to produce three dimensional images, but this procedure is not performed in this study of lightweight aggregate porosity.

5.2.2.4 Scanning electron microscopy

Scanning electron microscopy, SEM, is a method used to image at higher magnification and resolution than optical microscopy. SEM can be performed qualitatively by either backscattered electron imaging or secondary electron imaging. Secondary electron imaging, SEI, is chosen to look at the topography of the lightweight aggregates since specific topographic characteristics are lost when using a polished section, which is needed to properly perform backscattered electron imaging. The specimens are prepared by first oven drying and then
placing each aggregate on top of SEM mounts by the use of carbon tape. After mounting has
occurred, the aggregates are then sputter coated with gold palladium for thirty seconds to better
eliminate charging of the specimen. After applying the sputter coating, the mounted and covered
specimen is placed in the microscope.

The SEM used to image the specimen is a JEOL JSM-6060LV low vacuum scanning
electron microscope. The electron source for the microscope is a pre-centered tungsten hairpin
filament with a probe current of 1 pA to 1000 nA. The microscope is run at 5 keV with a
working distance of 10. The low keV is used to lessen the effects of charging seen on the
specimen.

5.3 Results

5.3.1 Absorption tests

Below the results for the submersion absorption tests are presented in Figures 23 and 24.
In Figure 23 one can see that trulite has a high initial absorption rate followed by a much slower absorption rate. In Figure 24, one can see that hydrocure is greatly affected by time of soaking. Figure 24 also shows the inadequacies associated with submersion testing for LWA. During the test the researcher had to administer agitations, seen at around hours ten and forty, in order for bubbles trapped around the aggregate to be released. These bubbles form around the aggregate surface and cause for error in measuring absorption. As shown in Figure 24, the bubbles collected on the aggregates during testing greatly affect the data logged during the test.
5.3.2 Desorption Tests

Desorption Tests are run at several different relative humidity points in order to characterize the two LWA used. Figure 25 below shows the desorption results for both trulite and hydrocure. Hydrocure has additional relative humidity points compared to trulite since hydrocure is believed to be more sensitive to humidity change.
Figure 25: Desorption curves for expanded shale and expanded slag aggregates.

As is shown in Figure 25, both aggregates lose moisture readily at high relative humidity. The hydrocure outperforms the trulite slightly when considering desorption. The salts used in desorption testing are shown below in Table 3 with their respective relative humidity (Greenspan, 1977).

<table>
<thead>
<tr>
<th>Salt</th>
<th>RH (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Potassium sulfate</td>
<td>97.5</td>
</tr>
<tr>
<td>Potassium nitrate</td>
<td>94</td>
</tr>
<tr>
<td>Barium chloride</td>
<td>90</td>
</tr>
<tr>
<td>Sodium chloride</td>
<td>75.35</td>
</tr>
<tr>
<td>Sodium bromide</td>
<td>58</td>
</tr>
<tr>
<td>Magnesium nitrate</td>
<td>53</td>
</tr>
<tr>
<td>Environmental Chamber</td>
<td>50</td>
</tr>
</tbody>
</table>
5.3.3 XRD

Figures 26 and 27 show XRD results for trulite and hydrocure, respectively. The mineral components of the LWA vary greatly in comparison. The variation in composition of material is intuitive because of the processes used in producing such materials. The trulite aggregate is seen to be slightly amorphous in the XRD results meaning it has possibly reactive tendencies in concrete. Trulite crystal phases found include merwinite and akermanite. Hydrocure crystal phases found include silicon oxide and magnesium silicate.

![XRD of expanded slag used in testing](image)
5.3.4 Computed Tomography

Four typical CT cross-sectional images of LWA are shown below in Figures 28 through 31. Figures 28 and 29 display hydrocure where Figures 30 and 31 are of trulite. These images allow for qualitative understanding of the porous microstructure that exists inside of LWA. The images show great contrast between the inner pore structure of hydrocure and trulite.
Figure 28: CT image of expanded shale
Figure 29: Alternate CT image of expanded shale
Figure 30: CT cross sectional image of expanded slag aggregate
5.3.5 Scanning Electron Microscopy

Secondary electron images of both LWAs are taken and shown below. Hydrocure images are shown at varying magnifications in order to shown the topography of the LWA pore structure along with the interconnectivity of the pore structure seen for the hydrocure aggregate. Trulite is imaged in a similar way, but much different structure is seen when considering the
trulite. Trulite exhibits a less connected structure compared to hydrocure, which is reflected in the images provided in Figures 35-39.

A low accelerating voltage is used in scanning of both LWAs in order to better prevent charging of the specimen. Charging is still seen in a few of the images displayed due to the porous nature of the LWA causes for issues when sputter coating is applied. Also a larger spot size is used for clarity of images.

Figure 32: Secondary electron image of expanded shale LWA showing porous nature of the aggregate
Figure 33: Higher magnification secondary electron image of expanded shale LWA

Figure 34: Low magnification of porous surface of expanded shale LWA
Figure 35: Low magnification image of expanded slag aggregate

Figure 36: Secondary electron image of expanded slag LWA showing contents inside pore space
Figure 37: Higher magnification of expanded slag to identify interesting features.

Figure 38: Even higher magnification to better see the unique features inside the pore structure of the expanded slag.
5.4 Discussion and Conclusions

From the extensive testing performed on both LWAs several differences are made clear when comparing the LWAs that help to understand the results when comparing SLA#1 and SLA#2 in Chapter 3. When examining first the absorption behavior the LWA one understands from the submersion testing that the expanded shale benefits from long term submersion while trulite does not benefit from long term submersion. Also with the submersion testing the results confirm past work that several issues arise with submersion testing that cause for issues in reporting results. The main issue found with submersion testing is the collection of bubbles on the aggregate surfaces that can create measurement errors leading to low specific large errors when proper agitation is not provided to the aggregates.
The results of desorption tests show acceptable behavior from both LWA sources, but with the expanded shale exhibiting a higher loss of internal RH at higher external RH. The importance of drop in internal RH of the aggregate is associated with the distance the water is able to move inside the cement paste matrix. When free water is released from the internal curing source material at higher RH levels the free water is able to travel much further distances. If the free water is not able to be released from the LWA until a lower cement paste RH the water will not be able to travel as far since the cement hydration process will be further along in the process causing for a denser cement paste structure. The denser structure by definition is less accessible to penetration of any kind, including water. Due to the reasons listed, from the desorption experiments hydrocure is slightly more advantageous to use for internal curing.

XRD is performed on the aggregates in order to obtain a full characterization of each material and determine crystallinity of the materials. Trulite is found to be made of akermanite and merwinite with a slight amorphous hump. The amorphous hump is characteristic of reactive materials such as cement and SCMs. The amorphous hump leads one to believe trulite is not fully inert inside the concrete. Further work is needed to determine the possible reactivity of the trulite aggregate. Upon XRD testing, hydrocure is found to be made of silicon oxide and magnesium silicate. Hydrocure does not show an amorphous hump of any kind and is believe to be inert inside the concrete matrix.

CT scanning is performed on both LWAs in order to better understand the internal pore structure of each aggregate. CT scanning is used as opposed to back-scatter electron imaging since CT scans provide information for all planes of a specific aggregate as opposed to one plane which is the case for back-scattered imaging. From the CT scans the hydrocure is shown to exhibit a structure that includes small pores that have high interconnectivity. The network of
pores in the hydrocure shown allow for water to flow quickly in and out of the aggregate. Differently the trulite CT images show, as hypothesized, large internal pores that are not well connected to the outside pores of the aggregate. Due to poor connection of the large internal pore network, these large pores fill at a much slower rate and therefore are not useful as reservoirs for free water. The large pores, as discussed in Chapter 3, tend to instead trap water as opposed to freely releasing the water. When trulite is not soaked for extended periods of time it is able to perform much closer to what is expected of the aggregate from the maximum absorption rate found for the aggregate. Under extended soaking, the large internal voids trap water and cause for a lower w/cm concrete because the trapped water is accounted for when moisture contents are found before batching of the concrete.

Secondary electron images of both aggregates reveal differences in the surface and pore structure of the aggregates. Hydrocure is seen to exhibit a porous surface with an intricate internal pore structure that can be examined through outer porosity. When imagining is performed at high magnification inside pores of hydrocure, more pores are found, Figure 33. The pores seen at high magnification are further evidence of the highly interconnected pore structure of hydrocure. The secondary electron images of trulite provide further evidence of lower amounts of porosity. Trulite has a denser surface with larger surface pores. Inside the larger pores, interesting features can be seen and some additional porosity is found, but not nearly to the extent of hydrocure. From the secondary electron imaginative it is clear that the porosity in the two LWAs used is very much different even though both aggregates have shown capacity for internal curing applications.

The largest idea to be drawn from the characterization of the aggregates is that in order to have the highest efficiency of your aggregate for internal curing applications is to fully
characterize the aggregates. The aggregates each have unique and individual characteristics that affect the application of internal curing and need to be better understood.
6. Discussion and Conclusions

All of the mitigation strategies examined in this study are able to improve the resistance to early age cracking and therefore are more likely to produce the longer service lives sought by the Tollway. Reduction in shrinkage at early ages is achieved through the use of common mitigation strategies used in the concrete industry. The use of shrinkage reducing admixtures, saturated lightweight aggregates, and gradation optimization are each able to produce concretes that outperform the base concrete mixture specified by the Tollway when considering early age distresses such as restrained shrinkage testing and free shrinkage. The best performing mixes are the mixes using shrinkage reducing admixture, but due to the costs associated with using shrinkage reducing admixture, the other mitigation strategies may be more cost effective options to reduce early age cracking.

Another finding of the project is that not all lightweight aggregates behave similarly. Lightweight aggregates can have vastly different physical microstructure due to how they are manufactured which affects the absorption and desorption characteristics of the aggregate. The absorption and desorption characteristics in turn effect the aggregate’s ability to be an efficient internal curing source. The differences in the aggregate must be taken into account when working with lightweight materials. In order for the aggregates to be properly used the practicing engineer must know the materials used well. Simply using a LWA will not cause concrete to magically perform better in shrinkage tests. It is important to examine the aggregates and characterize these aggregates fully in order to optimize the results of internal curing. Evidence is shown in the concrete mixture using expanded shale, where the effective w/cm is much lower than that of the other mixes tested due to errors in lightweight aggregate preparation yet it is still only slightly outperformed by the standard Tollway bridge deck mix.
From the RCP testing it is clear that higher w/cm concretes can still perform well when considering chloride penetration. Each mitigation strategy is seen to cause better performance when considering chloride testing, even at higher w/cm. The w/cm is not incredibly high at 0.42, but it is seen through preliminary testing that the 0.42 w/cm concrete mixes add great amounts of workability and constructability in comparison to lower w/cm mixtures examined. A higher w/cm allows for a much higher constructability and a much higher resistance to early age distresses. Since the chloride testing performed shows adequate performance, one believes that in concrete bridge decks it is more important to consider early age cracking compared to chloride penetration when trying to protect the reinforcing steel of the bridge deck from chloride penetration. The chloride penetration through the concrete is shown to be slow and therefore not nearly as costly when examining the idea of durability when compared to early age cracking. The results of the RCP tests is the reason the majority of the study has placed interest on early age cracking resistance compared to chloride permeability.

Lastly it is important to remember that fresh properties of concrete greatly affect constructability of the concrete during placement. The fresh properties are time-sensitive, and therefore they must be accounted for when designing a concrete for the field. In addition to performing slump loss and air loss testing in the laboratory setting, additional slump and air content testing in the field can be seen as beneficial to determine constructability of the mix during the placement and to aid in quality assurance and quality control of the concrete.
References


Eclipse 4500. (n.d.). Retrieved 4 22, 2013, from Grace Concrete Products:


Appendix A: Preliminary Testing

A.1: Proposed test matrix

Appendix A includes information in stage one of the project. The data found in stage one of the project includes fresh properties and compression testing of fifteen different concrete mixes, 3 different w/cm for each mix. Three different w/cm are examined for each mix in order to produce three point curves for each mix. Three point curves allow a research engineer to understand the robustness of a concrete mix before deciding the w/cm to be used for the design mix.

Table 4: Proposed test matrix for stage one of testing.

<table>
<thead>
<tr>
<th>Stage 1 Tests</th>
<th>BS</th>
<th>OPT</th>
<th>SLA</th>
<th>SRA</th>
<th>ULT</th>
</tr>
</thead>
<tbody>
<tr>
<td>C39</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>C192</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<td>Slump Loss</td>
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<td>X</td>
<td>X</td>
<td>X</td>
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<tr>
<td>Air Loss</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>
A.2 Results

Figure 40: Three point curve showing robustness of BS mix design
Figure 41: Three point curve showing robustness of OPT mix with additional lower w/cm

Figure 42: Three point curve for SLA mixes
Figure 43: Three point curve for SRA mixes.

Figure 44: Three point curve for ULT mixes.
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<td>7/3/12</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
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<td>494</td>
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<td>409</td>
<td>452</td>
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<td>0</td>
<td>0</td>
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<td>136</td>
<td>136</td>
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<td>0</td>
<td>131</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>151</td>
<td>142</td>
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<td>Lafarge NewGen Slag</td>
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<td>106</td>
<td>114</td>
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<td>144</td>
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<td>0</td>
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<td>0</td>
<td>402</td>
<td>394</td>
<td>398</td>
<td>395</td>
<td>388</td>
<td>391</td>
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<td>0</td>
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<td>0</td>
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<td>1800</td>
<td>1800</td>
<td>1800</td>
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<td>1510</td>
<td>1595</td>
<td>1487</td>
<td>1501</td>
<td>1886</td>
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<td>1714</td>
<td>1831</td>
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<td>0</td>
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</tr>
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<td>Saturated Aggregate</td>
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<td>1.23</td>
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<td>0</td>
<td>0</td>
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<td>1412</td>
<td>1379</td>
<td>1374</td>
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<td>978</td>
<td>986</td>
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<td>1323</td>
<td>1002</td>
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<tr>
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<td>Roble's Shoals, IL</td>
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<td>263</td>
<td>263</td>
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<td>184</td>
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<td>237</td>
<td>232</td>
<td>226</td>
<td>220</td>
<td>220</td>
</tr>
</tbody>
</table>
| Target Slump, in.   | 6-8     | 6-
| Design Air Content, % | 6      | 6       | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      | 6      |
| Slump Flow, in.     | ASTM/C1611 | 5.75 | 6.25 | 5.25 | 3.25 | 4.25 | 7 | 8.5 | 8.5 | 4.25 | 2.5 | 6.5 | 7.25 | 7 | 6.75 | 7 | 4.25 | 6 | 3.5 |
| Air Content, %      | ASTM/C231 | 5.8% | 6.9% | 6.3% | 7.8% | 7.0% | 6.2% | 5.2% | 6.2% | 7.1% | 6.2% | 6.3% | 5.6% | 5.3% | 4.4% | 4.4% | 4.3% | 4.5% | 4.5% |
| Temperature, °F     | ASTM/C1064 | 83 | 81 | 81 | 81 | 81 | 78 | 78 | 77 | 75 | 80 | 75 | 76 | 76 | 77 | 76 | 76 |
| Fresh Density, lbf/ft³ | ASTM/C138 | 146 | 144 | 145 | 147 | 147 | 148 | 151 | 147 | 146 | 147 | 145 | 149 | 149 | 150 | 145 | 144 | 145 |
| Age                 | 3 day    | 5,580 | 4,647 | 5,900 | 6,277 | 6,577 | 7,130 | 8,207 | 5,777 | 5,267 | 7,857 | 5,367 | 5,090 | 5,770 | 4,857 | 4,473 | 4,833 | 4,280 | 3,837 |
|                     | 7 day    | 6,780 | 5,703 | 6,840 | 7,313 | 7,813 | 8,337 | 9,783 | 6,777 | 6,457 | 9,187 | 6,983 | 6,733 | 7,167 | 6,170 | 5,517 | 7,173 | 4,670 | 5,923 |
|                     | 28 day   | 8,177 | 7,060 | 8,000 | 9,187 | 9,860 | 9,873 | 11,407 | 8,313 | 8,130 | 10,643 | 8,667 | 8,167 | 8,710 | 7,440 | 7,137 | 9,657 | 9,043 | 8,897 |

Table 5: Tabular results for stage one testing
Appendix B: Additional information from testing

B.1: Proposed testing matrix

Appendix B includes data presented in tables that is presented in the thesis and additional testing performed in stage 2 of the project that is not presented in the thesis document.

Table 6: Proposed Testing Matrix for second stage testing. Tests in green and blue are performed.

<table>
<thead>
<tr>
<th>Stage 2</th>
<th>Performance Tests</th>
<th>BS</th>
<th>OPT</th>
<th>SLA</th>
<th>SRA</th>
<th>ULT</th>
</tr>
</thead>
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<td>C39</td>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
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<td>x</td>
<td>x</td>
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<td>x</td>
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<td>x</td>
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<tr>
<td>C403</td>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Slump loss</td>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Air loss</td>
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<td>x</td>
<td>x</td>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>
B.2: Additional information

Table 7: Tabular results of slump and air loss testing.

<table>
<thead>
<tr>
<th>BS Mix</th>
<th>Slump</th>
<th>Air</th>
<th>Time</th>
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<tbody>
<tr>
<td></td>
<td>8.25</td>
<td>5.6%</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>5.1%</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.8%</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>6.5</td>
<td>4.3%</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>6.25</td>
<td>4.3%</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>5.5</td>
<td>4.3%</td>
<td>53</td>
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<th>Air</th>
<th>Time</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>6</td>
<td>6.8%</td>
<td>0</td>
</tr>
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<td></td>
<td>3.25</td>
<td>5.8%</td>
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<tr>
<td></td>
<td>3.25</td>
<td>5.5%</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>3.75</td>
<td>5.2%</td>
<td>37</td>
</tr>
<tr>
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<td>3.5</td>
<td>5.2%</td>
<td>47</td>
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</thead>
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<td></td>
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<td>6.6%</td>
<td>50</td>
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<td>5.0%</td>
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<tr>
<td></td>
<td>8</td>
<td>4.5%</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>4.1%</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>6.75</td>
<td>4.2%</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>6.75</td>
<td>4.6%</td>
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<table>
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<tr>
<td></td>
<td>7.5</td>
<td>6.4%</td>
<td>14</td>
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<td>6.1%</td>
<td>23</td>
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<td>4.5</td>
<td>5.6%</td>
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<tr>
<td></td>
<td>3.75</td>
<td>5.3%</td>
<td>54</td>
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Table 8: Tabular results for RSTM testing

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<td>1.791</td>
<td>0.379</td>
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<td>0.654</td>
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<td>1.345</td>
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<td>1.743</td>
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74
Figure 45: Elastic modulus of concretes tested (CTL).
Figure 46: Results for linear shrinkage testing (CTL).