Extended Life Concrete Bridge Decks Utilizing Internal Curing to Reduce Cracking

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16. Abstract

With the ongoing concern about premature cracking of concrete bridge decks that reduces the service life of bridges and results in increased maintenance and replacement costs, this work aimed at assessing the benefits of using lightweight fine aggregate (LWFA) in concrete mixtures to assist the Ohio Department of Transportation (ODOT) in preparing a specification to increase the probability of achieving crack-free, long-lasting bridge decks.
A laboratory testing program was developed and conducted to address concerns specific to bridge decks. The laboratory testing led to the recommended mix design for implementation on a bridge construction project in Ohio. Construction of the two bridge decks involved a control using a conventional mix design and the other containing slag cement and internal curing (IC). The decks were instrumented and load tested shortly after construction and inspected one year after placement. A life-cycle cost analysis was also conducted to assess feasibility and affordability of the recommendations.

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INTRODUCTION

There is an ongoing concern about premature cracking of concrete bridge decks. Bridge deck cracking reduces the service life of bridges and results in increased maintenance and replacement costs. Ideally, a deck should last as long as the support structure with minimal maintenance. However, bridge decks typically need to be replaced two or more times before the structure needs to be replaced. Such a differential in element service life is inefficient and costly and can cause unnecessary, repeated work zone hazards.

Recent research has reported the benefits of using lightweight fine aggregate (LWFA) in concrete mixtures to provide internal curing (IC). These benefits include reduced risk of early-age cracking and improved durability. The theory behind using LWFA is that the pre-soaked fine particles act as reservoirs to provide water for hydration without influencing the water-to-cementitious materials (w/cm) ratio. Uniformly distributed LWFA particles in a mixture provide a benefit to the whole volume of the element rather than only the surface, as is the case with conventional curing. This improvement in uniformity and hydration is intended to improve durability while reducing the effects of dimensional change, such as shrinkage. These benefits are most useful in bridge decks that are exposed to aggressive environments and are at a high risk of cracking (Taylor et al. 2016).

This report describes work aimed at assisting the Ohio Department of Transportation (ODOT) in preparing a specification to increase the probability of achieving crack-free, long-lasting bridge decks.

BACKGROUND

Public agencies face the challenge of designing and constructing long-lasting bridge decks. Generally, the deck is subject to cyclic loading, not only from traffic, but also from daily and seasonal variations in weather. Some failure mechanisms in a deck include the following:

- Cracking, particularly at the surface
- Cold weather damage, including salt scaling
- Polishing of the surface, leading to a loss of skid resistance
- Corrosion of reinforcement

These failure mechanisms may be cumulative, and the measures taken to deal with one mechanism may be contraindicated for another. For instance, while a strong and impermeable concrete mixture is desirable to address the applied loads and environmental concerns, such a mixture is at a greater risk of shrinkage cracking.

An additional complication is that a "perfect" mixture delivered to a site is still subject to construction practices such as insufficient curing that may compromise its quality.

The following innovative solutions have been demonstrated to have the potential to reduce the risk of cracking while enhancing durability.

Internal Curing

According to the American Concrete Institute (ACI), "[i]nternally cured concrete uses pre-wetted absorptive materials that contain moisture. The absorbed moisture is released as the internal humidity of the concrete drops below 100 percent to enhance and maximize the hydration of cement" (ACI 308 2013). The provision of additional water at the surface after setting may be an inadequate approach to hydration because the water is unlikely to penetrate far enough into the concrete, leaving most of the volume untreated (Bentz et al. 2007).

IC practice in the US is to include pre-saturated LWFA in the mixture because the aggregate can be uniformly distributed throughout the mixture, which thus maximizes the hydration benefits throughout the whole volume rather than merely around discrete large particles. The use of superabsorbent polymers (SAP) is popular in Europe. These products are based on polyacrylamide, a thermoset polymer with chemical crosslinks that prevents the dissolution of the polymer in water (Siriwatwechakul et al. 2012).

The amount of LWFA required for IC is based on the need to provide about 7 lbs of water for every 100 lbs of cementitious material content. This equates to about 20 to 30 percent replacement of fine aggregate by volume.

The reported benefits of IC include the following:

- Reduced mixture density of about 100 lbs/yd^3 with the inclusion of 20 to 30 percent LWFA by volume (Byard et al. 2012)
- Increased hydration, and therefore increased strength (Geiker et al. 2004)
- Reduced modulus of elasticity due to the lower stiffness of the LWFA (De la Varga et al. 2012)
- Reduced moisture gradients throughout the thickness of the slab, and therefore reduced warping (Bentz and Weiss 2011)
- Elimination of autogenous shrinkage and early swelling before drying shrinkage begins, which compensates for the later drying shrinkage (Bentur et al. 2001, Craeye et al. 2011)
- Reduced cracking due to lower shrinkage, modulus of elasticity, coefficient of thermal expansion, and temperature peak; increased strength and toughness; and improved hydration (Byard and Schindler 2010, Schlitter et al. 2010)
- Improved permeability, including improved chloride penetration; sorptivity; air permeability; and conductivity (Zhutovsky and Kovler 2012, Schlitter et al. 2010, Bentz 2009)

A number of bridge decks using IC have been constructed in the US. The following observations have been made: reduced plastic and drying shrinkage cracking in a pavement project in Texas (Byard and Ries 2012), good performance and negligible differences in placing and finishing compared to traditional mixtures in a bridge deck construction project in Colorado (Bates et al. 2012), reduced deck cracking in New York state (Streeter et al. 2012), a lower chloride diffusion coefficient in a bridge deck in Indiana (Schlitter et al. 2010), and improved permeability in a bridge deck in Iowa (Taylor et al. 2016).

The in-place cost of the mixture may increase by about 4 percent, but the service life and life cycle analysis are substantially improved over that of an equivalent mixture without IC (Taylor et al. 2016, Castrodale 2014, Cusson et al. 2010).

Shrinkage Reducing Admixtures

The use of shrinkage reducing admixture (SRA) is another approach that reportedly reduces the cracking risk dramatically. Adding shrinkage reducing chemical admixtures to the mixture can reduce stresses by reducing the surface tension in the pore solution, which tends to pull pores closed when water is removed from the capillaries (Kosmatka and Wilson 2016). However, this product normally influences other mixture properties such as air entrainment. It also carries a cost premium, and an evaluation should be made to determine whether the benefit is sufficient to warrant the premium.

Mixture Proportioning

Because a significant contributor to the risk of cracking is the amount of thermal- and moisture-related shrinkage occurring in the mixture, careful proportioning of mixtures with well-graded aggregates that help minimize the paste content along with a selection of supplementary cementitious materials (SCMs) will help reduce cracking. The following guidelines are likely to minimize the risk of cracking (Darwin et al. 2011, Taylor and Wang 2014, Taylor et al. 2012):

- Maximum paste content below 25 percent (by volume), achieved by requiring an optimized aggregate gradation and a maximum cement content of 540 lbs/yd^3
- Maximum w/cm ratio of 0.44 to 0.45
- Moderate (~4,000 psi) rather than high compressive strength
- Air content in the range of 6.5 percent to 9.5 percent
- Use of slag cement
- Use of internal curing
- Maximum aggregate size of 1 in.

Workmanship

The following activities can help reduce cracking risks:

- Control batch temperature before and after placing and during weather events
- Control system moisture state
- Avoid activity that adds additional water to the system until after final set
- Maintain a good pre-saturated condition for LWFA during storage and batching
- Apply a minimum amount of finishing and avoid excessive vibration

LITERATURE REVIEW

Overview

There is an ongoing concern about premature cracking of concrete bridge decks despite continued efforts to prevent it. Such cracking reduces serviceability, shortens a bridge's lifetime, and results in increasing maintenance and replacement costs. Ideally, a deck should last as long as the support structure with minimal maintenance. However, the current state-of-the-practice typically results in a bridge deck needing to be replaced two or more times before the superstructure needs replacing. Such a differential in elemental service life is inefficient and costly and can cause unnecessary, repeated work-zone hazards.

A challenge is that methods used to reduce concrete permeability, and thereby increase bridge deck lifetime, also tend to increase the risk of concrete deck cracking. The situation becomes even more complex when considering how these "improved impermeability" concrete mixes interact with a bridge system that is already relatively stiff. However, innovative methods, such as IC, materials such as shrinkage-reducing admixtures, and a better understanding of mixture proportioning appear to offer a potential means of moving toward this ideal.

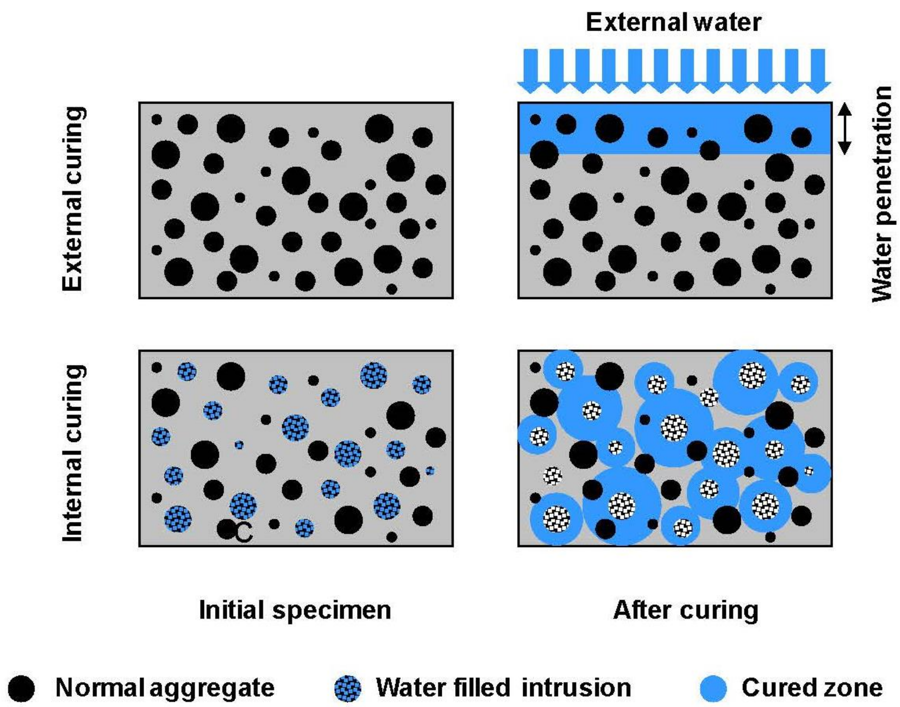
Following is a review of the literature on this topic. The bulk of the discussion is focused on internal curing because this is a relatively new topic. For the more mature aspects such as shrinkage-reducing admixtures and mixture control, the review relied on other rigorous reviews developed by experts in the field. The literature review ends with some recommended guidelines based on the discussion.

Internal Curing

Introduction

Curing is provision of sufficient heat and moisture to promote sufficient hydration of a concrete mixture. Traditional methods of controlling the moisture state have been to limit water loss from free surfaces, or to provide additional water from the exterior. The former is of limited benefit in low w/cm systems (w/cm ratio less than about 0.40) because the water content is insufficient to hydrate all of the cement present—leading to so-called desiccation, increasing autogenous shrinkage, and setting up stress profiles within the element. Providing extra water from the exterior is only useful to the outer inch (or less) because the water cannot penetrate any farther into the system during curing. Providing extra water may also set up moisture profiles through the section encouraging warping of flat elements.

The concept behind internal curing is to deliver added curing water throughout the system, without compromising the w/cm ratio at the time of mixing (see Figure 1).



Castro et al. 2010

Figure 1. Conceptual illustration of the differences between external and internal curing

Internal curing can be achieved by including particles that contain free water into the mixture, but that water is only released when the relative humidity (RH) of the pore system drops below a given value. Common approaches are to include relatively small amounts of lightweight fine aggregate (LWFA) or super-absorbent polymers into the mixture.

Fine particles are preferred because each particle influences a zone a fixed distance away from the particle's surface; thus, uniform distribution of small particles closer together protects a greater paste volume than a similar volume of large particles further apart (De la Varga et al. 2012, ACI 2013).

The remainder of this chapter discusses reported research on internal curing and how it may applied to concrete bridge decks.

Background

"Internally cured concrete uses pre-wetted absorptive materials that contain moisture. The absorbed moisture is released as the internal humidity of the concrete drops below 100 percent to enhance and maximize the hydration of cement." (ACI 2013).

The cementitious combinations used in concrete today tend to hydrate faster than older cement systems due to changing chemistry and fineness. In addition, w/cm ratios are somewhat lower as engineers seek to obtain increased strength and potential durability, all at a greater rate.

Associated with these changes is an increasing risk of desiccation within the microstructure because all of the mixing water is consumed early in the hydration process. This desiccation tends to increase autogenous shrinkage because the pores are drained, leading to increased risk of early cracking, along with increased permeability (ACI 2013).

Traditional curing approaches such as sealing the surface to prevent evaporation may not be sufficient to prevent desiccation, because there is just not enough water in the mixture. Provision of additional water at the surface after setting is also ineffective because the water is unlikely to penetrate far enough into the concrete, leaving most of the volume untreated (see Table 1).

Table 1. Distance of water travel from surfaces of internal reservoirs

Hydration age	Estimated travel distance for water	
days	in.	mm
Less than 1	0.84	21.4
1 to 3	0.15	3.90
3 to 7	0.04	0.98
More than 28	0.01	0.25

Source: After Bentz et al. 2007

One approach to providing sufficient water for hydration, without raising w/cm ratio values, is to provide reservoirs of water throughout the matrix. One means of doing this is to include a given amount of pre-wetted LWFA in the mixture.

The aggregate should have a pore structure fine enough that the water is not released prematurely, but coarse enough that it will hold enough water and release it when needed. The aggregate should be the size of fine aggregate so that the volume of paste affected by it is maximized. Details of these requirements are discussed below.

While the idea is not new, having been discussed by Klieger in 1957, it is relatively recent that the idea has come to find acceptance for use in bridge applications.

A review by Hoff describes research in several countries around the world on the topic (Hoff 2002). While much of the early interest was in so-called high-performance systems with very low w/cm ratios, more recent work is starting to demonstrate that there are benefits for more every-day mixtures.

Materials

Type

The most common means of providing internal curing in the US is by the use of expanded LWFAs, likely because of their relatively low cost and availability in most of the country. Less common in the US but readily available in Europe are so-called super-absorbent-polymers (Craeye et al. 2011, Wyrzykowski et al. 2012, Siriwatwechakul et al. 2012, Mechtcherine and Reinhardt 2012). Other materials are reported in the literature, including eucalyptus pulp (Jongvisuttisun et al. 2012), jute fiber (Ozawa and Morimoto 2012), and ceramic waste (Suzuki et al. 2009), that have not found acceptance in the marketplace.

Characteristics

Four fundamental requirements define the quality of a product: moisture content, desorption, fineness, and uniformity. Each of these is discussed in this section.

Work by Bentz et al. (2005) developed a method of determining the amount of water required to be released by the product:

$$M_{LWA} = (C_f \times CS \times \alpha_{max}) / (S \times \phi_{LWA})$$

where:
M_{LWA} = mass of (dry) lightweight aggregate (LWA) needed per unit volume of concrete
C_f = cement factor (binder content) for concrete mixture
CS = chemical shrinkage of cement
α_{max} = maximum expected degree of hydration of cement
S = degree of saturation of LWA
ϕ_{LWA} = absorption (desorption) capacity of LWA

In general, the solution of this expression leads to a need for about 7 pounds of water for every pound of cement (Barrett et al. 2014). This means that the amount of aggregate needed can be determined based on how much water will be released by the material—typically about 95 percent of that absorbed.

Desorption is the ability of the material to release water back into the matrix when the RH of the pore system drops below about 95 percent. Differences between efficient and less efficient products are illustrated in Figure 2.

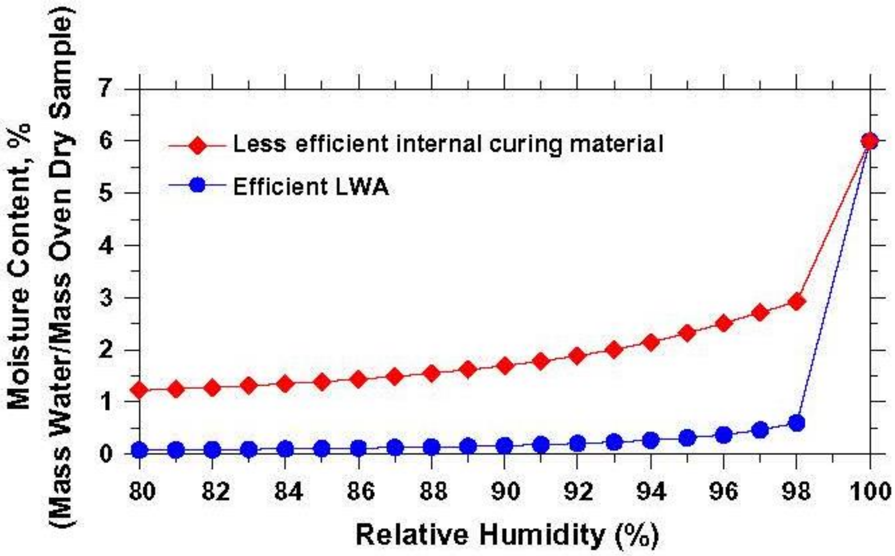


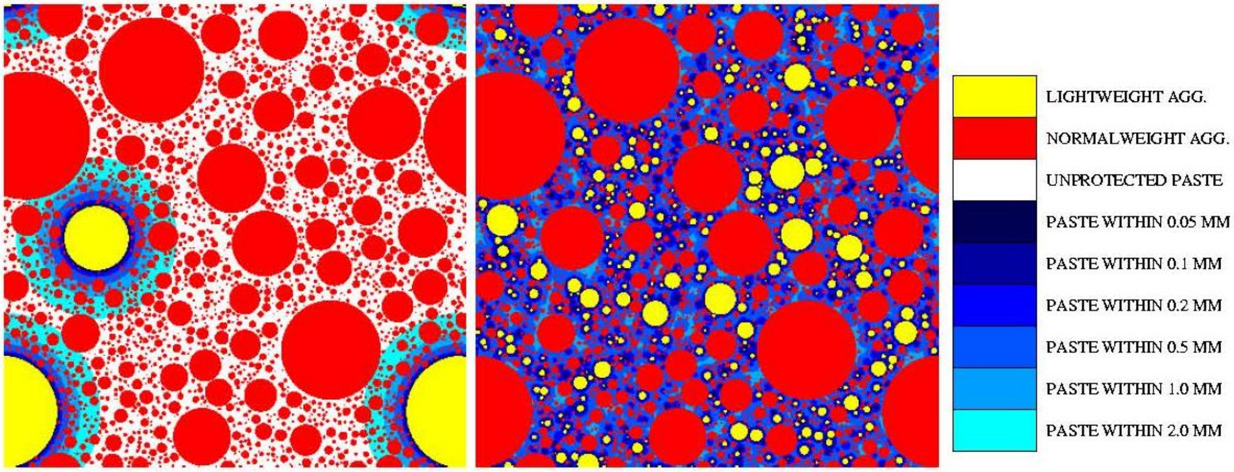
Figure 2. Example of more- and less-efficient desorption behavior

The pore system of the product should be coarser than that of the hydrated cement paste, so that water is removed by capillary action when the paste pores begin to dry out. Experimental work by Schlitter et al. (2010) confirmed that water did not leave the LWA until set had occurred and a relatively fine pore system was established in the paste.

Work by Castro et al. (2011) indicated that the range of aggregate products they tested all fell into a similar band of desorption performance.

Several researchers have reported that water coming out of the particles can only travel a limited distance (in the order of millimeters) into the paste (Table 1) (Kovler et al. 2004, Bentz 2007, Bentz et al. 2007, Schlitter et al. 2010).

It is therefore desirable to have the reservoirs close together to ensure that all or most of the paste is within reach of the needed water as illustrated in Figure 3.



After Bentz et al. 2005 with images from Barrett et al. 2014

Figure 3. Conceptual illustration showing protected paste volume of two mixtures with similar lightweight aggregate replacement volumes: coarse aggregate replacement (left) and fine aggregate replacement (right)

Like an air-void system, a large number of small particles close together is preferred over a small number of large particles. Finer particles have been shown to influence shrinkage more, likely because of the volume of paste protected (Liu and Zhang 2010).

Because approximately 5 percent of the water in the aggregate may never be released, there will be a difference between the absorbed water and that available for internal curing. This is influenced by the particle sizes, meaning that uniformity of gradation is important (Wei et al. 2014).

Other uniformity factors that should be monitored include the desorption of the product, and most critically, the moisture content at the time of batching.

Specifications

LWFAs for use in internal curing applications are specified in ASTM C1761. Parameters addressed in this specification include the following:

- Organic impurities, which may affect strength
- Staining, which will affect color
- Loss-on-ignition, which may affect water demand
- Clay lumps and friable particles, which will affect water demand
- Grading, as discussed above
- Bulk density, which will affect absorption
- Water absorption, as discussed above
- Desorption, as discussed above

There is no US specification that addresses super-absorbent polymers, but additional information can be obtained from Mechtcherine and Reinhardt (2012).

Effects

The effects of including internal curing materials in a mixture are discussed in the following sections.

Fresh Properties

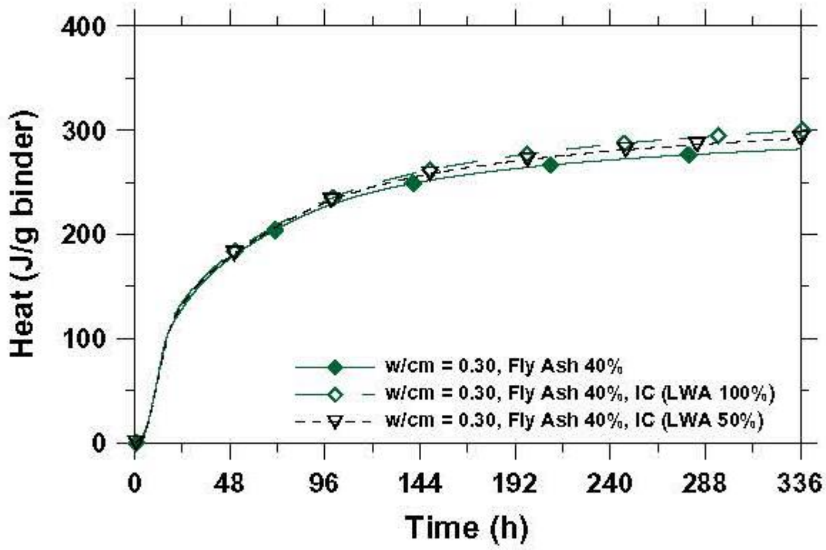
Workability: Little has been reported in the literature regarding changes to workability except the notes from field demonstrations discussed in a later section. Experience by the authors has been that workability is largely unaffected, while finishing and placing crews on a bridge deck and a pavement did not notice when the mixture changed from conventional to IC containing LWFA, and back to conventional fine aggregate.

Density: It is to be expected that the density of a mixture will decrease by about 100 lb/yd^3 with the inclusion of 20 to 30 percent LWFA by volume (Byard et al. 2012).

Hardened Properties

Hydration: The fundamental idea behind internal curing is to promote hydration of the cementitious system, particularly for those with w/cm ratios less than about 0.42 that are at risk of desiccation.

Enhanced hydration has been demonstrated by observing heat of hydration (Byard et al. 2012, Craeye et al. 2011), most notably several days after mixing (De la Varga et al. 2012) (see Figure 4).



Reprinted from De la Varga et al. 2012 with permission from Elsevier.

Figure 4. Effect of changing internal curing replacement level

The increased hydration is also observed in increased strength (Geiker et al. 2004), which is discussed below, and reduced absorption and conductivity (Henkensiefken et al. 2009a).

Strength and Modulus of Elasticity

The effect of internal curing on strength generally appears to be favorable (Byard and Schindler 2010, Byard et al. 2012), especially at later ages (De la Varga et al. 2012). However, some researchers have reported only limited (Bentur et al. 2001, Liu and Zhang 2010, Cusson and Hoogeveen 2008) or indeed negative (Paul and Lopez 2011, Sahmaran et al. 2009, Schlitter et al. 2010) effects. This may not be surprising because strength is a relatively insensitive test that is strongly influenced by other parameters such as w/cm ratio.

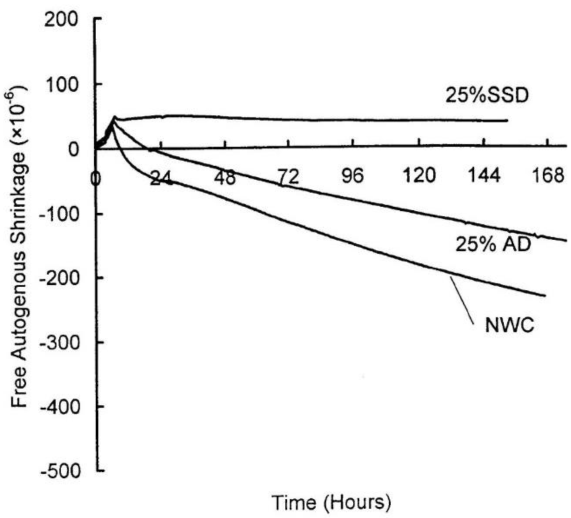
Modulus of elasticity is reportedly reduced (De la Varga et al. 2012), likely due to the lower stiffness of the LWA. Reduced stiffness and increased creep may be considered a benefit, especially in unreinforced slabs, because this will reduce the risk of random cracking.

Craeye et al. (2011) reported that use of super-absorbent polymers reduced mechanical strength and modulus of elasticity, along with higher creep deformation.

Shrinkage

Despite the potential negative effects on strength, the primary purpose for using internal curing is to reduce autogenous shrinkage and the associated warping and cracking. Strength can always be controlled in a mixture by adjusting the w/cm ratio and cementitious system.

Many researchers have reported significantly reduced shrinkage and the associated stresses with the inclusion of both types of internal curing materials into mixtures. Full (Bentur et al. 2001, Byard and Schindler 2010) or partial (Craeye et al. 2011, Geiker et al. 2004, Paul and Lopez 2011, Wei et al. 2014, Schlitter et al. 2010) elimination of autogenous shrinkage has been reported for high performance and conventional mixtures. Findings from many of the authors have shown an initial swelling before shrinkage begins, which partially compensates for the later shrinkage that does occur (Bentur et al. 2001, Craeye et al. 2011).



AD=air-dry, NWC=normal-weight concrete, SSD=saturated-surface-dry

Reprinted from Bentur et al. 2001with permission from Elsevier.

Figure 5. Comparison of free autogenous shrinkage between normal-weight concrete and concretes in which 25 percent by volume of total normal-weight aggregate was replaced by lightweight aggregates

The reduced autogenous shrinkage may be attributed to the water from the lightweight materials keeping the internal RH of the system high (Henkensiefken et al. 2009b). Increasing amounts of LWA increase the benefit.

In addition to reducing autogenous shrinkage, inclusion of LWFA has been demonstrated to reduce drying shrinkage, also helping to reduce cracking risk and warping effects (Sahmaran et al. 2009, Schlitter et al. 2010).

Cracking

Closely tied to reduced shrinkage is the risk of cracking. In general, mixtures that contain internal curing materials exhibit significantly lower cracking risk (Craeye et al. 2011) due to the following:

- Reduced shrinkage (autogenous and drying) (Byard and Schindler 2010, Henkensiefken et al.

2009b, Schlitter et al. 2010)

- Reduced modulus of elasticity (Cusson and Hoogeveen 2008, Henkensiefken et al. 2009b)
- Increased strength and toughness (Byard and Schindler 2010, Byard et al. 2012, Henkensiefken et al. 2009b)
- Reduced coefficient thermal expansion (Byard and Schindler 2010)
- Reduced temperature peak (Byard and Schindler 2010)
- Improved hydration

Permeability

Associated with the longevity of a concrete structure is the ability of the mixture to prevent the penetration of deleterious fluids that lead to freeze-thaw distress or chemical attack such as steel corrosion. Concrete permeability is strongly affected by the type of binder and the w/cm ratio, with decreasing water content leading to reduced percolation of voids in the paste. However, decreasing the w/cm ratio also increases the risk of desiccation as discussed above. Another factor that controls permeability is the degree of hydration. All of these factors can be improved by the use of internal curing. A number of measurement approaches have been reported:

- Chloride penetration using silver nitrate ($AgNO_3$) is reduced between 15 and 25 percent depending on age of exposure (Bentz 2009)
- Sulfate expansion is reduced (Bentz et al. 2014)
- Sorptivity is reduced by up to 46 percent (Henkensiefken et al. 2009a, Liu and Zhang 2010, Schlitter et al. 2010) although Zhutovsky and Kovler (2012) reported a small increase, likely due to the increased porosity of the aggregate
- Air permeability is reduced by 50 to 60 percent (Zhutovsky and Kovler 2012)
- Conductivity is reduced (Henlensiefken et al. 2009a, Schlitter et al. 2010)

According to Schlitter et al. (2010), freeze-thaw performance was not affected, likely because the low w/c ratios of the mixtures dominated the factors that affect performance.

Design

A review by Rao and Darter (2013) investigated the potential effects of internal curing, including those based on inputs used in the AASHTOWare Pavement ME Design procedure.

Parameters such as strength, modulus of elasticity, coefficient of thermal expansion, and unit weight all pointed toward extended life for equivalent sections. A reduction in cracking as discussed above also will provide a major benefit.

Additionally, the improvement in durability from the reduction in permeability will provide a benefit, although this is not a parameter considered in most structural design procedures.

While the results point to the potential to build thinner elements, it is likely conservative at this stage, until more field data are collected, to use conventional thicknesses and derive increased sustainability from the increased life.

Construction

Proportioning

Based on the work by Bentz (2007), the rule of thumb is that about 7 lbs of water is needed per 100 lbs cementitious material in a mixture. An LWFA that has a grading similar to fine aggregate and sorption/desorption values required by ASTM C1761 should therefore replace about 30 percent of the fine aggregate by volume (i.e., about 250 to 300 lb/yd^3).

Storing

To provide benefit, the LWFA should be internally wet, but surface dry for batching. Because batching is by weight, and products can absorb up to 20 percent of their mass, it is essential that the moisture state of the material be continually monitored and the batch weight adjusted accordingly. It is not essential that the product be saturated.

Water absorbed into the aggregate from the paste is readily delivered back to the paste when drying occurs, but may effectively reduce the w/cm ratio (Golias et al. 2012) and degrade the workability.

The New York State DOT (NYSDOT) requires that the aggregate stockpile be sprinkled for 48 hours and then allowed to drain for 12 hours before use.

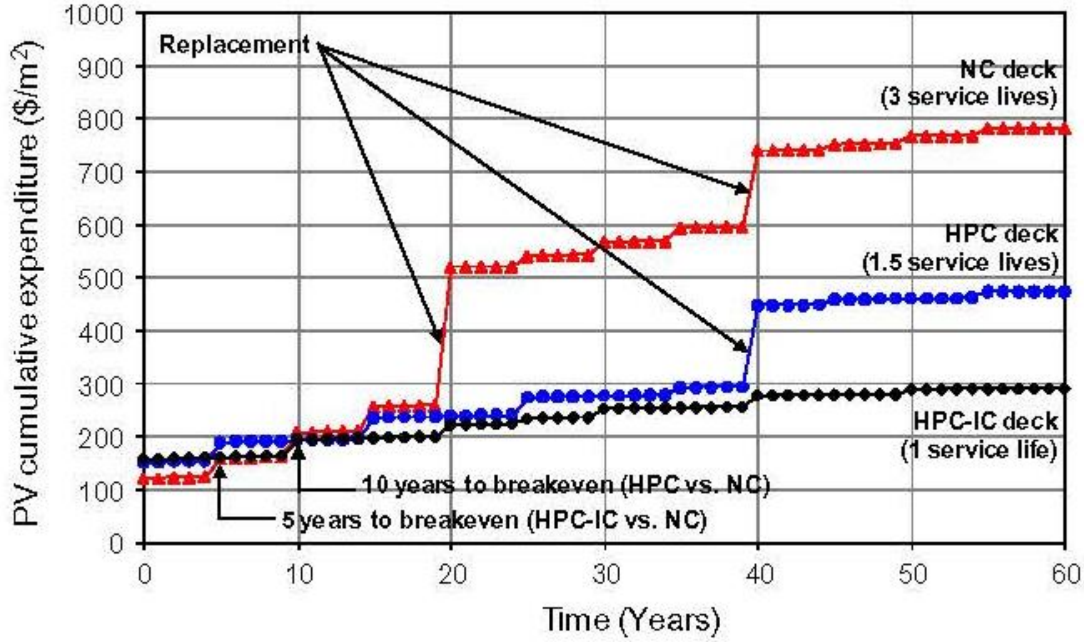
Mixing, Transporting, and Finishing

No special precautions are needed with handling, placing, or finishing a mixture containing internal curing materials.

Cost

There is a premium associated with the use of internal curing. The following section discusses the balance of benefit and cost as reported in the literature.

Modeling by Cusson et al. (2010) reported that a high-performance mixture for internal curing used on a bridge deck could reduce present value by 38 percent compared to the equivalent mixture over a period of 60 years. Alternatively, the same information could be presented as the internal curing mixture providing a 63-year service life compared to 40 years for the equivalent mixture, attributed to reduction of initial cracking and chloride diffusion (see Figure 6). The premium to use the product was a 4 percent increase in in-place cost.



HPC=high-performance concrete, HPC-IC=internally-cured high-performance concrete, NC=normal concrete, PV=present value

Figure 6. Comparison of present value cumulative expenditures for three bridge deck alternatives

Daigle et al. (2008) reported that reduction in cracking could conservatively be expected to provide an extra 10 years of service life in a high-performance concrete bridge deck, due to the absence of initial cracking.

The material cost increase is usually about $10/yd^3 along with some extra costs for storage and handling (Castrodale 2014).

Case Studies

A number of case studies have been reported. Observations include the following:

- Plastic and drying shrinkage cracking were significantly reduced in an intermodal terminal pavement project in Hutchins, Texas (Byard and Ries 2012).
- New York continues to use IC mixtures in bridge decks because of reduced deck cracking (Streeter et al. 2012).
- Testing on bridge decks in Monroe County, Indiana, demonstrated a lower chloride diffusion coefficient than plain concrete up to 91 days of age.

Specifications

A review by Masten (2015) of 27 state departments of transportation (DOTs) is summarized in Table 2.

Table 2. Survey responses for experience trying internal curing

State	Response
Colorado	www.codot.gov/programs/research/pdfs/2014/ic.pdf/view.
Florida	We have a current research project ongoing and is providing very promising data for its use in pavement and bridge decks. We hope to allow its use in the near future.
Illinois	IDOT is currently drafting a specification for internal curing with lightweight fines, and currently only experimented with it via various university research projects concerning early bridge deck crack mitigation (Mondal and Hindi) and pavement slab mechanics (Roesler).
Illinois Tollway	www.illinoistollway.com/documents/10157/90097/High-Performance+Concrete+for+Bridge+Decks-Final+Report.pdf The Tollway is funding research to use internal curing for CRCP. Demonstration sections will be built on the Tollway mainline in 2016.
Indiana	INDOT currently has an Experimental Feature Study for 6 bridges built with IC-HPC in 2013, 2014, and 2015. The specification for IC-HPC only exists as a unique special provision. The Department plans on submitting and interim report to the FHWA later this year. INDOT not only plans to use IC-HPC in bridge decks, but in deck railing and reinforced concrete bridge approach slabs as well.
Iowa	Field application for bridge deck was half of deck on a county project.
Kansas	Dave Meggers presentation.
Louisiana	Placed 65 cuyd in conjunction with a local municipality on two of five spans of a slab span bridge. 600+yd^3 are expected to be placed on a LA DOTD bridge project in April 2015.
Michigan	Tried internal curing of ultra-fast set pavement repair mixture via research at the University of Michigan.
Minnesota	Testing is in early stages. The original intention was from a lightweight aggregate standpoint and now are exploring for internal curing purposes.
Missouri	Later this year, planning to batch concrete utilizing lightweight sand to determine fresh and harden concrete properties. Currently looking for a bridge project to utilize internal curing.
Ohio	ftp://ftp.mdt.mt.gov/research/LIBRARY/FHWA-OH-2007-06.PDF (Delatte, Mack, Cleary 2007).
Texas	Small test section placed with lightweight aggregate in Dallas Dist. We were not looking at internal curing at the time, just use of lightweight aggregate.
Utah	Sharing experience and report at NCC 2015.

Source: Masten 2015

Requirements by the NYSDOT and Louisiana Department of Transportation and Development (LA DOTD) included the following:

- LWFA material to comply with ASTM C1761
- Aggregate to be stockpiled and managed to maintain a uniform moisture state
- Aggregate to be soaked by sprinkler for 48 hours followed by 12 to 15 hours drying

Summary

Internal curing, properly conducted using appropriate materials, has the potential to improve bridge deck longevity due to the following:

- Improved ability to resist penetration of aggressive fluids
- Reduced shrinkage and associated cracking
- Reduced stiffness and associated cracking

Shrinkage-Reducing Admixtures

Introduction

Another approach to reducing cracking risk in a mixture is to use chemical approaches to reducing the shrinkage. Such chemicals, SRAs, have been available since the early 1980s and are discussed in this chapter.

Background

SRAs were first patented in 1982 in Japan and 1985 in the US (Nmai et al. 1998). SRAs have found acceptance in the construction of slabs-on-grade helping to grow the market in flat slabs for industrial uses. SRA use in bridge decks has also been growing over time.

The mechanism behind their effectiveness is based on changing the surface tension in the pore solution. As water is removed from capillaries due to drying, the surface tension tends to pull the pores closed, leading to shrinkage of the system. By reducing this surface tension, the amount of movement is reduced (Kosmatka and Wilson 2016).

Materials

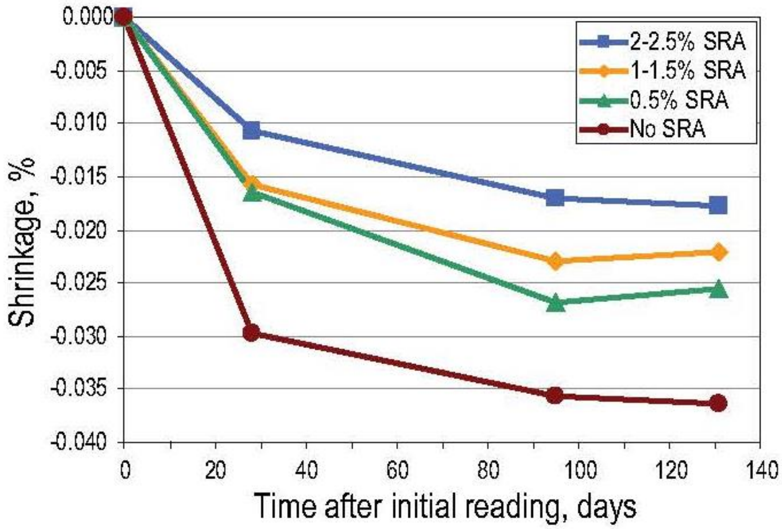
SRAs are based on polyoxyalkylene alkyl ether or propylene glycol (Nmai et al. 1998, Kosmatka and Wilson 2016). They are low viscosity, water-soluble liquids that reduce surface tension in the pore solution.

Products available in the US are required to comply with the requirements of ASTM C494 Type S.

Effects

General reported trends on the effects of SRAs include the following:

- A reduction in surface tension when compared to distilled water (Bentz et al. 2001, Bentz 2006). This effect increases with dosage up to a plateau (Rajabipour et al. 2008).
- Reduced autogenous and drying shrinkage leading to a reduced risk of cracking (Bentz 2006, Folliard and Berke 1997, Mora-Ruacho et al. 2009, See et al. 2003, Weiss et al. 1998). Shrinkage has been reported to be reduced by up to 50 percent at 12 weeks (see Figure 7) (Nmai et al. 1998, Maggenti et al. 2013).
- Increased pore RH under sealed conditions (Bentz et al. 2001) associated with a reduced evaporation rate (Mora-Ruacho et al. 2009).
- A small increase in porosity (Lopes et al. 2013), which may be expected because pores are not closing as much under surface-tension forces.
- A decrease in the drying rate of cement pastes (Bentz et al. 2001, Bentz 2006). A potential side effect of this is that the amount of water available to freeze in the pores will be increased, potentially increasing the risk of frost-related damage if the air-void system is inadequate (Bentz 2006).
- Little change in workability (Nmai et al. 1998).
- Air content may be decreased (Lopes et al. 2013) or unaffected (Nmai et al. 1998).
- Time of set may be delayed (Lopes et al. 2013, Nmai et al. 1998), particularly when used with a superplasticizer (Brooks et al. 2000).
- No change in hydration rates (Bentz et al. 2001), although reduced solubility of alkalis has been reported that may slow hydration and early strength gain, unless addition of the SRA to the mixture is delayed (Rajabipour et al. 2008).
- The data on strength are mixed. No change in 28-day compressive strength has been reported (Bentz et al. 2001, Nmai et al. 1998), as has a reduction in early strength for equivalent mixtures (Folliard and Berke 1997, Lopes et al. 2013).
- Increased pore solution viscosity leading to reduced diffusion and sorption of aggressive ions (Bentz 2006, Sant et al. 2010).



Source: Unpublished Caltrans data used in internal presentations from Maggenti et al. 2013

Figure 7. Comparison of concrete shrinkage over time for mixtures with and without shrinkage-reducing admixtures

Construction

No special precautions are needed other than to ensure that the air-void system is satisfactory. Slightly delayed setting may influence saw-cutting and form-removal times.

Case Studies

A number of case studies have been reported (Nmai et al. 1998, Bae 2004, Maggenti et al. 2013). Observations include the following:

- Reduced shrinkage and shrinkage-related cracking
- Reduced warping in slabs on grade

Summary

SRAs also have the potential to improve bridge deck longevity due to the following:

- Reduced shrinkage and associated cracking

Mixture Proportions

Introduction

This chapter provides a brief review of other mixture-related factors that will contribute to performance of bridge decks. This topic has been studied in detail by several researchers (Wilson 2013, D'Ambrosia et al. 2013, and Darwin et al. 2011, for example) and their findings are summarized here.

Background

A critical aspect of bridge decks is that they should survive the environment they are exposed to for the intended lifetime. Key factors to help achieve that aim are that bridge decks should be impermeable and crack free. Both of these factors help to prevent ingress of aggressive solutions that may attack the aggregates, the paste, or the reinforcing system.

Unfortunately, some of the actions that reduce the risk of one issue (e.g., high permeability) will increase the risk of another (cracking). This chapter discusses how a compromise may be found to achieve the best possible system.

Cracking

Controlling cracking in concrete slabs is a balance of complex and sometimes mutually exclusive factors (Wilson 2013, D'Ambrosia et al. 2013, Darwin et al. 2011). Fundamental factors are discussed below.

Shrinkage

Increasing shrinkage, whether autogenous or drying, increases deformation and thus strains in a restrained system. Any activity that helps reduce shrinkage will reduce cracking risk. The following factors are critical (Taylor and Wang 2014):

- Paste content of the mixture: Increasing paste content leads to increasing shrinkage. Related to this are the water content, cementitious content, and the w/cm ratio, which all directly influence paste content.
- Supplementary cementitious material (SCM) selection and dosage (part of Table 3).
- Aggregate quality: In particular, the clay content because increasing clay will increase movement with moisture variations.
- The environment to which the concrete is exposed: The later that concrete drying begins, the lower the shrinkage, because more water is being consumed in hydration, and heightened strength increases the stress that can be carried.

Wang developed the summary of factors that influence shrinkage shown in Table 3.

Table 3. Summary of influence factors on concrete shrinkage

Categories	Factors	Specific factors	General findings and conclusions on shrinkage
Paste quality	Cement characteristics	Chemistry	Higher sulfate and gypsum content may reduce shrinkage
		Fineness	Through change of the rate of hydration and water demand, finer cement leads to higher shrinkage
	Supplementary cementitious material	Fly ash	Class F fly ash potentially decreases shrinkage; Class C fly ash increases shrinkage with increasing dosage
		Slag cement	Conflicting results, depending on paste content; overall, comparable to ordinary PCC mixtures
		Silica fume	Conflicting results; depending on w/cm, curing period, and paste content; limited replacement may reduce long-term shrinkage
		Ternary mixtures	Normally, adverse effect can be diminished (i.e., PC+slag cement+silica fume) or even reduced (i.e., PC + slag cement + F fly ash)
	Chemical admixtures	Superplasticizers	May increase concrete shrinkage with a given w/cm ratio and cement content; dependent on admixture chemistry
	Air content		No significant effect on the magnitude of drying shrinkage if it is less than 8 percent
Paste quantity	w/cm		Influence is relatively small with a constant paste content for w/cm ratio > 0.40
	Paste content		For a given w/cm ratio, shrinkage linearly increases with increasing paste content/volume
Other factors	Aggregate	Type	Influenced by mixing water demand and the stiffness of the aggregate
		Gradation	Indirect influence through the change of water demand and paste content; increased size results in decreased paste content, so decreased shrinkage
		Fines	Most fines increase shrinkage
	Curing	Duration	Increasing curing period reduces the amount of unhydrated cement, resulting in reduced overall shrinkage
	Environment	Relative humidity	Higher relative humidity leads to lower shrinkage
		Temperature	High temperatures accelerate moisture loss leading to higher shrinkage
		Wind	Similar to temperature effect
	Construction		Improper addition of water during finishing, and poor curing will increase shrinkage
	Geometry		Increasing the volume-to-surface area ratio decreases shrinkage

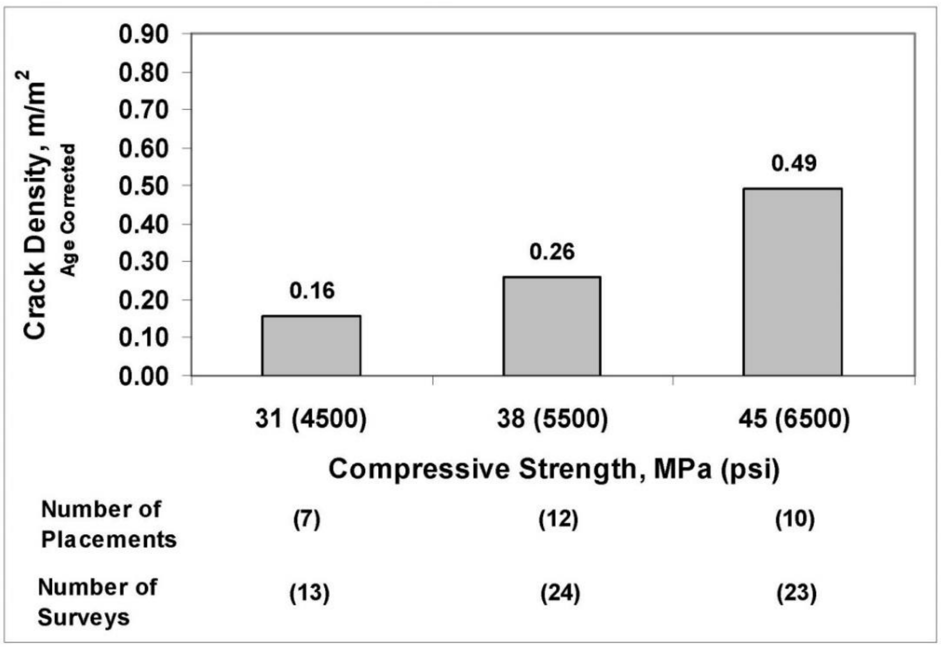
Source: National Concrete Pavement Technology Center from Taylor and Wang 2014

Stiffness

Increasing stiffness means that, for a given strain, stresses incurred will also increase. Increasing the w/cm ratio and air content, along with the use of LWFAs, reportedly all decrease stiffness.

Strength

Increasing strength helps the system carry stress, but the factors that increase strength also increase the negative factors such as shrinkage and stiffness. In general, no more than a structurally adequate strength is required. Consideration may be given to assessing strength at later ages, thus allowing SCMs to be included for other purposes (see Figure 8).



Lindquist et al. 2005

Figure 8. Mean crack density for monolithic bridges

Summary

Guidelines developed by Darwin et al. (2011) to minimize the risk of cracking include the following:

- Maximum paste content below 25 percent (by volume) by requiring an optimized aggregate gradation and a maximum cement content of 540 lb/yd^3 (320 kg/m^3)
- Maximum w/cm ratio of 0.44 to 0.45
- Moderate (~4000 psi) rather than high compressive strength
- Air content range of 6.5 to 9.5 percent
- 1-in. maximum sized aggregate

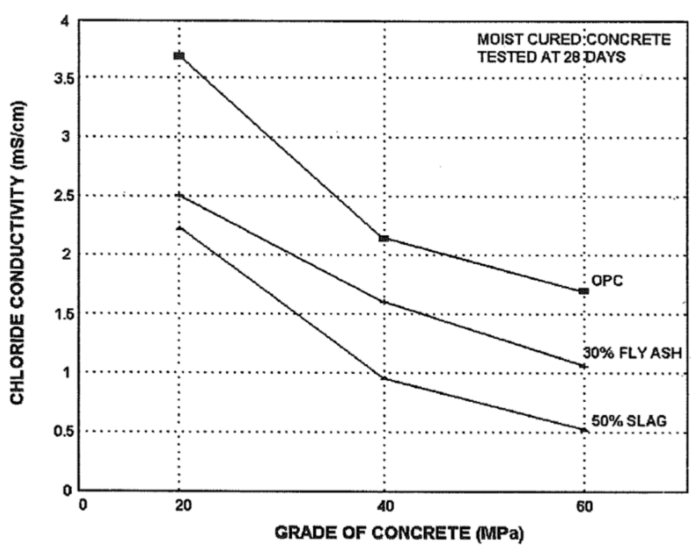
- Slump range of 1.5 to 3 in.
- Include slag cement
- Use internal curing

Permeability

The primary motivation behind seeking to reduce permeability of concrete mixtures is that most deterioration mechanisms involve the presence of water. Any approach that reduces the transportation of fluids will therefore help to protect the interior paste and the reinforcing steel.

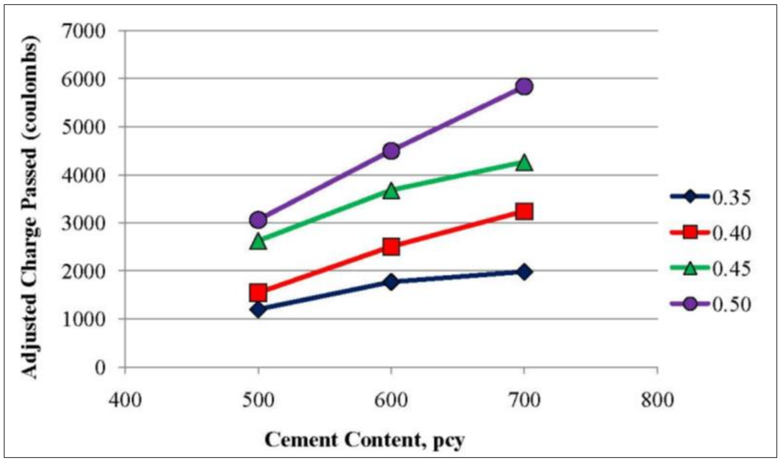
A review and experimental program by Yurdakul (2010) is summarized below. Key factors that influence permeability include the following:

- **w/cm ratio:** Figure 9 illustrates that increasing the w/cm ratio significantly raises the charge passed in an ASTM C1202 rapid chloride permeability (RCPT) test. The w/cm ratio values above about 0.4 significantly increase the probability of percolation of the pores, thus setting a reasonable target value.
- **Binder type:** Inclusion of appropriate SCMs will generally reduce permeability (see Figure 10). Ternary systems have been demonstrated to be effective at minimizing negative side effects of SCMs while making the most of their benefits (Taylor et al. 2012).
- **Binder content:** Figure 10 also illustrates that increasing binder content will increase permeability. This is because, in general, aggregate is less permeable than paste.
- **Degree of hydration:** Increasing hydration has the effect of reducing pore sizes and reducing the connectivity between them, thus reducing permeability. Effective curing is therefore essential to improve potential durability.



Alexander et al. 2001

Figure 9. Chloride conductivity results showing better performance in mixtures containing supplementary cementitious materials



Yurdakul 2010

Figure 10. Effect of cement content on rapid chloride penetration

Workmanship

Introduction

This chapter provides a brief review of workmanship-related factors that also influence durability of bridge decks and discusses how the factors may be addressed.

Temperature

Reviews have indicated that increasing ambient temperature at the time of concrete placement increases the risk of cracking (Darwin et al. 2010). Darwin has recommended that concrete temperature be limited to 70°F at time of placing. This can be achieved in hot weather by shading aggregate stockpiles, and using chilled water or ice in the mixture. Increasing amounts of SCM in the mixture will also help to reduce the heat rise due to hydration, thus reducing temperature profiles and movements in the system.

Storage, Batching, Handling, and Finishing

No special actions are required for conventional mixtures. However, LWFA for internal curing does require some additional attention. The fundamental aim is to ensure that the material is close to, but below, a saturated surface, dry moisture state when it is introduced into the mixer.

Management of the LWFA stockpile is just as, or more, critical as that with conventional aggregates. Moisture gradients must be avoided and care must be taken to avoid extracting material from too close to the ground, where it is likely to be too wet.

Sprinkling the stockpile for 48 hours is reportedly sufficient for most materials in use in the US. Materials that are too dry will tend to absorb water from the fresh paste, thus reducing the w/cm ratio, but also possibly affecting workability in a negative way. The moisture state of the material at time of weighing should be determined (ESCSI 2018) and consideration given to adjusting the water content of the mixture. This process must be approached with care because the aggregate will not absorb all of the potential water initially, so adding too much water to the mixture is a risk. (Darwin 2014, ACI 2013)

On the other hand, water on the surface of the material will add to the free water in the mixture, thus compromising the w/cm ratio, and should be avoided. The stockpile should be allowed to dry for about 12 hours before batching (ESCSI 2018).

It is recommended that two stockpiles be maintained: one in preparation and one in use (ESCSI 2018).

Batching, mixing, and transporting mixtures containing LWFA should be consistent with good practice as for all concrete (ACI 2013).

In some cases, finishing should be kept to a minimum to prevent pulling excess paste and water to the surface. Vibrating screed is the preferred method of finishing. A float may be used but should not be needed. It is recommended that texture be provided by grooving the hardened concrete (Darwin 2014).

Curing

Darwin (2014) recommends that moist curing be applied using moist burlap and plastic for 14 days before a curing compound is applied and then protected for another 7 days. This is to minimize moisture loss until the mixture has gained sufficient maturity to resist shrinkage-related stresses. A review by D'Ambrosia (2013) found that five states specify curing as use of wet burlap for a minimum of 7 to 10 days.

Moist curing is still essential for mixtures containing LWFA, because it is still important to prevent premature drying at the surface to control cracking risk and enhance potential durability.

Differential temperatures between the girders and the slab should be controlled and minimized, by protecting both the girders and the slab (Darwin 2014).

Specifications

Internal curing has been adopted by the NYSDOT as their default requirement for bridge decks (NYSDOT 2018). The National Concrete Pavement Technology (CP Tech) Center has published a guide specification (Weiss and Montanari 2017) based on their requirements as well as on guidance from the ESCSI (2012). The aggregate to be used for internal curing is specified in ASTM C1761.

Recommendations

Balancing all of the above discussion, the following are suggested as target parameters when seeking to balance the need to achieve low permeability with minimum cracking:

- w/cm ratio range of 0.42 to 0.45
- Combinations of up to four SCMs (and limestone) based on the following:
  - 50 percent Portland cement clinker minimum
  - 25 percent Class F fly ash maximum
  - 35 percent Class C fly ash maximum
  - 50 percent slag cement maximum
  - 8 percent silica fume maximum
  - 15 percent interground limestone maximum
- Paste content maximum of 25 percent by volume
- IC to be included to deliver 7 lb. water per 100 lb. of cementitious materials in a mixture
- Air content range of 6.5 to 9.5 percent
- 1-in. maximum sized aggregate
- Maximum concrete temperature of 70°F
- External curing of 14 days minimum
- Finishing activities to be kept to a minimum

TEST PROGRAM

A laboratory testing program was developed and conducted to address concerns specific to bridge decks.

Cementitious Materials

Single samples of Portland cement, slag cement, and silica fume were used as cementitious materials. The cement and slag cement were obtained from a supplier that provides materials to the Ohio market. The chemical composition of these materials is shown in Table 4.

Table 4. Chemical composition of cementitious materials

Chemical, %	Type I-II	Slag cement	Silica fume
CaO	62.6	-	0.4
SiO_2	19.5	-	97.9
Al_2O_3	5.3	-	0.2
Fe_2O_3	3.3	-	0.1
MgO	2.6	-	0.2
NaEq	1.0	0.8	0.5
SO_3	3.8	-	0.2
Insoluble residue	0.5	-	-
LOI	1.3	-	-
C_3S	55.0	-	-
C_2S	14.0	-	-
C_3A	8.0	-	-
C_4AF	10.0	-	-
Specific gravity	3.15	2.90	2.21
Fineness, m^2/kg	400	574	-

Aggregate

The properties of the aggregates used are shown in Table 5. The aggregates were obtained from a source near the laboratory in Iowa but were considered sufficiently similar to Ohio materials.

Table 5. Aggregate information

	Notation	Source	Nominal max size, in.	Type	Specific gravity	Absorption, %		Desorption at 93% RH, %
						24 hours	Ultimate	
Coarse aggregate	CA	Martin Marietta, Ames, IA	1.0	Limestone	2.68	-	0.8	-
Intermediate aggregate	Inter CA	Martin Marietta, Ames, IA	3/8	Limestone	2.68	-	1.2	-
Fine aggregate	FA	Hallet, Ames, IA	-	River sand	2.66	-	0.9	-
LWFA 1	SO	Solite Hydrocure, Brooks, KY	-	Expanded shale	1.75	17.6	24.0	0.95
LWFA 2	TI	Trinity, Livingston, AL	-	Expanded clay	1.23	22.2	32.6	0.92

The individual aggregate gradations are shown in Figure 11.

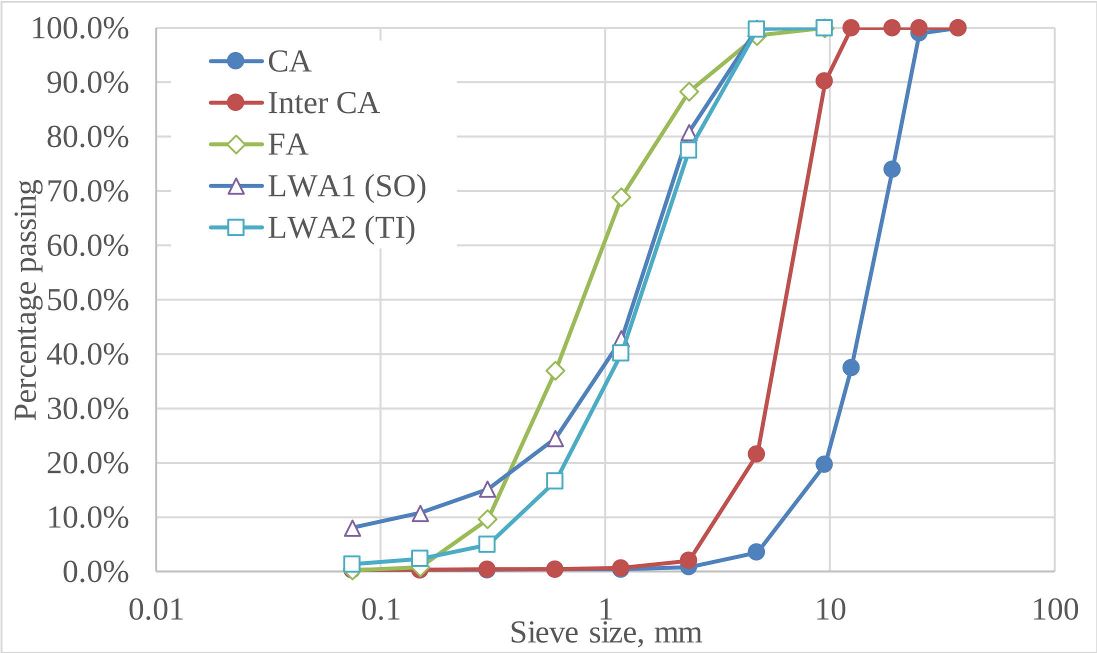


Figure 11. Individual aggregate gradations

The three combinations of aggregate systems were proportioned to fit into a Tarantula curve (Ley et al. 2012) and are summarized in Table 6.

Table 6. Combinations of aggregate systems

	Combination 0, %	Combination 1, %	Combination 2, %
CA	46.3	49.4	46.9
Inter CA	17.5	12.0	15.0
FA	36.2	30.9	30.5
SO	0.0	7.7	0.0
TI	0.0	0.0	7.6
Total	100.0	100.0	100.0
Voids, %	32.2	20.6	28.6

Mixture proportions were selected using an approach proposed by Taylor et al. (2015). The combined volume of aggregate voids was measured in accordance with a modified ASTM C29 procedure. In order to provide a minimum of 7 lbs of absorbed water per 100 lbs of cementitious material, 20 percent (by weight) of the fine aggregate was replaced with LWFAs.

The combined aggregate gradations are shown in Figure 12.

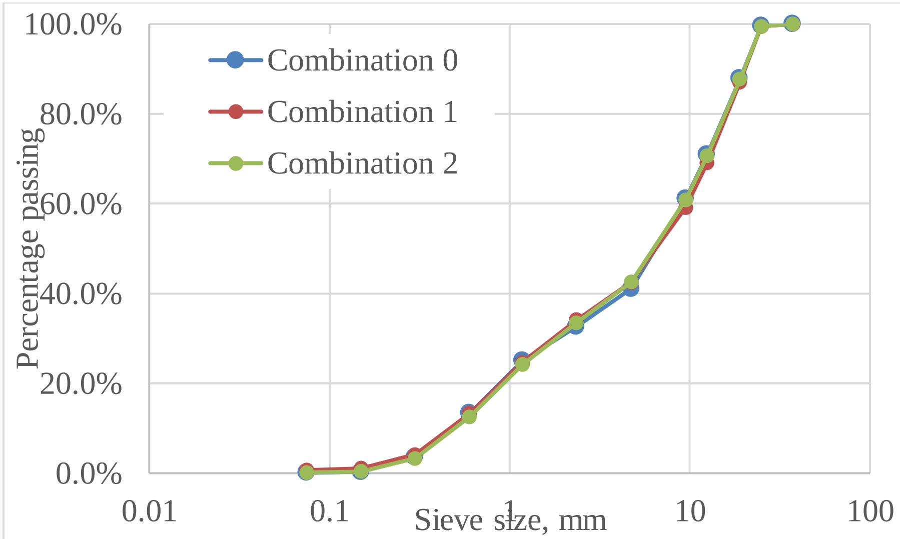


Figure 12. Combined aggregate gradations for the three aggregate systems

It can be observed that the three systems had similar combined aggregate gradations. Combination 1 is illustrated in a Tarantula curve, power 45 curve, and Shilstone workability factor chart in Figure 13.

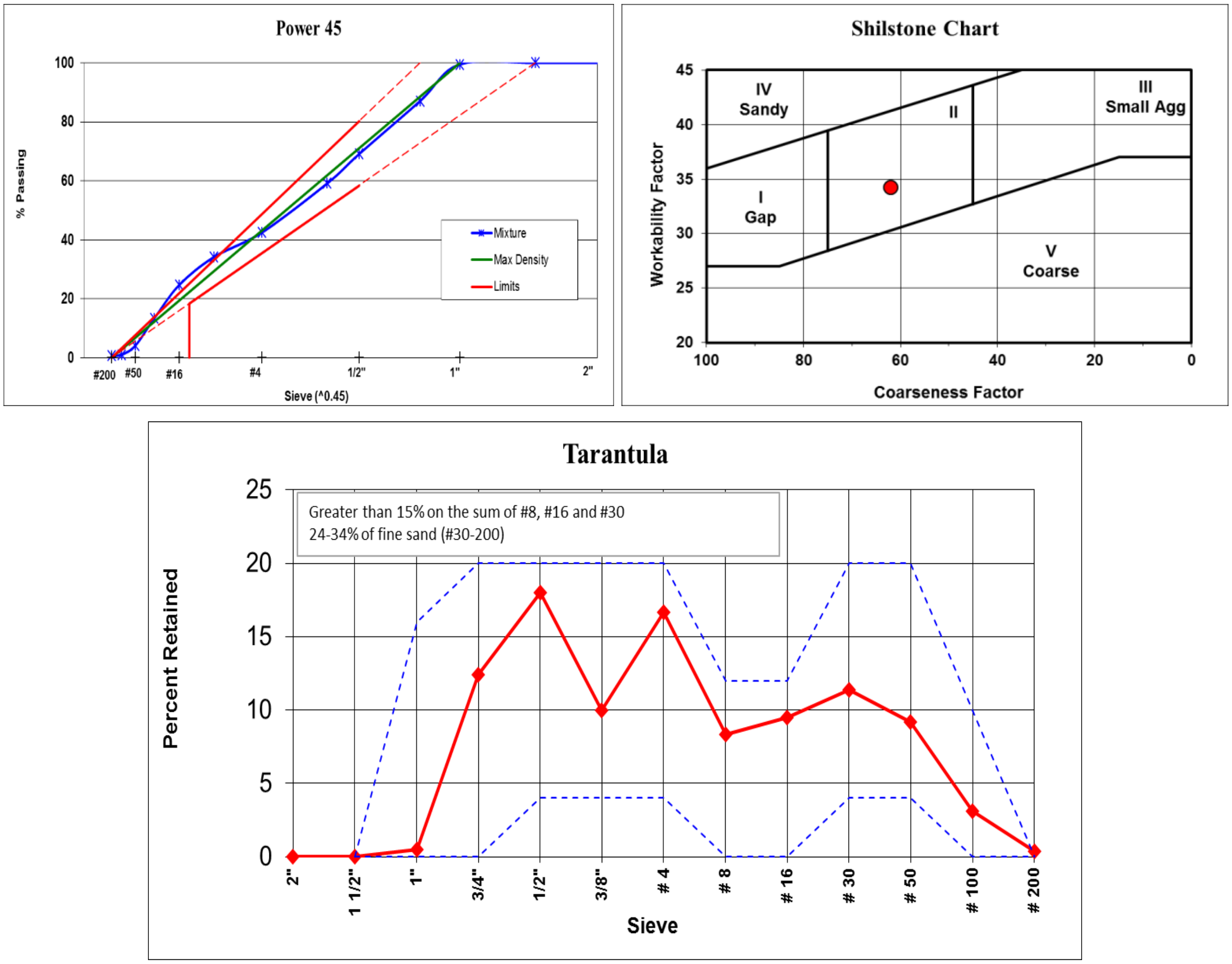


Figure 13. Aggregate system Combination 1 in power 45 chart (upper left), Shilstone workability factor chart (upper right), and Tarantula curve (bottom)

Mixture Proportions

A matrix of 11 mixtures was prepared with the following characteristics:

- Cementitious materials:
  - 30 percent slag cement
  - 50 percent slag cement
  - 30 percent slag cement and 4 percent silica fume
- Aggregate combinations:
  - Combination 1 as control (without LWFA)
  - Combination 2 (with Solite LWFA)
  - Combination 3 (with Trinity LWFA)
- Fixed binder content of 564 lbs/yd^3
- Fixed w/cm ratio of 0.45
- Target air content: 5–8 percent
- Target slump: 3–5 in.

The mixture proportions are shown in Table6. The chemical admixtures used in the matrix are air entraining agent (AEA), ASTM C494 Type A and F high-range water-reducing agent (HRWR), and SRA.

Mixtures OH10 and OH11 shown in Table 7 were based on 50 percent slag cement mixtures with SRA introduced. The dosage of SRA in the OH10 and OH11 mixtures was 1.5 gal/yd^3 which is the upper limit of the recommended range.

Table 7. Mixture proportions

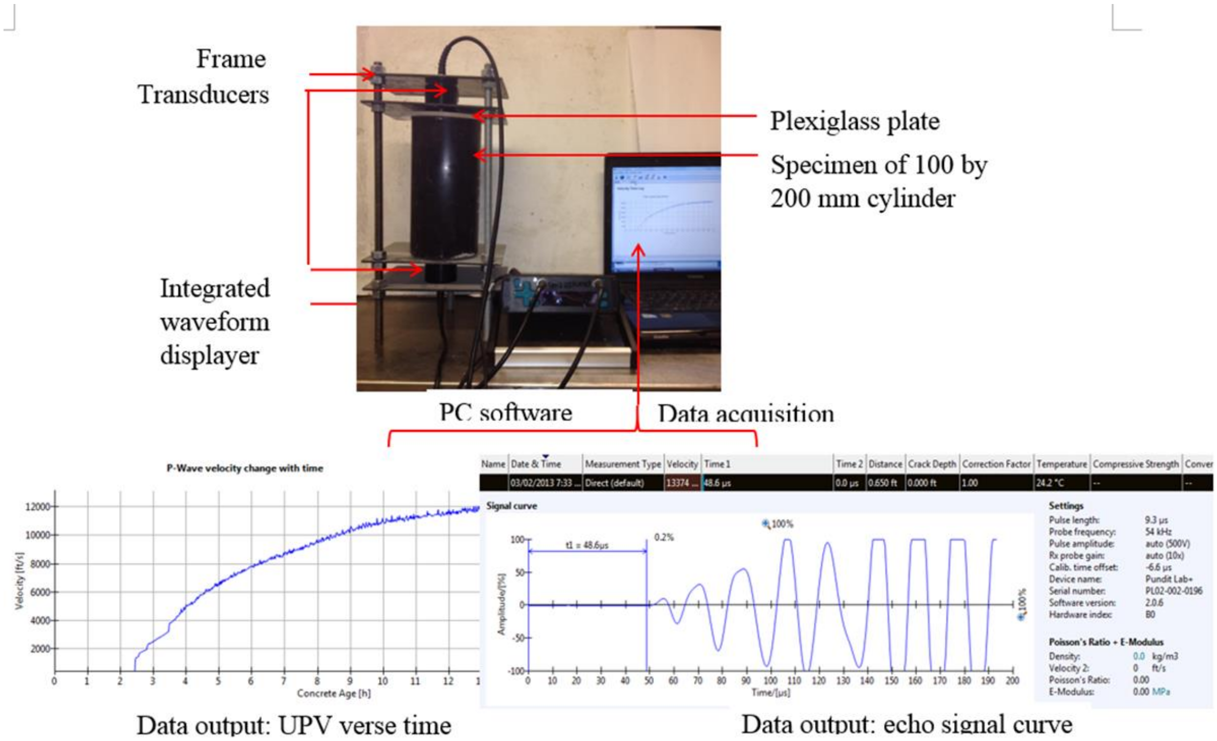
Mix	Mixture Design										
	C I,II	Slag cement	SF	LWFA	CA	Inter Agg.	FA	Water	HRWRA	AEA	SRA
ID	lbs/yd^3	lbs/yd^3	lbs/yd^3	lbs/yd^3	lbs/yd^3	lbs/yd^3	lbs/yd^3	lbs/yd^3	oz/cwt	oz/cwt	gal/yd^3
OH1(30S)	392	168	0	0	1423	536	1111	252	0.0	1.1	0.0
OH2(50S)	278	278	0	0	1423	536	1111	250	0.0	1.1	0.0
OH3(30S4SF)	369	168	22	0	1423	536	1111	251	1.0	1.1	0.0
OH4(30S/SO)	397	170	0	227	1450	351	908	255	0.0	1.1	0.0
OH5(50S/SO)	282	282	0	227	1450	351	908	253	0.0	1.1	0.0
OH6(30S4SF/SO)	374	170	23	227	1450	351	908	255	1.0	1.1	0.0
OH7(30S/TI)	397	170	0	210	1291	413	840	255	0.0	1.1	0.0
OH8(50S/TI)	282	282	0	210	1291	413	840	253	1.0	0.9	0.0
OH9(30S4SF/TI)	371	168	22	210	1291	413	840	253	2.0	0.9	0.0
OH10(50S/SO/SRA)	282	282	0	227	1450	351	908	253	1.0	0.9	1.5
OH11(50S/SRA)	278	278	0	0	1423	536	1111	250	1.0	1.0	1.5

Test Methods

The following laboratory tests were conducted:

- Air content (ASTM C231)
- Unit weight (ASTM C138)
- Semi-adiabatic calorimetry (ASTM C1753) (Figure 14)
- Ultrasonic pulse velocity (UPV) determination of initial set, which has been proven to be correlated to initial set measured by penetration resistance using ASTM C403 and calorimetry using ASTM C1753 (Wang et al. 2016) (Figure 15)
- Strength development (tensile and compression) up to 28 days (ASTM C496 and C39)
- Elastic modulus development up to 28 days (ASTM C469)
- Abrasion test (ASTM C944)
- Drying shrinkage (modified ASTM C157 – 7 days moisture curing)
- Restrained shrinkage (ASTM C1581) (Figure 16)
- Surface resistivity up to 56 days (AASHTO TP 95-11)

Figure 14. Calorimetry test device for measuring the temperature development of concrete



Wang et al. 2016. Author copy courtesy of www.icevirtuallibrary.com.

Figure 15. Test setup and data acquisition system of UPV measurement

Figure 16. Restrained shrinkage ring test apparatus

Results

Fresh Properties

The fresh properties of all the mixtures, including slump, air content, and unit weight, are shown in Table 8.

Table 8. Fresh properties

Mix	**Fresh properties**		
	Slump	**Air content**	**Unit weight**
ID	**in.**	**%**	**lbs/ft^2**
OH1(30S)	3.0	6.0	144.6
OH2(50S)	3.5	6.0	145.5
OH3(30S4SF)	4.0	8.0	142.0
OH4(30S/SO)	4.8	7.6	136.0
OH5(50S/SO)	4.0	6.4	139.1
OH6(30S4SF/SO)	3.3	8.0	134.8
OH7(30S/TI)	3.0	7.2	133.5
OH8(50S/TI)	3.8	7.6	134.2
OH9(30S4SF/TI)	3.0	7.5	132.3
OH10(50S/SO/SRA)	4.5	7.0	136.5
OH11(50S/SRA)	2.5	5.5	147.8

All the mixtures with LWFA replacement have a unit weight in the range of 132.3 to 139.1 lbs/ft^2, which is clearly lower than the unit weights of the mixtures without LWFA, as expected. The SRA in OH11 seems to decrease the total air content, which is consistent with the literature.

Ultrasonic Pulse Velocity (P-Wave)

Initial set was determined using a p-wave propagation technique with a commercial device (Wang et al. 2016). The system was placed in the laboratory environment at approximately 73°C and 50 percent RH. The setting time data are shown in Figure 17.

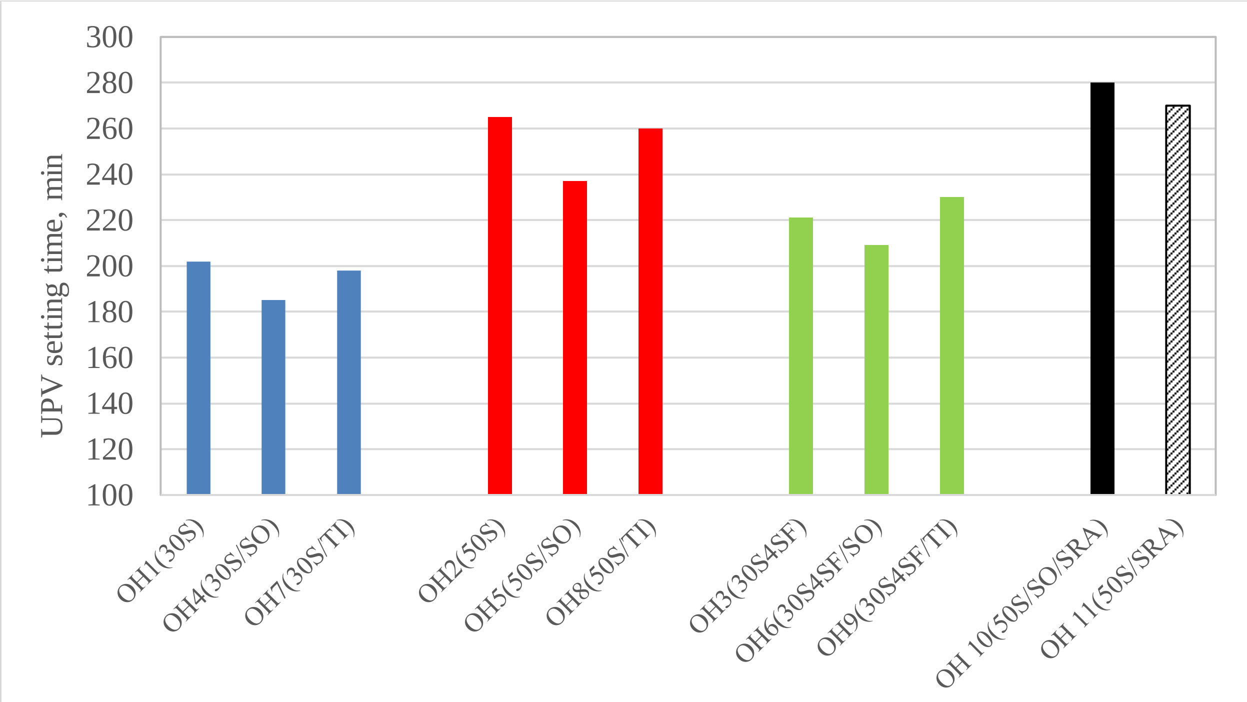


Figure 17. Initial setting time measured by UPV

It can be observed that the mixtures with 50 percent slag cement replacement have an extended initial setting time compared to the other two cementitious groups. Mixtures in which Solite LWFA was introduced have slightly shorter initial setting times, while SRA seems to increase setting time.

Semi-Adiabatic Calorimetry

The calorimetry curves for 24 hours after mixing are presented in Figure 18.

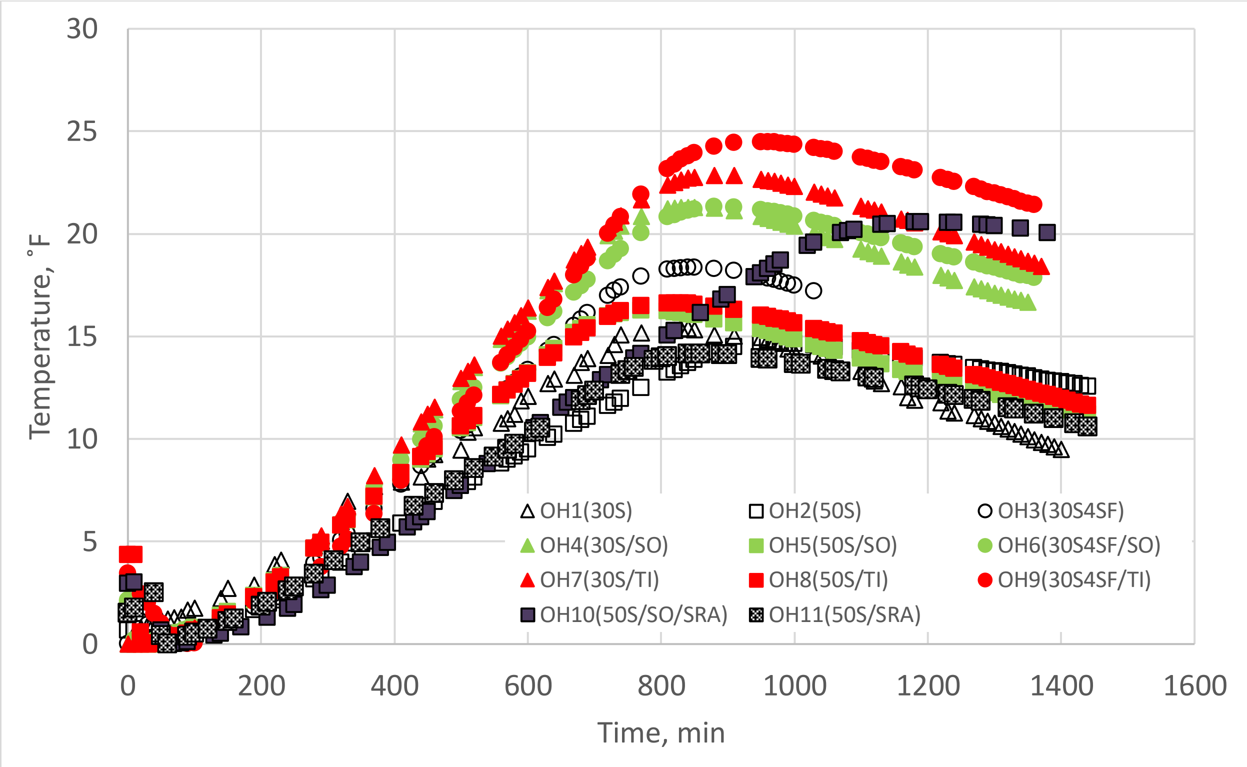


Figure 18. Temperature rise measured by calorimetry test

Mixtures with LWFA seem to have a higher temperature rise than those without LWFA. This is most likely because the water desorption from the LWFA provides continuous hydration, which generates more heat. The greater the replacement of cement in a mixture, the greater the reduction in temperature. The mixtures containing silica fume seem to have a higher temperature rise, as predicted in the literature. SRA does not seem to influence the temperature rise.

Calorimetry curves can also be used to predict initial setting times using the 20 percent fraction method (Wang et al. 2016). Figure 19 compares the initial setting time measured by the UPV and calorimetry methods. Generally, calorimetry-measured setting time seems to be higher than that measured by the UPV method.

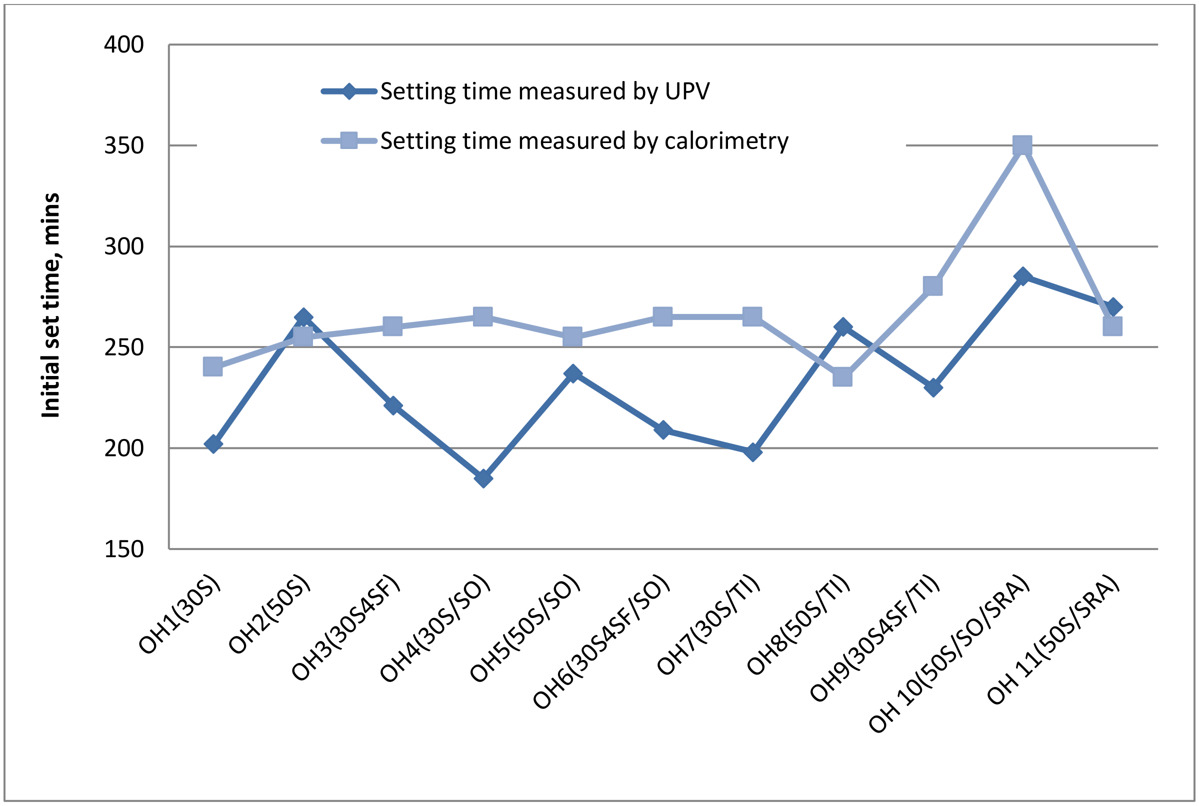


Figure 19. Initial setting times measured by the UPV and calorimetry approaches

Surface Resistivity

Criteria for assessing surface resistivity are shown in Table 9. The surface resistivity results are shown in Figure 20.

Table 9. Chloride penetrability classification

Chloride ion penetrability	AASHTO TP 95 (kohm-cm)	ASTM C1202 (Coulombs)
High	< 12	> 4,000
Moderate	12–21	2,000–4,000
Low	21–37	1,000–2,000
Very low	37–254	100–1,000
Negligible	> 254	< 100

Source: AASHTO TP 95 (2014)

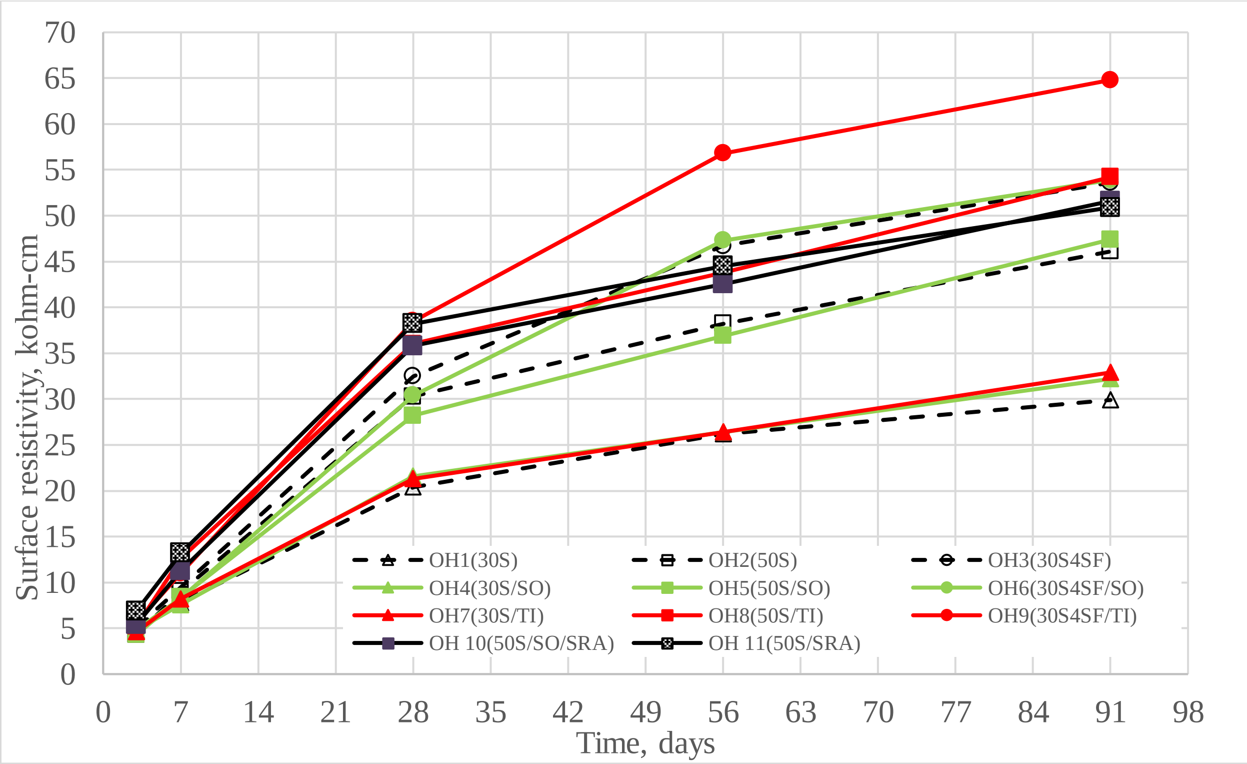


Figure 20. Surface resistivity results

At 28 days, all the mixtures fall within the “Low” or “Very low” classifications according to AASHTO TP 95. The mixtures’ surface resistivity values seem to fall into categories based on the combinations of cementitious materials in a mixture. The addition of silica fume densifies the system, resulting in a high surface resistivity. Mixtures with LWFA, either SO or TI, seem to have higher resistivity values (meaning a lower permeability) than the mixtures without LWFA in each group. LWFA continuously releases water to promote the hydration process, which densifies the system and produces a lower permeability.

Mechanical Properties

Strength and Modulus of Elasticity

Mechanical properties, including compressive strength, splitting tensile strength, and modulus of elasticity (MOE), were measured at 1, 3, 7, and 28 days, as shown in Table 10.

Table 10. Mechanical properties

Mix	Tensile splitting strength				Compressive strength				Modulus of elasticity			
	psi				psi				ksi			
ID	1 day	3 day	7 day	28 day	1 day	3 day	7 day	28 day	1 day	3 day	7 day	28 day
OH1(30S)	203	354	460	478	1473	3323	4423	6725	3275	4800	5475	5800
OH2(50S)	104	310	366	437	1119	2418	3794	6381	3475	4350	5150	6125
OH3(30S4SF)	188	326	403	578	1304	2691	3979	6094	3500	4550	4475	5500
OH4(30S/SO)	122	272	363	604	1045	2007	2859	4773	2750	3425	4000	4450
OH5(50S/SO)	139	-	405	623	941	-	3131	6446	2775	-	4000	4900
OH6(30S4SF/SO)	194	319	443	616	1302	2499	3866	5933	2750	3750	4500	4875
OH7(30S/TI)	212	339	397	580	1715	3268	4549	6396	3025	3775	4175	4875
OH8(50S/TI)	200	276	478	625	1333	2270	3874	6224	2750	3350	4050	4650
OH9(30S4SF/TI)	212	535	494	672	1832	4234	5580	7499	2775	4050	4300	4925
OH10(50S/SO/SRA)	193	-	561	642	1203	-	4100	6583	3000	-	4375	5250
OH11(50S/SRA)	209	346	473	629	1338	2574	4177	6340	3325	4300	5100	5375

– denotes that the data are not available.

Generally, mixtures containing LWFA and/or SRA have improved tensile splitting strength, especially at 28 days. However, the compressive strength improvement is marginal. It is notable that the introduction of LWFA reduces the MOE at all ages, which in turn should result in a lower risk of cracking.

Abrasion Test

The abrasion resistance of the concrete surfaces was measured by the rotating-cutter method in accordance with ASTM C944. A 4 in. by 8 in. cylinder was cut into 4 in. by 2 in. disks, and one finished surface and four cut surfaces were tested. A double load (44 lb-ft [197 N]) was applied for three cycles, and mass loss was recorded after each two-minute cycle. Example pictures of the finished and saw cut surfaces of mixture OH4 after abrasion testing are shown in Figure 21. Some of the LWFA particles were removed during the abrasion process, as can be seen in the figure.

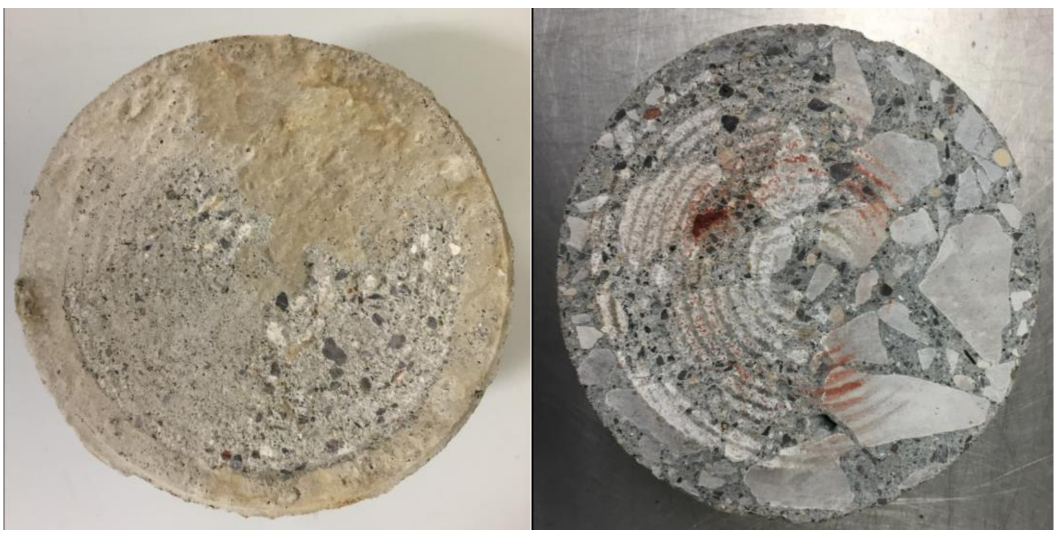


Figure 21. OH4 trowel-finished surface (left) and saw cut surface (right)

Total abrasion mass loss after three cycles is shown in Figure 22, along with scatter bars for the cut surfaces. In terms of saw cut surfaces, the LWFA mixtures do not show significant differences from the mixtures without LWFA. Differences in the finished surfaces are likely due to the effects of finishing and bleeding.

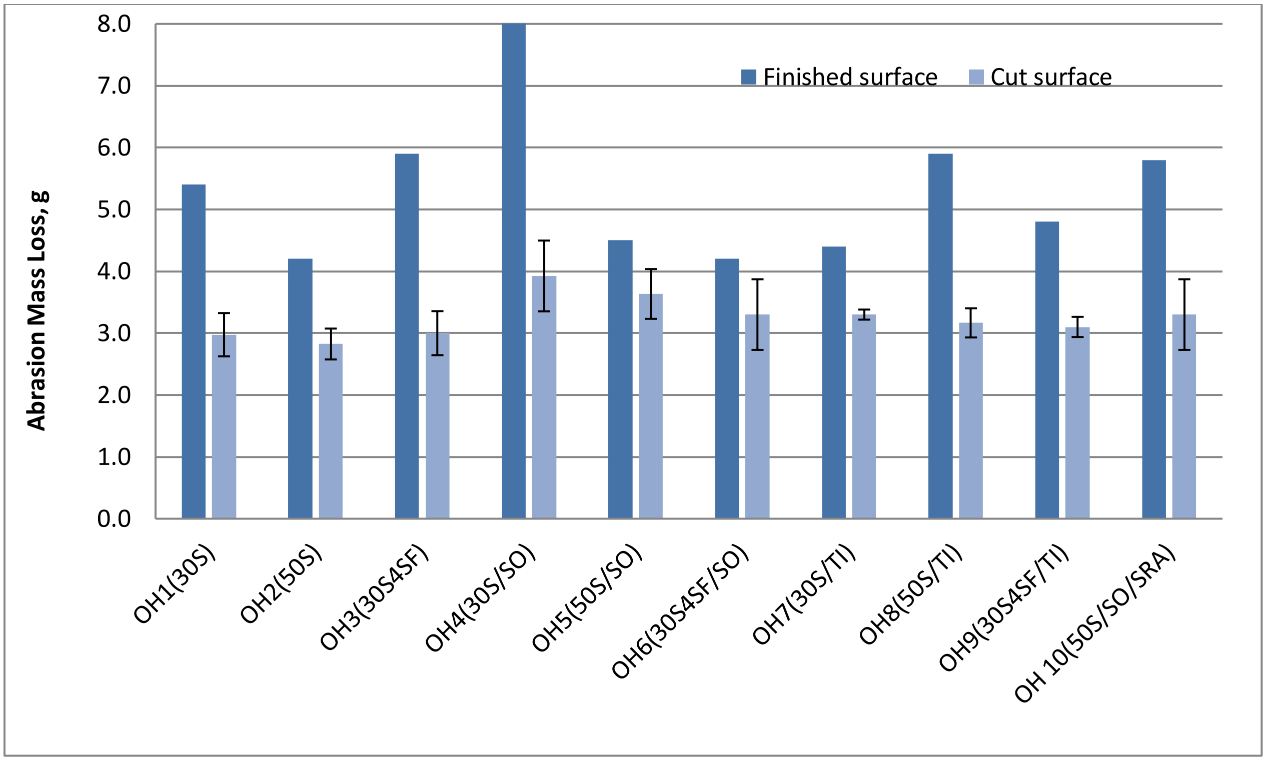


Figure 22. Abrasion mass loss

Shrinkage

Free Shrinkage

Free shrinkage refers to the total shrinkage of unrestrained prism specimens and may include autogenous and drying shrinkage components. Free shrinkage was measured in accordance with modified ASTM C157. Moisture curing was applied for seven days before an initial reading was taken. Data were then collected for up to 56 days of drying.

Figure 23 shows the free shrinkage strain with time of drying. It was observed that mixtures with LWFA have a lower free shrinkage over the time of drying. The mixture with SRA and no LWFA behaved in a similar manner to the LWFA mixtures, and the mixture with both SRA and LWFA exhibited the least free drying shrinkage.

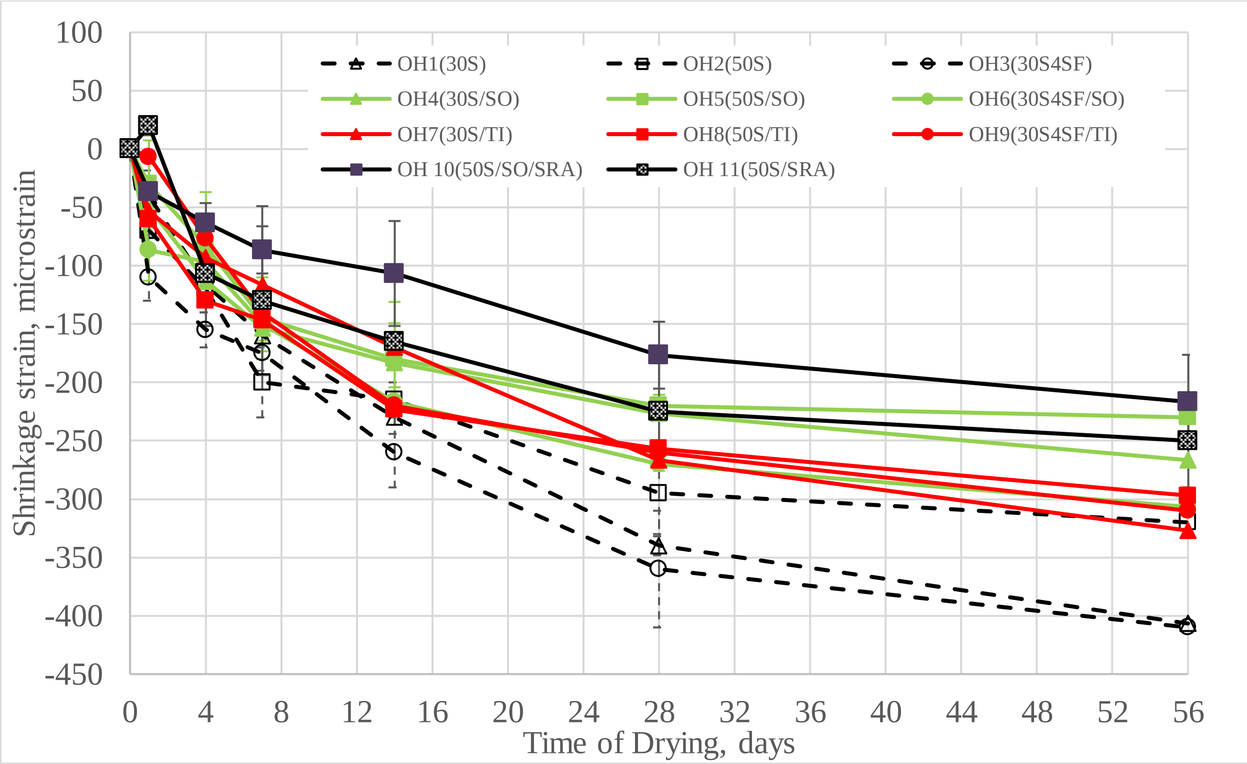


Figure 23. Free shrinkage results (after seven days moisture curing)

Restrained Shrinkage (Ring Test)

Shrinkage under restrained conditions was evaluated in accordance with ASTM C1581 using a ring apparatus. The concrete was hand-sieved with a 3/4 in. sieve before casting the rings in order to satisfy the maximum aggregate size requirement of the standard.

The development of strain in the steel rings is shown in Figure 24.

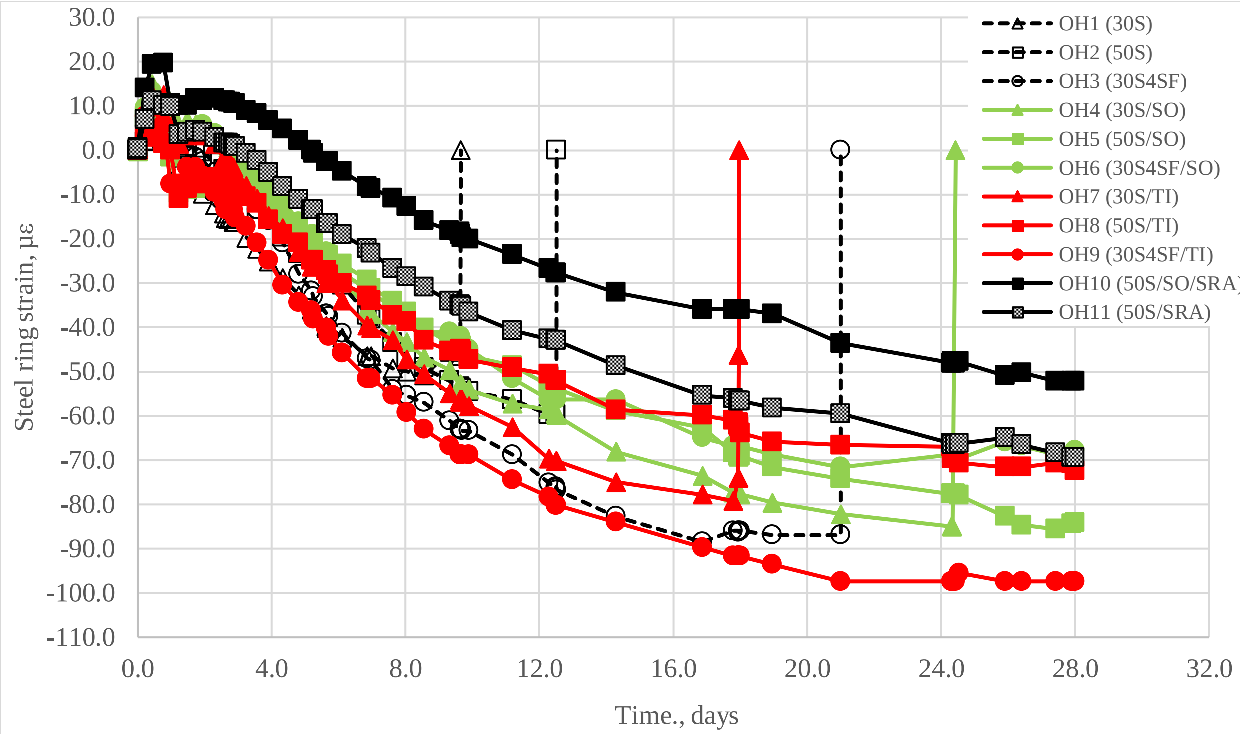


Figure 24. Strain of steel rings resulting from concrete shrinkage

Among all cementitious combinations, control group mixtures OH1, OH2, and OH3 without LWFA cracked before 20 days. All of the mixtures containing 30 percent slag cement cracked within 28 days, but mixtures OH4 and OH7 with LWFA cracked at later ages than the other mixtures in this category, approximately 25 and 18 days, respectively. Other mixtures reached a plateau around 18 days and did not crack up to 28 days, when the test was stopped.

An average shrinkage strain rate factor can be used to evaluate the cracking potential; the greater the shrinkage strain rate factor, the faster the concrete shrinkage increases. According to ASTM C1581, the shrinkage strain ε_s is plotted against the square root of elapsed time t to obtain the average strain rate factor α, as shown in Equation 1.

$$\varepsilon_s = \alpha\sqrt{t} + k \quad (1)$$

Figure 25 shows the average strain rate factor calculated for all mixtures.

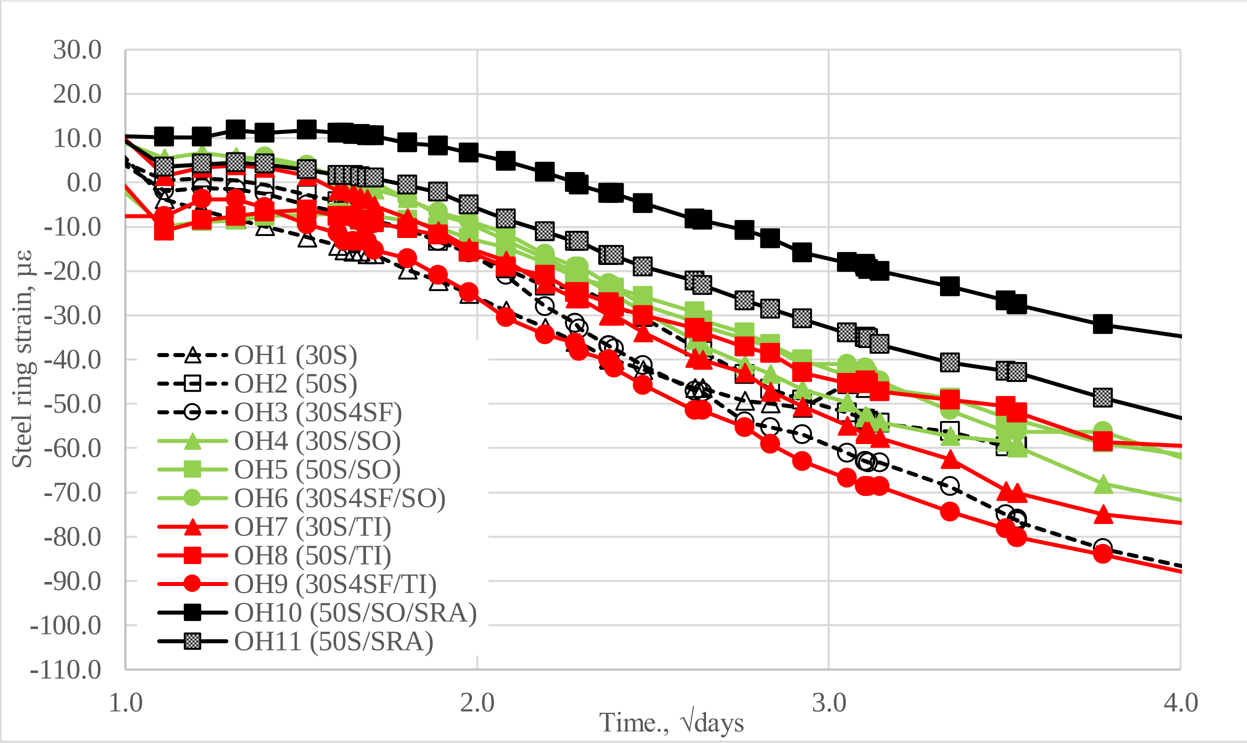


Figure 25. Determination of average strain rate factor

A drying age between 1 and 16 days was selected to determine the slope of the linear regression relationship because this range appeared to represent the time in which highest rate of shrinkage occurred.

A stress rate q was then determined based on Equation 2, which is provided in ASTM C1581 to evaluate cracking potential.

$$q = \frac{G|\alpha|}{2\sqrt{t_r}} \tag{2}$$

where G is the constant factor 10.47×10^6 psi and t_r is the elapsed time at cracking or elapsed time when the test is terminated for the specimen, in days (28 days in this case).

Table 11 provides detailed information regarding the net time to cracking, strain rate factor, stress rate, and cracking potential evaluated through ASTM C1581.

Table 11. Detailed information regarding the ring test

	Net time to cracking (days)			Strain rate factor	Stress rate	Cracking potential (ASTM C1581)
	Ring 1	Ring 2	Ring 3	(in./in.x10^{-6})/day$^{1/2}$	psi/day	
OH1 (30S)	9.6	8.7	10.6	-36.0	-63.9	High
OH2 (50S)	12.4	no	no	-29.2	-43.3	Moderate-High
OH3 (30S4SF)	19.7	no	no	-34.3	-40.5	Moderate-High
OH4 (30S/SO)	24.4	no	no	-31.3	-33.1	Moderate-High
OH5 (50S/SO)	no	no	no	-22.2	-22.0	Moderate-Low
OH6 (30S4SF/SO)	no	no	-	-27.4	-27.1	Moderate-High
OH7 (30S/TI)	18.0	20.9	-	-33.1	-40.9	Moderate-High
OH8 (50S/TI)	no	no	-	-21.2	-20.9	Moderate-Low
OH9 (30S4SF/TI)	no	no	-	-33.4	-33.0	Moderate-High
OH10 (50S/SO/SRA)	no	no	no	-18.2	-18.0	Moderate-Low
OH11 (50S/SRA)	no	no	no	-22.6	-22.4	Moderate-Low

The two mixtures with 50 percent slag cement replacement, OH5 with Solite LWFA and OH8 with Trinity LWFA, were assessed to have moderate-low cracking potential, while OH1 with 30 percent slag cement and no LWFA seemed to be the mixture most prone to cracking. OH10 with both SRA and Solite LWFA had the lowest shrinkage stress rate, while OH11 with only SRA had a stress rate close to that of OH5 and OH8 with either Solite or Trinity LWFA.

Hossain and Weiss (2004 and 2006) proposed a quantitative method to compute the residual stress in the concrete rings and a tool to determine the cracking potential, $\Theta_{CR,}$ (i.e., a measure of how close the ring specimen is to failure), by comparing the actual residual stress with the strength of the material. This approach was modified by Wang et al. (2012), based on an evaluation of tensile splitting strength.

The shrinkage-induced stress in restrained concrete was computed based on the steel ring strains and in consideration of the equilibrium of the pressure between the concrete and steel interfaces. The pressure p on the outer side of the ring is expressed in Equation 3.

$$p = \varepsilon_s E_s \frac{R_{so}^2 - R_{si}^2}{2R_{so}^2} \tag{3}$$

where ε_s is the strain of the steel ring measured on the interior side, E_s is the steel elastic modulus, and R_{so} and R_{si} are the outer and inner radii, respectively. Because ε_s is the strain actually measured, it includes the effect of creep.

The maximum shrinkage-induced stress on concrete at time t can be determined by Equation 4.

$$\sigma_{\text{Actual-Max}} = p\left(\frac{R_{OS}^2+R_{OC}^2}{R_{OC}^2-R_{OS}^2}+\nu\right) \tag{4}$$

where $\sigma_{\text{Actual-Max}}$ is the maximum shrinkage-induced stress, ν is the Poisson ratio, and R_{OS} and R_{OC} are the outer and inner radii of the steel and concrete, respectively (refer to ASTM C1581).

The simple ratio of the maximum shrinkage-induced stress and the tensile split strength is used to determine the cracking potential, as expressed by Equation 5 (Wang et al. 2012).

$$\Theta_{CR} = \frac{\sigma_{\text{Actual-Max}}}{f_{sp(t)}} \tag{5}$$

where $f_{sp(t)}$ can be determined by calculating the line of best fit for the development of tensile splitting strength over time for each mixture measured in this study.

Figure 26 shows the calculated cracking potential over time for all mixtures.

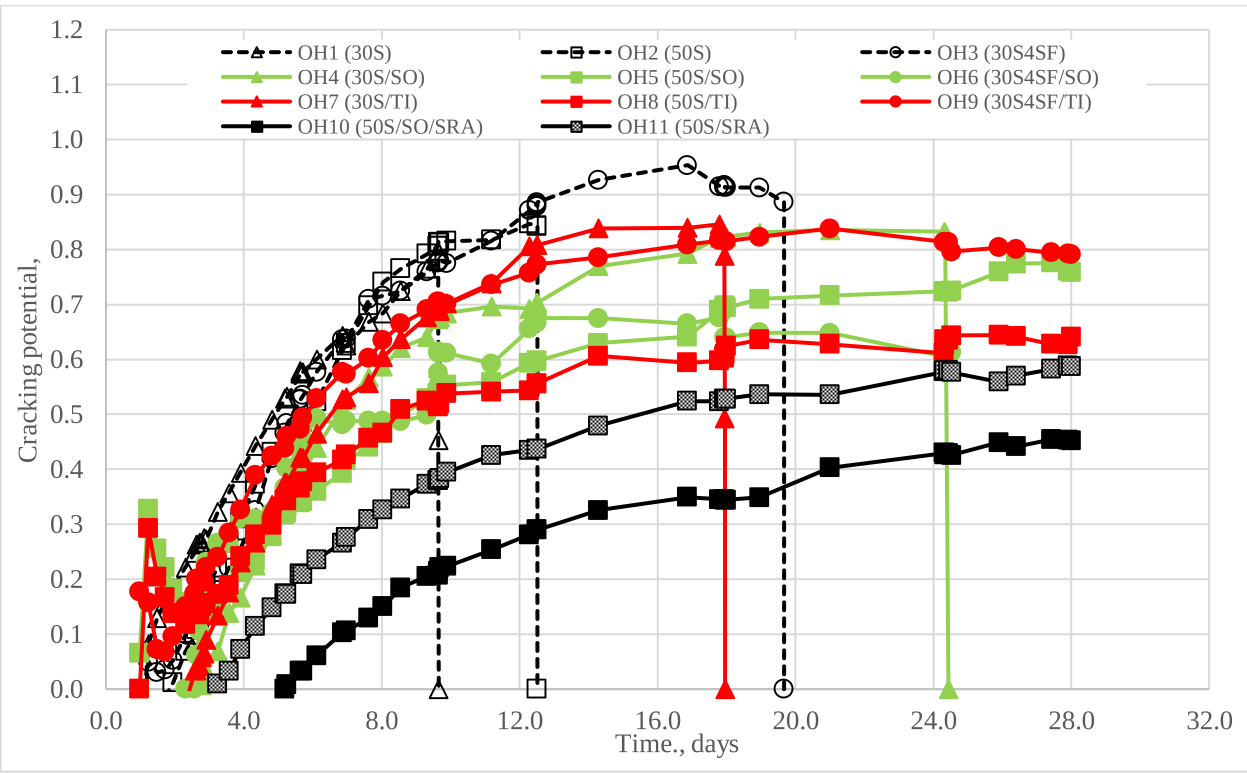


Figure 26. Cracking potential

Cracking would be expected to occur when the cracking potential reaches 1.0, i.e., when shrinkage-induced stress is equal to splitting tensile strength. As shown in Figure 26, mixtures having cracking potential values over 0.8 seem to be prone to cracking. This tendency may be attributed to the removal of large coarse aggregate particles in the ring test, but the tensile splitting strength values are based on the original mixtures. Also, it should be noted that the calculation of shrinkage-induced stress is based on linear elasticity for simplicity.

Again, the cracking potential assessed using this approach indicates that control mixes OH1 and OH2 reached 0.8 and cracked, similarly to OH4 and OH7 with 30 percent slag cement. It is worth noting that OH3 did not crack until it reached a cracking potential of 0.9 and OH9 passed beyond 0.8 without cracking. Those mixtures containing silica fume may densify the pore system, resulting in a higher splitting tensile strength, as shown in Table 9 which improved the resistance to cracking.

It is not surprising that OH10 and OH11 with SRA exhibited a lower cracking potential compared to the mixtures containing LWFA.

Selection of Mixture for Field Application

A number of tests identifying critical parameters for bridge decks have been conducted. Ranking of the various mixtures is not the same among the various tests. In order to assist with the selection of a mixture to use in the fieldwork, a system of pass/fail limits was applied to each test at 28 days. In some cases, the limits were selected based on "grading on the curve" rather than on an absolute engineering requirement, mainly because all of the mixtures were satisfactory for many tests, though some performed better than others. The data are summarized in Table 12.

Table 12. Summarized results and selected performance limits

Mix	Calorimetry temperature rise, °F	Tensile splitting strength psi	Compressive strength psi	Modulus of elasticity ksi	Surface resistivity kΩcm	Free shrinkage strain μ-strain	Cracking potential: ASTM C1581 stress rate psi/day	Cracking potential: ASTM C1581 classification	Cracking potential: $\sigma_{Actual\text{-}Max}/f_{sp(t)}$
OH1(30S)	15.4	478	6725	5800	20.4	-340	-63.9	High	0.80
OH2(50S)	15.0	437	6381	6125	30.3	-295	-43.3	Moderate-High	0.85
OH3(30S4SF)	18.3	578	6094	5500	32.5	-360	-40.5	Moderate-High	0.95
OH4(30S/SO)	21.3	604	4773	4450	21.6	-227	-33.1	Moderate-High	0.84
OH5(50S/SO)	16.3	623	6446	4900	28.2	-220	-22.0	Moderate-Low	0.78
OH6(30S4SF/SO)	21.3	616	5933	4875	30.4	-270	-27.1	Moderate-High	0.69
OH7(30S/TI)	22.9	580	6396	4875	21.3	-267	-40.9	Moderate-High	0.85
OH8(50S/TI)	16.6	625	6224	4650	36.0	-257	-20.9	Moderate-Low	0.64
OH9(30S4SF/TI)	24.5	672	7499	4925	38.5	-260	-33.0	Moderate-High	0.84
OH10(50S/SO/SRA)	20.6	642	6583	5250	35.8	-177	-18.0	Moderate-Low	0.45
OH11(50S/SRA)	14.2	629	4177	5373	38.2	-195	-22.4	Moderate-Low	0.59
Performance Limits	**20.0 max**	**600 min**	**5000 min**	**5000 max**	**28.0 min**	**-320 min**	**-25.0 max**	**Moderate-Low or better**	**0.80 max**

Green shaded cells indicate the test results that "passed" the imposed limits

Based on this approach, it can be seen that mixtures OH5 and OH8 were the only ones that passed all of the selected limits. Therefore, these systems should be considered for field use. Both of these systems contain 50 percent slag cement and either of the LWFA materials. Mixture 10 containing SRA passed all but one of the tests and may also be worth considering, especially in light of the very low cracking potential recorded. This recommendation must be balanced against the reported side effects of using SRAs and their costs.

BRIDGE IMPLEMENTATION

The aforementioned mix design was recommended for implementation on a bridge construction project in Ohio. The project selected was a pair of twin bridges, carrying Interstate 271 in Mayfield, Ohio, with construction occurring in summer 2017. The bridges were two-span, with total lengths of 193 feet. The southbound bridge deck was cast using traditional mix designs and was deemed the control deck, while the northbound bridge deck was cast using the recommended internal curing mix design. Use of SRA's was considered but precluded due their costs. These mix designs are presented in Table 13.

Table 13. Bridge deck mix designs

	Cement (lb/yd^3)	Slag (lb/yd^3)	Water (lb/yd^3)	Sand (lb/yd^3)	LWFA (lb/yd^3)	Coarse Agg. (lb/yd^3)	Coarse Agg. (lb/yd^3)
Control concrete	410	160	256.5	1305	-	1360	355
IC concrete	282	282	253.8	940	220	1380	345

Documentation of the deck placement was performed via on-site visual observations, as well as a series of embedded gages placed in the deck to monitor both early-age and long-term comparative performance. Table 14 details the fresh properties of both mixes, while Figure 27 shows the bridge prior to deck placement.

Table 14. Fresh properties of both mixes

	Unit weight (lb/ft^3)	Air content (%)	Slump (in.)
Control concrete	143.5	5.7	6
IC concrete	140.6	6.0	5

Figure 27. Bridge prior to deck placement, with wet burlap ready to be placed

On the day of deck placement, wet burlap was placed after finishing, with curing compound applied after 7 days.

Early-Age Instrumentation

Prior to placement of the deck, several gages were placed for embedding in the bridge deck. The instrumentation plan included the following:

- 5TE Decagon sensors: to measure temperature, bulk electrical conductivity, and relative permittivity (material property performance)
- Vibrating wire strain gages: to determine comparative performance in load carrying characteristics (structural performance)

The subsequent performance investigation can be divided into two parts: the material property performance and the structural performance. These two efforts will be further explained in the following sections.

Material Property Performance

Properties of the mixtures used in the two sections were monitored during and after construction.

Mixture proportions are shown in Table 15.

Table 15. Mixture proportions (SSD)

	Cement lb/yd^3	Slag Cement lb/yd^3	Water lb/yd^3	Sand lb/yd^3	LWFA lb/yd^3	Int. Agg., lb/yd^3	Coarse Agg., lb/yd^3
Control concrete	410	160	256.5	1305	-	1360	355
IC concrete	282	282	253.8	940	220	1380	345

Both mixtures had the same w/cm ratio of 0.45 and target air content, while the IC mixture had a higher slag cement content and about 20 percent (by mass) of the fine aggregate was replaced with lightweight fine material that was saturated before batching.

Fresh properties of the two mixtures are shown in Table 16.

Table 16. Fresh properties of the two mixtures

	Unit weight (lb/ft^3)	Air content (%)	Slump (in.)
Control concrete	143.5	6.0	6
IC concrete	140.6	6.0	4½

Semi-adiabatic calorimetry (Figure 28) of the two mixtures indicated that setting time was similar for both mixtures, but the peak temperature was significantly lower in the IC mixture, likely because of the additional slag cement. This may be considered beneficial because it reduces stresses due to temperature differentials through the thickness of the slab.

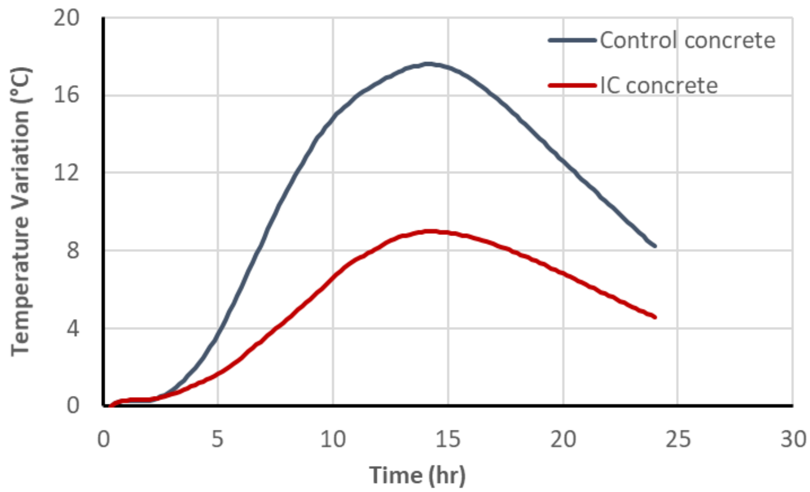


Figure 28. Semi-adiabatic calorimetry temperature rise

The temperatures of the mixtures were recorded using sensors embedded in the deck, as mentioned previously. Figure 29 illustrates that despite similar environmental temperatures, the IC mixture showed generally higher internal temperatures and smaller daily fluctuations for the first week. This supports the contention that the internal curing is promoting extended hydration, thus leading to higher mechanical performance in the long term, while damping temperature profiles, both of which will reduce the risk of cracking.

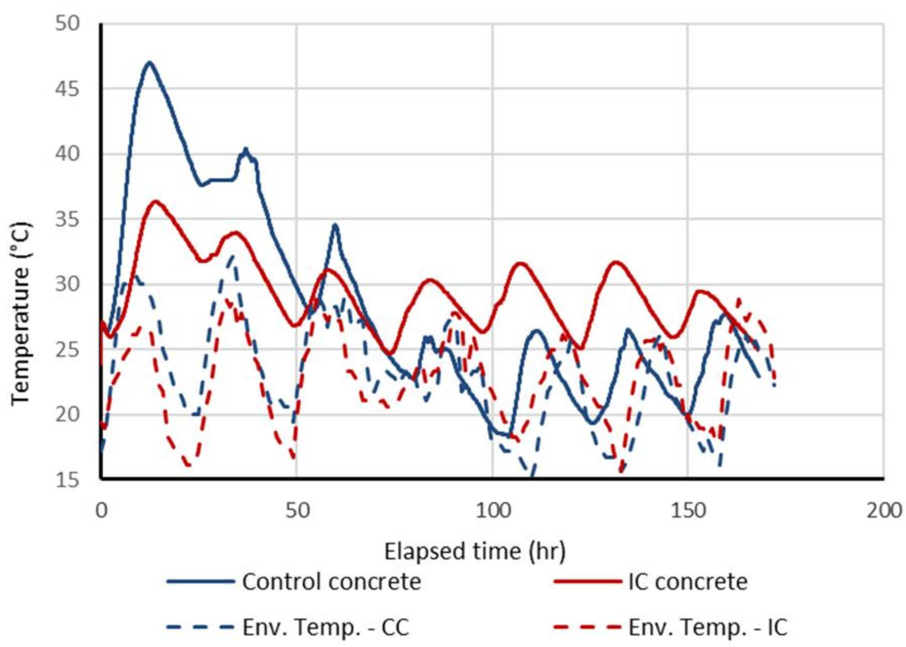


Figure 29. Temperature data

Mechanical properties of the mixtures are shown in Table 17.

Table 17. Hardened properties of the two mixtures

	Compressive strength, psi			Split Tensile Strength, psi		Resistivity, k ohm.cm	Relative Permittivity
	7 day	14 day	28 day	7 day	28 day	42 day	42 day
Control concrete	3980	4570	6030	355	385	18	15
IC concrete	4910	6570	6640	390	550	40	18

The increased strength of the test mixture would also contribute to reduced cracking risk in the long term. Increased resistivity is a measure of the improved hydration and impermeability of the mixture, potentially contributing to an increased lifetime because of an improved resistance to ingress of aggressive chemicals.

The higher relative permittivity of the test mixture near the bottom of the uncured, exposed surface of the slab is an indication that the moisture content of the system is higher, leading to the increased hydration, and thus the improved long-term properties that were measured. Little difference was observed between the top faces of the test and control slabs. This was likely due to the conventional curing applied to the decks.

Drying shrinkage of the mixtures was also assessed on field samples transported to the laboratory after 1 day (Figure 30). The test mixture shows less shrinkage than the control, again reducing the risk of cracking.

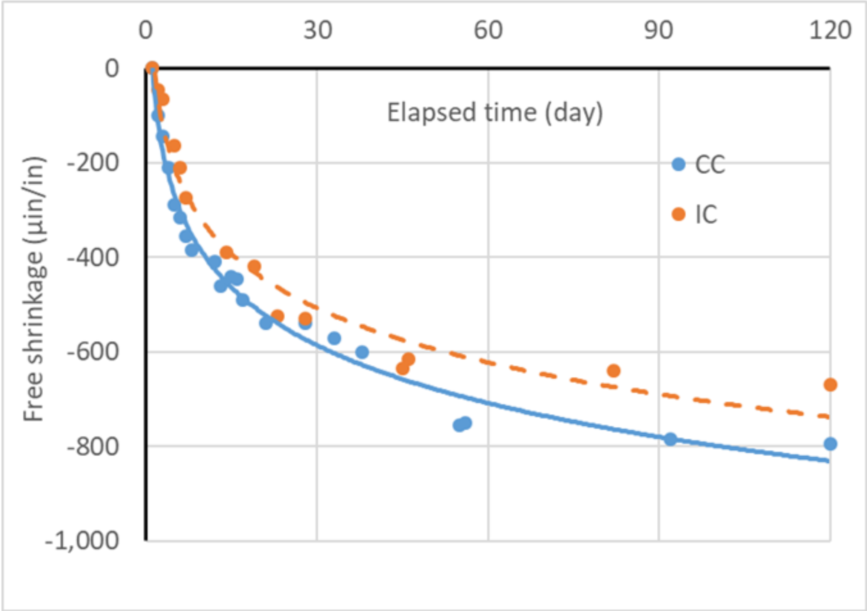


Figure 30. Unrestrained shrinkage

Structural Performance Testing

To assess the structural performance of both bridges, six vibrating wire strain gages were embedded in the bridge deck, in the locations shown in Figure 31. The gages were placed at mid-depth, attached to the bottom mat of the reinforcement, as shown in Figure 32.

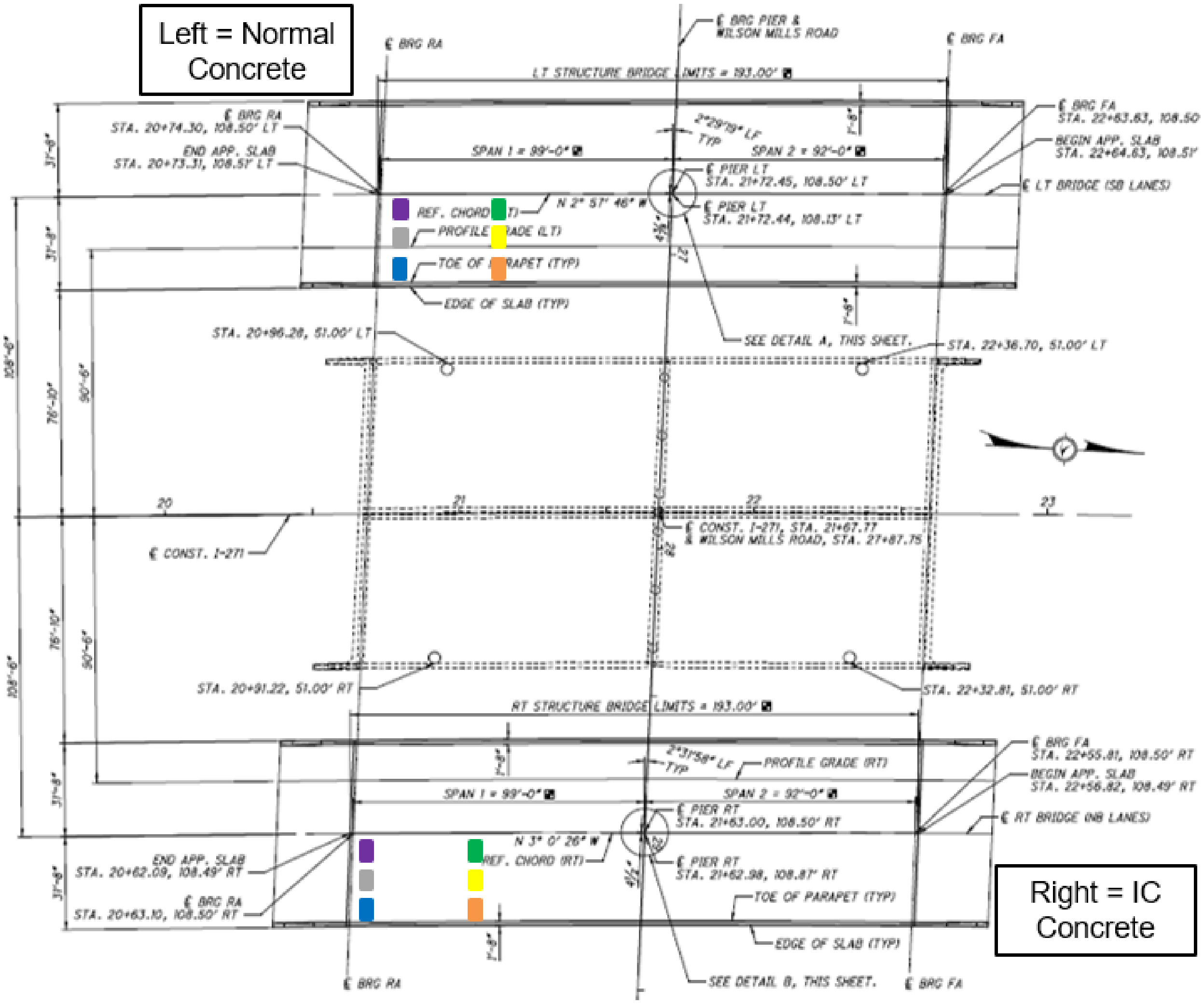
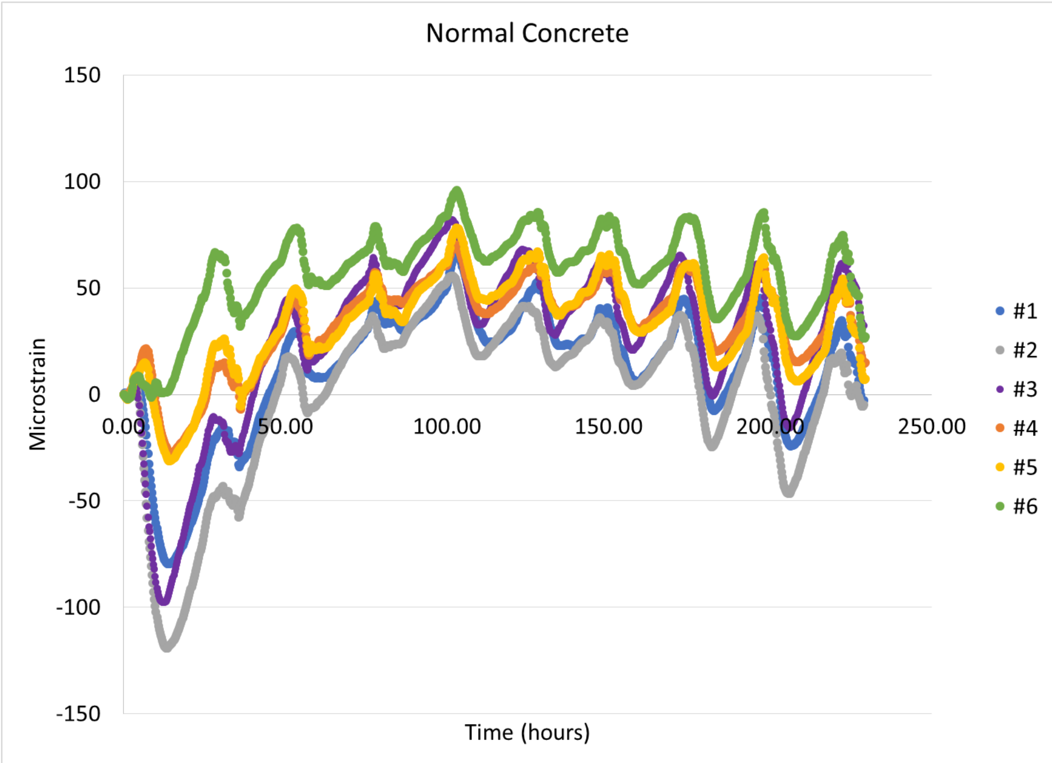


Figure 31. Embedded strain gage locations for both bridge decks

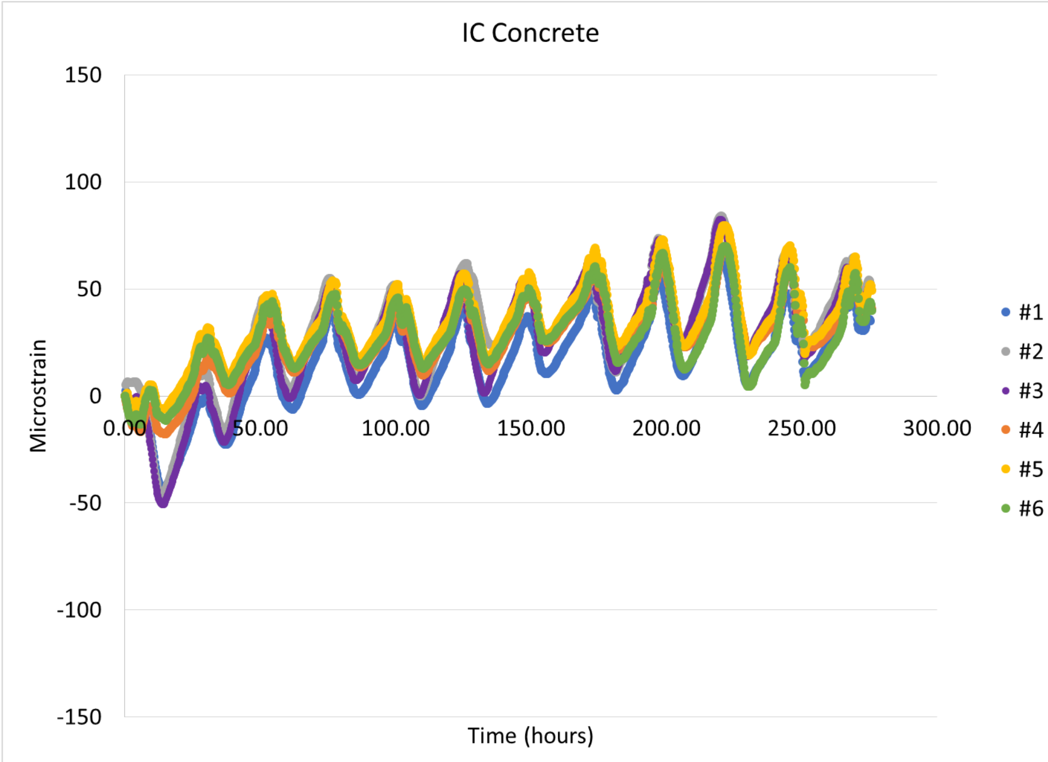
Figure 32. Embedded strain gage placement

The data from the strain gages was collected, and are seen in Figure 33 and Figure 34 for the first ~2 weeks after placement for each bridge.



Line colors correspond to the gage location schematic shown in Figure 31

Figure 33. Strain data for the normal concrete bridge deck



Line colors correspond to the gage location schematic shown in Figure 31

Figure 34. Strain data for the IC concrete bridge deck

As can be seen from these figures, the two bridges performed very similarly, with daily variations in strain due to temperature changes. Overall, the IC concrete bridge deck experienced slightly lower strains, with behavior similar to that of the normal concrete bridge, although the enveloped strain profile of the IC concrete was less than that of the normal concrete.

Live Load Distribution Evaluation

In addition to the embedded gages, which monitored the early-age performance of the two bridges, efforts were made to monitor the performance of the bridge up to a year after placement. These evaluations included live load testing of the bridge one month after placement, as well as crack mapping and condition assessment one year after placement.

For the live load testing of the bridge, strain gages were attached to the girders and bridge decks according to the instrumentation plan shown in Figure 35. A total of 33 strain gages were attached to each bridge, with sample placement images shown in Figure 36.

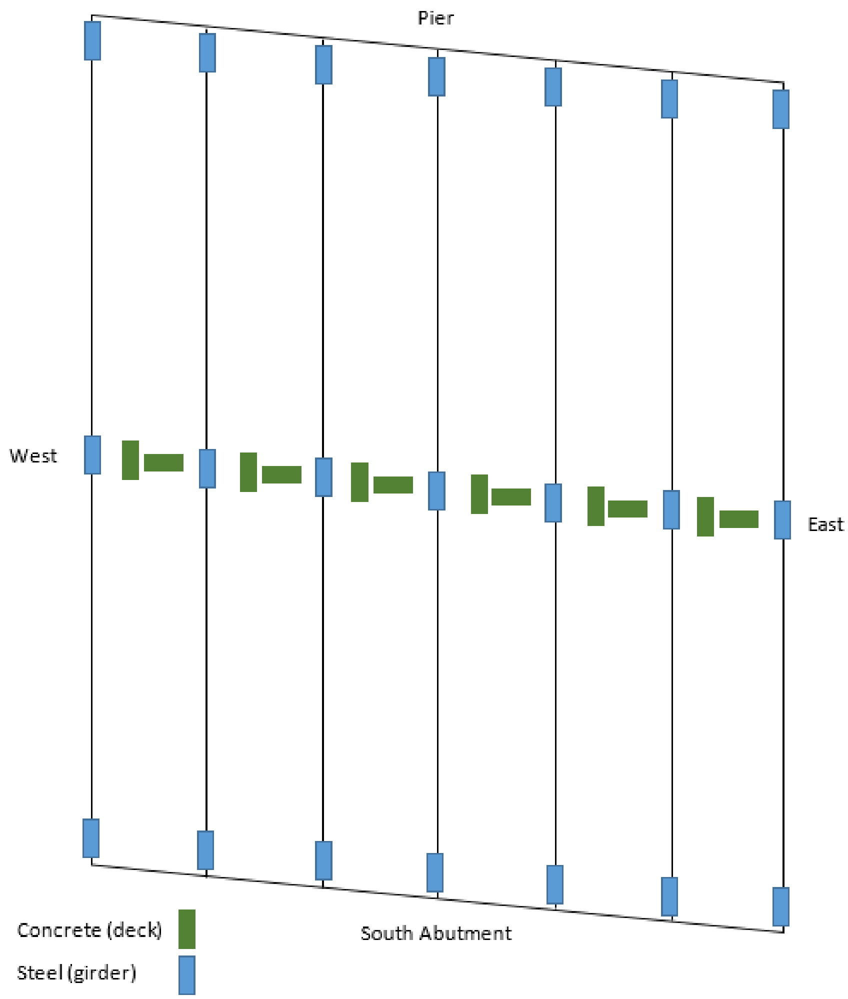


Figure 35. External strain gage instrumentation plan

Figure 36. Deck gage placement (left) and girder gage (right)

After the gages were attached to the structure, a weighted dump truck was driven across the bridge at a crawl speed. Five load cases were considered, as illustrated in Figure 37.

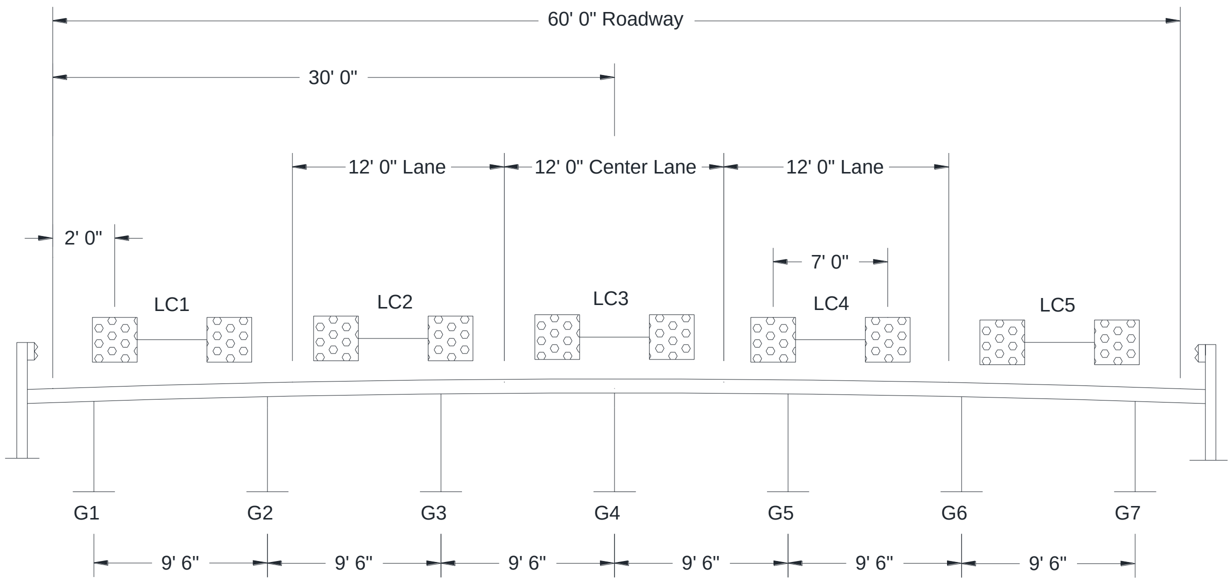


Figure 37. Live load testing truck load paths

The strain data from these load cases can then be used to determine the transverse load distribution and can be compared to the codified AASHTO design values. These data are presented in Figure 38 and cover both the normal concrete and the IC concrete.

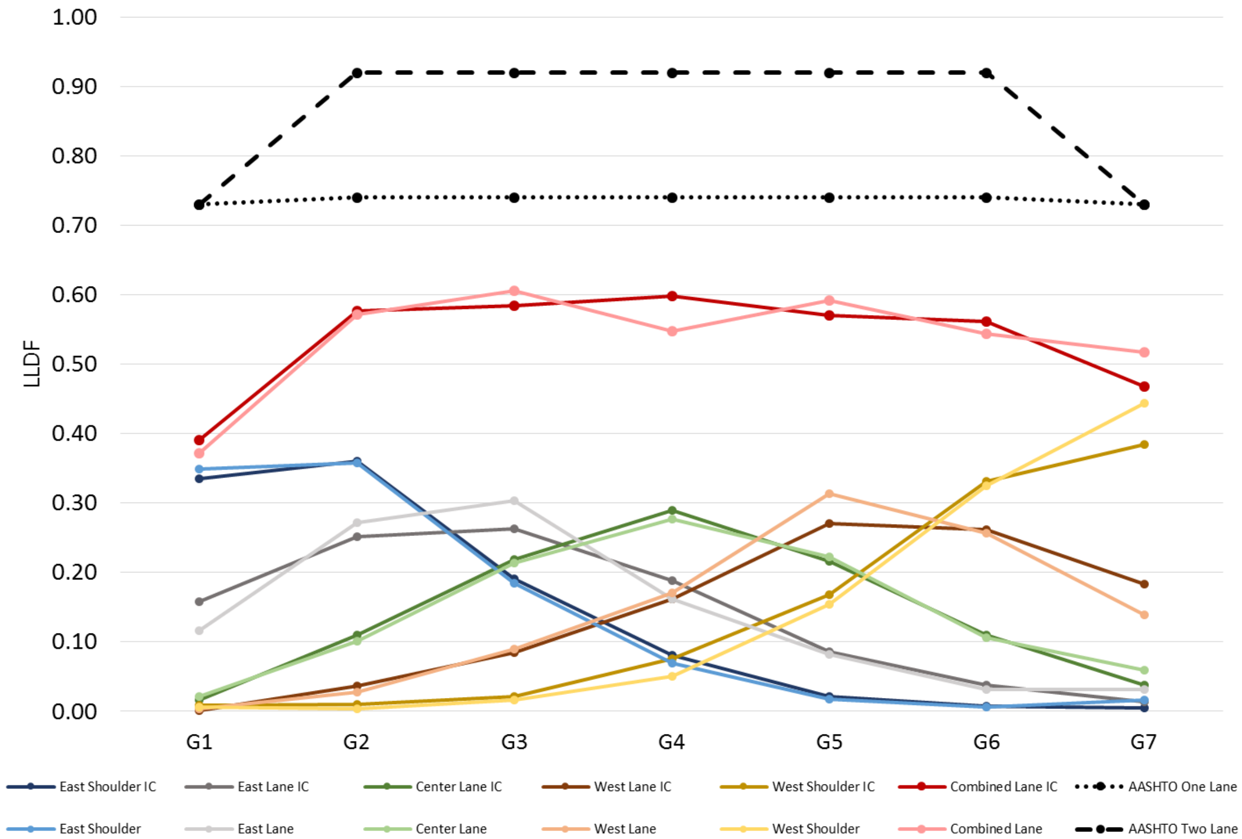


Figure 38. Transverse load distribution for both bridges.

As can be seen, the load distribution for both bridges was extremely similar, and in both cases fell below the AASHTO values.

In addition to determining the transverse load distribution of both bridges, the strain data can also be used to compare performance of the two bridges from a localized perspective. For example, Figures 39 and 40 show the data for the deck gages closest to the center girder for load case 3. In Figure 39, for the deck gages located just to the west of the center girder, the strains in the normal concrete were slightly greater than those seen in the IC deck. However, in Figure 40, for the deck gages located just to the east of the center girder, the strains in the IC deck were slightly higher.

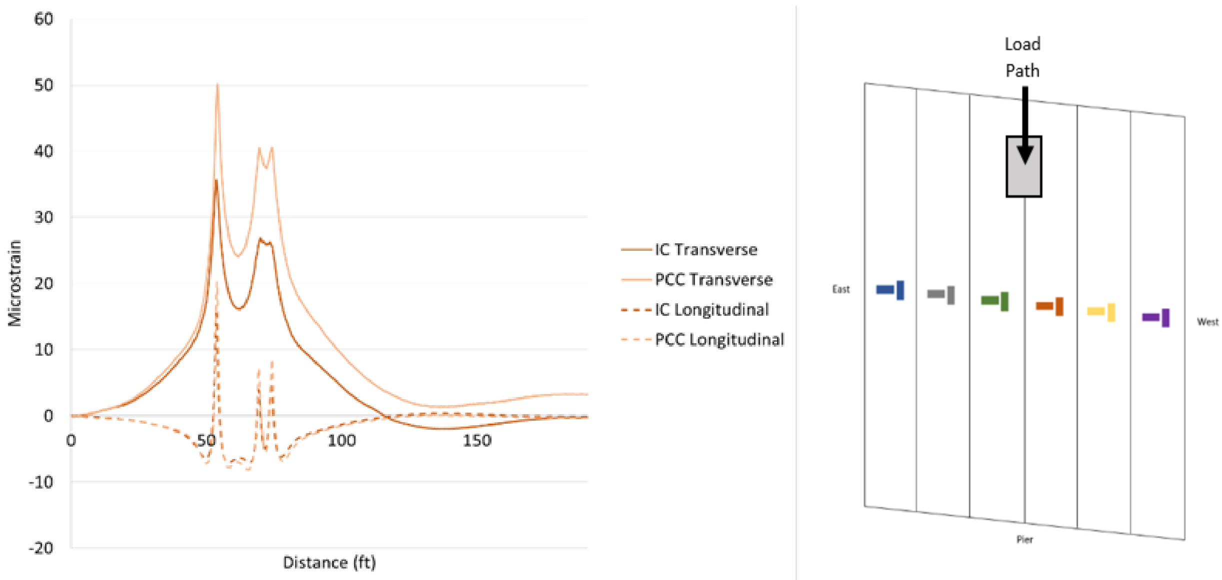


Figure 39. Load case 3 near deck gage data comparison (west)

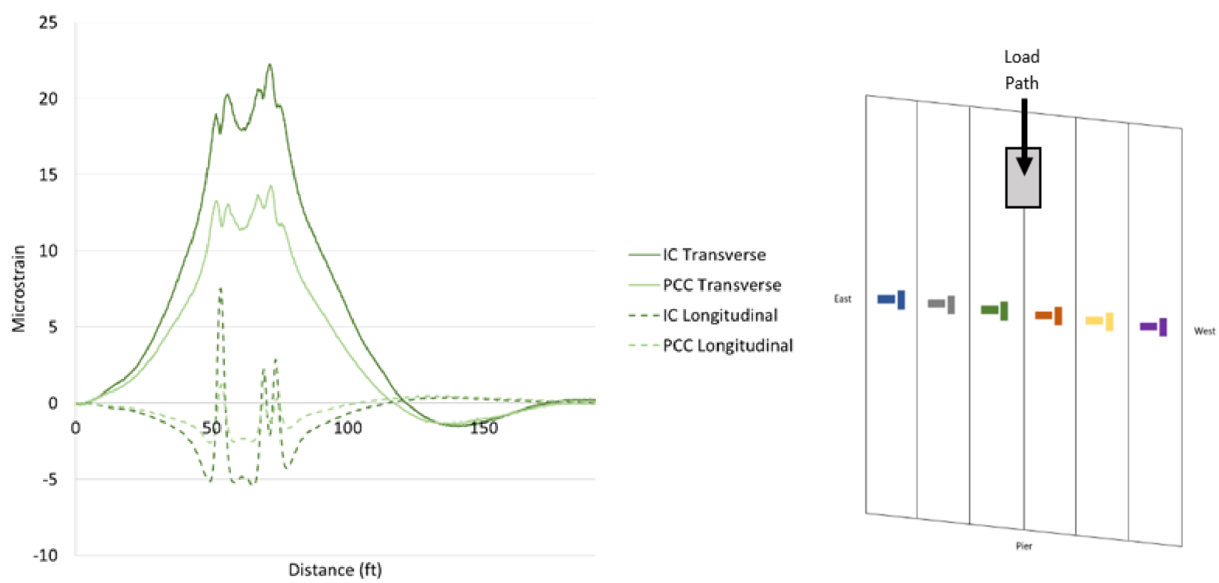


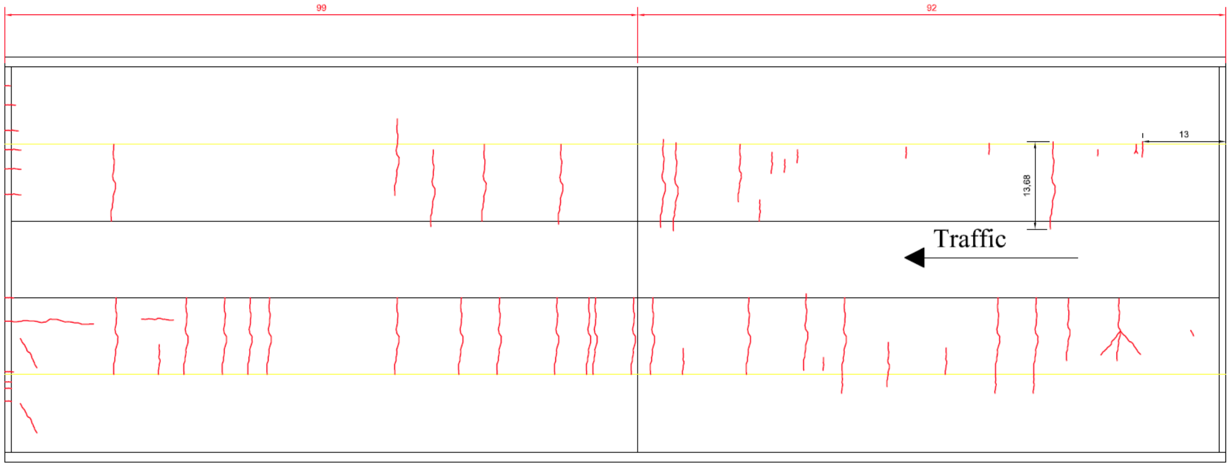
Figure 40. Load case 3 near deck gage data comparison (east)

As this example illustrates, there was no consistent difference in structural response between the two bridges. This was the case for all gage locations and load cases, signaling no discernable difference in structural performance between the two bridges.

Year One Inspection

The initial survey was performed on both brides approximately two weeks after deck placement. No visible cracks were found in the bridges at that time. Also, approximately one year after deck placement, both bridges were inspected to compare performance and identify any deterioration.

Based upon the strain data obtained from the embedded gages, the southbound bridge experienced greater strain and thus would be expected to experience greater cracking levels. The crack map for the southbound, normal concrete, bridge deck is shown in Figure 41, with an image showing an example of the observed cracking in Figure 42.



Conventional Concrete Deck

Figure 41. Normal concrete deck (southbound) crack map

Figure 42. Cracking on southbound, inner lane of the normal concrete deck

As can be seen, transverse deck cracks with widths averaging approximately from 0.01 to 0.013, varied in length, were observed at many locations on outer and inner lanes. The middle lane of the bridge was not surveyed because it was kept open to traffic during the survey. Longitudinal cracks were also observed near the bridge approaches, with width ranges from 0.02 to 0.05 inches. It was also noted that minor cracking was present on the grooves of the finished concrete surface of the outer lane. According to ACI 201-1R-08, it defined the crack severity as a fine for crack width less than 0.04 and a medium for crack width between 0.04 and 0.08. However, thermal stresses due to temperature variation could cause these cracks to widen over time. As a result, these cracks would provide a pathway of chloride ions to penetrate the deck and corrode the reinforcement. A crack map for the IC deck appears in Figure 43.

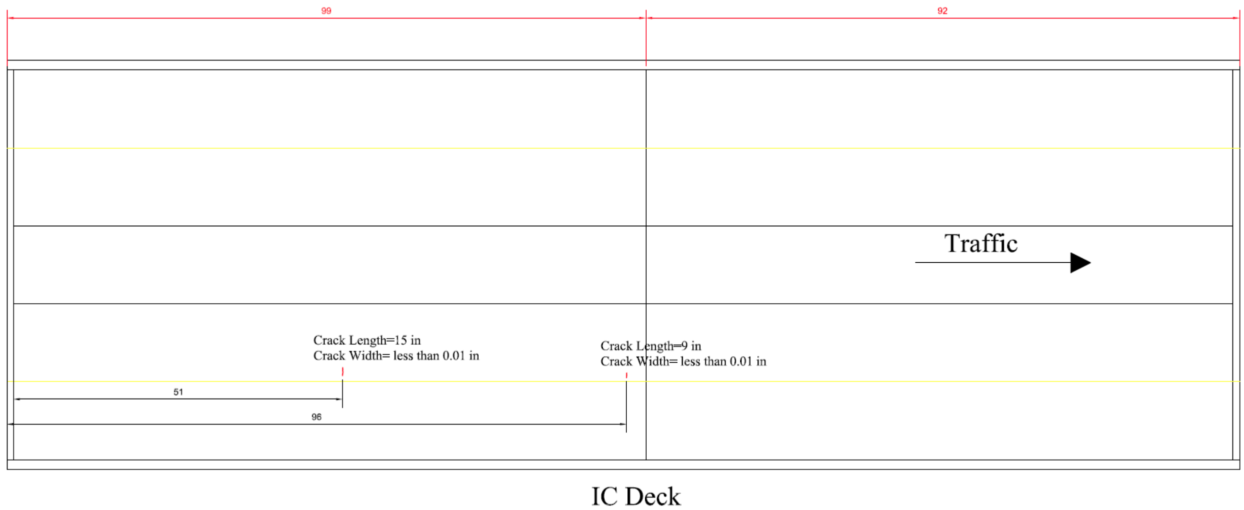


Figure 43. IC concrete deck (northbound) crack map

Considerably less cracking was present on this bridge, in addition to observations of better condition of the surface grooves. As illustrated by these crack maps, the bridge deck with the internal curing mix design outperformed that of the normal concrete mix in all areas observed by the inspection team.

LIFE CYCLE ASSESSMENT

Introduction

Steel reinforcement is susceptible to corrosion in the presence of moisture and oxygen. The rate of corrosion significantly increases with reduced pH due to carbonation and the presence of chlorides that act as catalysts. The dominant mechanism accelerating steel corrosion in bridge decks is chloride ion ingress from external resources like deicing salts.

The determining parameter in evaluating the service life of a reinforced concrete element is the transport properties of the concrete mixture and its exposure to external chlorides such as de-icing salts.

Life-365 ACI Service Life Prediction Model

The Service Life Prediction Model presented in ACI 365.1R-17 helps to evaluate time to initiation (t_i) and the time for propagation (t_p) to reach unacceptable levels of risk. Based on these parameters, it is possible to calculate the repair schedule and perform a life-cycle cost analysis using Life-365 software developed based on ACI 365.1 (ACI Committee 365 2017).

The service life prediction model presented in ACI 365.1 is a simplified approached based on Fickian diffusion. This means it is assumed that no cracking occurs in the concrete element and the dominant chloride ingress mechanism is diffusion.

Accepting these assumptions, it is possible to calculate the chloride content (C), at any determined time (t), at a specific depth from the exposed surface (x) of the concrete element with a known apparent diffusion coefficient (D) and chloride exposure condition using the following equation:

$$\frac{dC}{dt} = D \times \frac{d^2C}{dx^2} \tag{6}$$

Chloride concentration at the surface can be determined based on experimental work (ASTM C1556) or from the Life-365 database (based on the geographic location, type of the structure, and exposure). Apparent diffusion coefficient depends on the age and temperature of the concrete. The reference apparent diffusion coefficient (D_{ref}) at time (t_{ref} = 28 days) and absolute temperature (T_{ref} equals to 293K, 20°C) need to be adjusted using the following equations:

$$D(t) = D_{ref} \times \left(\frac{t_{ref}}{t}\right)^m \tag{7}$$

$$D(T) = D_{ref} \times exp\left[\frac{U}{R} \times \left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right] \tag{8}$$

Where m is a diffusion decay index, (t) is time (days), R is gas constant, U is the activation energy of the diffusion process (35000 J/mol), and T is absolute temperature (K).

Evaluation of Concrete Diffusion Coefficient

The diffusion coefficient of concrete (D_{ref}), critical chloride content (C_t), and diffusion decay index (m) can be measured by direct or indirect test methods, or estimated based on the material properties and mixture proportion design by using the available database. The standardized test method for measuring apparent chloride diffusion coefficient of cementitious materials is ASTM C1556 (ASTM C1556 2016). Some other indirect test methods, like rapid chloride migration, rapid chloride penetrability, and electrical resistivity (AASHTO T 259 2002, AASHTO T 277 2015, AASHTO T 358 2017) have been proposed to evaluate resistance of different concrete mixtures against chloride ion ingress.

Liu et al. (2015) investigated the relationship between apparent diffusion coefficient of concrete mixtures and electrical resistivity, which is illustrated in Figure 44.

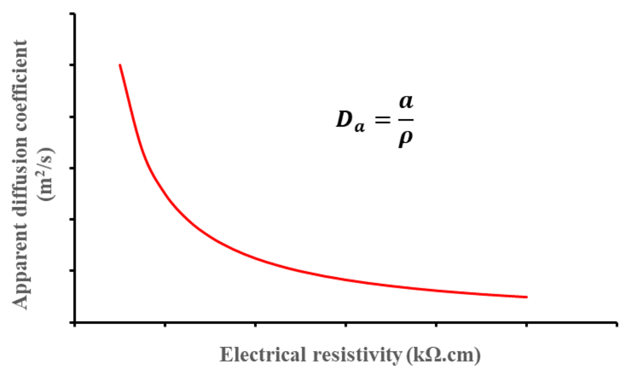


Figure 44. Correlation between diffusion coefficient and resistivity of concrete

The researchers studied a wide range of concrete mixtures containing different supplementary cementitious materials like fly ash (FA) and slag cement, and field samples. It was demonstrated that there was a reliable inverse relationship between diffusion coefficient and resistivity of concrete; the required calibration coefficient (a) was determined based on the cementitious materials incorporated.

Regarding concrete mixtures containing plain Portland cement, it is generally accepted that C_t and m can be assumed as equal to 0.05 percent of the weight of concrete (or around 0.4 percent of the OPC content) and 0.2, respectively (Breit 1998). In addition, D_{ref} (m^2/sec) at 28 days and 20°C can be estimated using the following equation

$$D_{28} = 1 \times 10^{(-12.06+2.4\times w/cm)} \tag{9}$$

After corrosion initiation (t_i), a moderate corrosion rate is considered for control concretes. Corrosion rate mainly depends on the moisture and oxygen diffusivity and electrical resistivity of the concrete material as well as the properties of the reinforcement and early-age cracking of the concrete cover. It can be evaluated using the corrosion current measured in ASTM G109 test method (ASTM G109 2013).

Propagation period (t_p), which is described as the time after (t_i) to the time of the first repair (t_r), is the parameter used as a corrosion rate in the ACI 365 service life prediction model. It can be considered between 3 to 7 years (Weyers 1998) for bridge decks constructed with black reinforcement. There is not enough reliable data available on the performance of epoxy-coated reinforcement in the field conditions. Therefore, as recommended by ACI 365.1, to conduct a comparative study, the propagation period in this study was based on the experience of Ohio bridge engineers and adjusted for internally cured concrete.

Effects of Slag Substitution on Concrete Diffusion Coefficient

Substituting a portion of OPC with supplementary cementitious materials like FA or slag cement does not significantly affect the critical chloride threshold or reference diffusion coefficient; however, it can significantly increase the diffusion decay index (m), which can be adjusted using the following equation:

$$m = 0.2 + 0.4 \times (\%FA/50 + \%slag/70) \tag{10}$$

The maximum replacement ratios of FA and slag cement using this equation are 50 and 70 percent, respectively. Also, the calculated value for (m) cannot exceed 0.6 in any condition. This equation demonstrates that although the concrete mixtures with SCMs may have a similar diffusion coefficient at early ages, it will significantly decrease over time.

Effects of Internal Curing on Concrete Diffusion Coefficient

The results of a study conducted by Bentz (2009) on mortar specimens demonstrated that appropriate internal curing can substantially decrease diffusion coefficient of mortars up to 55 percent of that for the control mixture. Internal curing promotes hydration reactions by providing required moisture through the whole body of the concrete, leading to lower porosity and up to 20 percent more C-S-H at 28-days (Cusson and Margeson 2010). It was assumed that it would not significantly affect diffusion decay index.

As previously discussed, there is a reliable inverse correlation between the diffusion coefficient and resistivity of cement-based materials. The results of the electrical resistivity test (AASHTO T 358 2017) conducted on the field samples with standard curing (Figure 45), indicated that electrical resistivity of internally cured concrete specimens containing 50 percent slag was 1.88 times higher than control concrete at 28 days. Therefore, it can be conservatively estimated that the diffusion coefficient of internally cured concrete slab is decreased to 60 percent of that of the control concrete at 28 days.

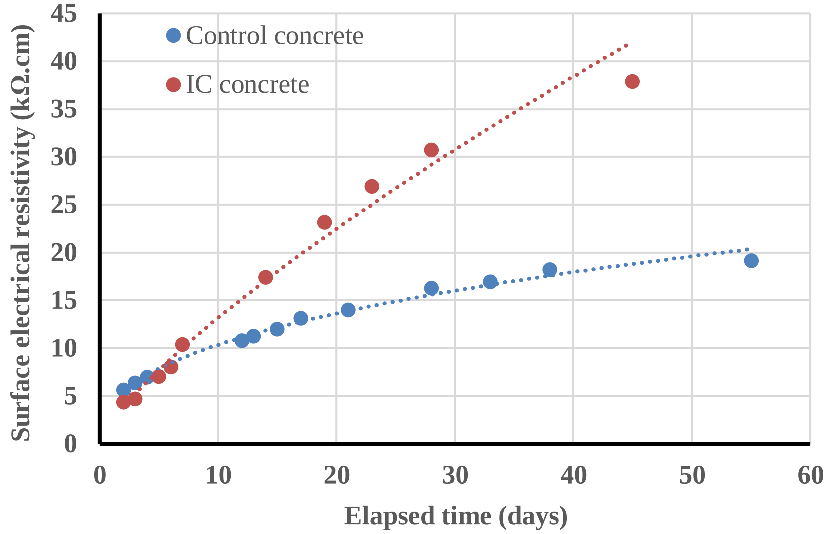


Figure 45. Effect of internal curing on concrete surface electrical resistivity

Service Life Predictions

The mixture designs of control and internally cured concretes used in the field are presented in Table 18.

Table 18. Concrete mixture designs used in field

	OPC (lb/yd^3)	GGBFS (lb/yd^3)	W/cm ratio	Coarse aggregate (lb/yd^3)	Intermediate coarse aggregate (lb/yd^3)	Fine aggregate (lb/yd^3)	LWFA (lb/yd^3)	Air content (%)
Control concrete	410	160	0.45	1360	355	1305	-	6
Internally cured concrete	282	282	0.45	1380	345	940	220	6

The bridge decks were constructed with a 40-year design life in an urban area of Cleveland, Ohio, where the winter weather is cold with heavy snow. Therefore, as the bridge decks are exposed to deicing salts, the surface chloride concentration is estimated to be 0.85 percent of the weight of concrete (ACI Committee 365 2017; Weyers et al. 1993). The surface chloride concentration starts to linearly increase from zero, and reaches the maximum value at 7.4 years; afterwards, it is assumed constant at 0.85 percent.

All reinforcements were epoxy-coated to postpone corrosion propagation. The minimum cover of the reinforcement was 2.5 in., and the thickness of the slab was 8¾ in. (see Figure 46).

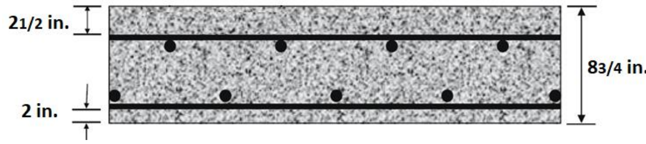


Figure 46. Cross section of the bridge decks

The experience of bridge engineers in Ohio showed that the bridge decks in urban areas need an overlay repair around the age of 25 years to achieve the design life of 40 years. Therefore, a 7-year propagation period was selected for the control mixtures.

Electrical resistivity measurement of the field specimens demonstrated that the resistivity of internally cured concrete (substituting 50 percent slag) is 88 percent higher than control concrete; also, the literature indicated that internal curing increased C-S-H content by 20 percent and decreased water permeability by 20 percent at 28 days age (Cusson and Margeson 2010). Therefore, a longer propagation period can be assumed for the internally cured concrete mixture.

The results of the service life prediction analysis, using Life-365 v 2.2.3, are presented in Table 19.

Table 19. Service life prediction of the bridge decks

	D_{ref} (cm^2/sec)	m	C_t	Initiation period (year)	Propagation period (year)	Service life prediction (year)
Control concrete - 30% slag	1.6231E-8	0.37	0.050	18.5	7	25.5
Internally cured concrete - 50% slag	9.7386E-9	0.49	0.050	64.6	7	71.6

The results indicated that increasing the slag substitution percentage from 30 percent to 50 percent and using internal curing significantly increased the service life from 25.5 to 71.6 years.

Figure 47 demonstrates the variation of the diffusion coefficient of the mixtures based on the elapsed time and temperature variation in Cleveland, Ohio.

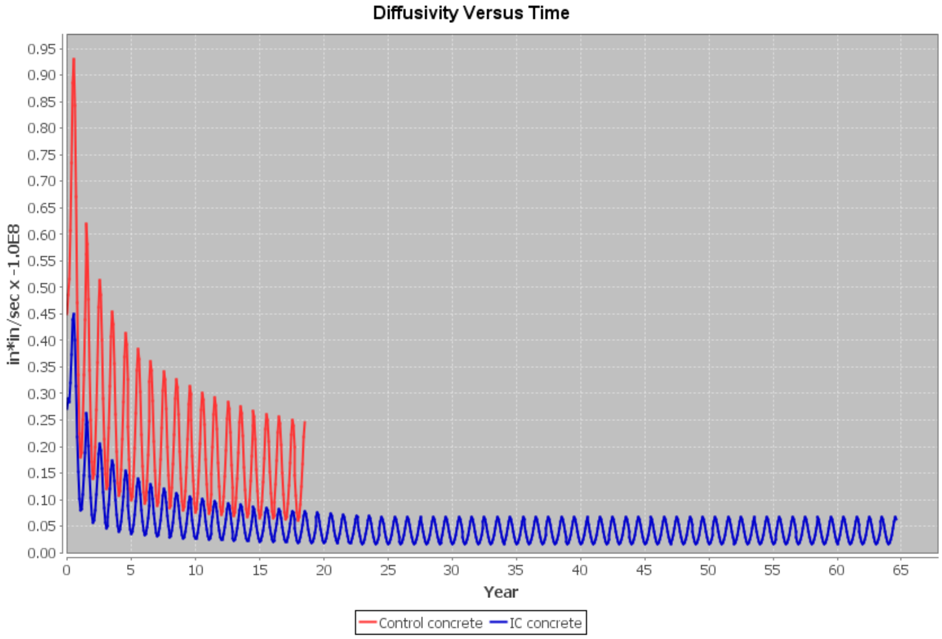


Figure 47. Variation of the diffusion coefficient of mixtures

As presented, the diffusion coefficient generally decreased over time. Internally cured mixture had a lower 28-day diffusion coefficient, which decreased even more rapidly over time because of higher substitution of slag. It was assumed that hydration reactions effectively stop after 25 years, so the diffusion coefficient no longer decreases.

Using all the parameters discussed, it was possible to estimate the chloride ion content at 2.5-in. depth (concrete cover) over time, as illustrated in Figure 48.

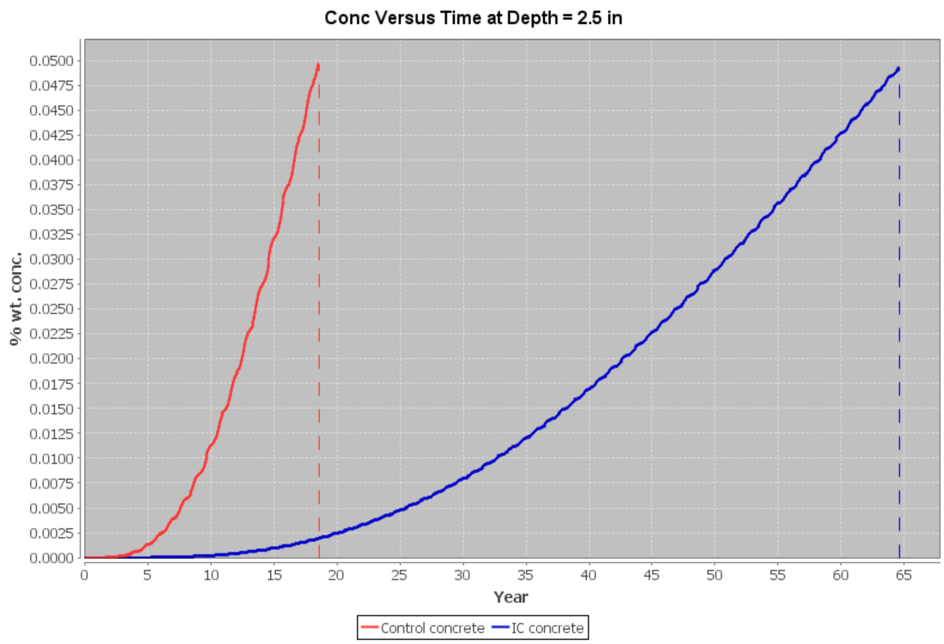


Figure 48. Chloride ion content at 2.5-inch depth over time

The time when the chloride ion concentration at 2.5-inch depth reached the threshold value is the onset of the corrosion propagation period. As demonstrated, the internally cured mixture was expected to have a longer corrosion initiation time.

Figure 49 summarizes the service life prediction analysis that was carried out.

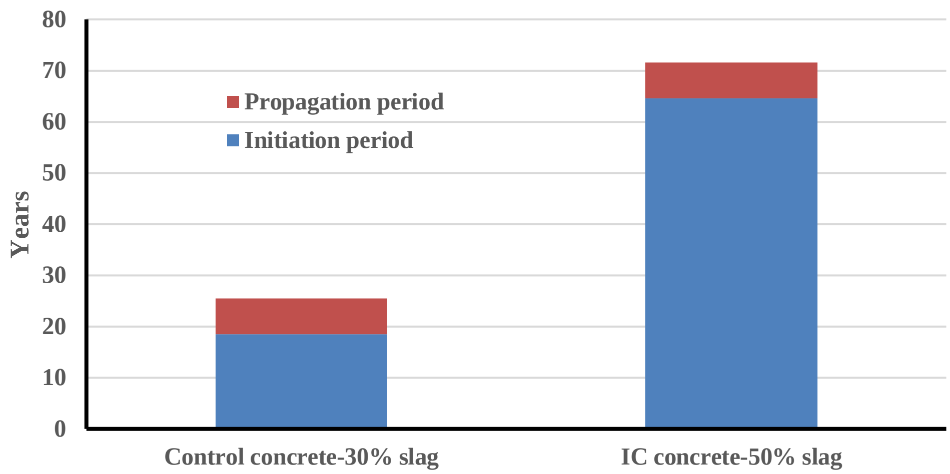


Figure 49. Service life prediction of the alternatives

Internally cured concrete was expected to have longer corrosion initiation period while the propagation period was conservatively assumed constant. It can be concluded that the service life of internally cured concrete is almost three times longer than the control concrete.

Life-Cycle Cost Analysis (LCCA)

A 100 by 100-ft section was considered for the life-cycle cost analysis (LCCA) study. The analysis period was assumed to be two-and-a-half times longer than the design life, and they equaled 100 and 40 years, respectively. Repair time interval, area to repair, and repair cost are assumed constant for all cases and equal to 10 years, 10 percent of the total area, and 37 $/ft^2, respectively.

The base year was assumed as 2017, inflation rate at 1.8 percent, and discount rate at 2 percent. The results of the LCCA including initial construction cost ($), repair cost ($), and total life-cycle cost ($) are indicated in Table 20 for all three alternatives.

Table 20. Life-cycle cost of alternatives per the unit area of the bridge deck

	Concrete unit volume cost ($/CY)	Initial construction cost ($)	Repair cost ($)	Life-cycle cost ($/ft^2)
Control concrete - 30% slag	470	153,266	151,868	305,134
Internally cured concrete - 50% slag	550	174,994	43,158	218,152

As demonstrated, the total life-cycle cost of internally cured concrete incorporating 50 percent slag is around 29 percent less than the original concrete.

Figure 50 demonstrates cumulative current costs in the LCCA. As presented, original control concrete incorporating 30 percent slag had the higher current dollar per square foot rate while the internally cured alternative exhibited the lower rate.

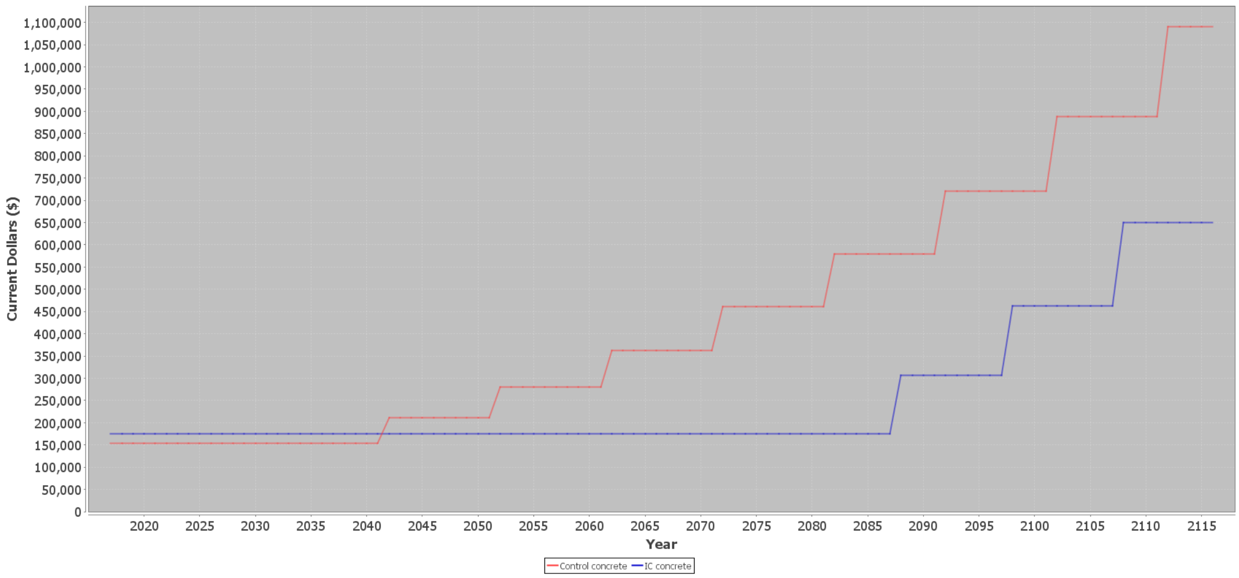


Figure 50. Cumulative current costs in LCCA

Sensitivity analyses of the discount rate (%) and repair cost ($) are represented in Figure 51 and 52.

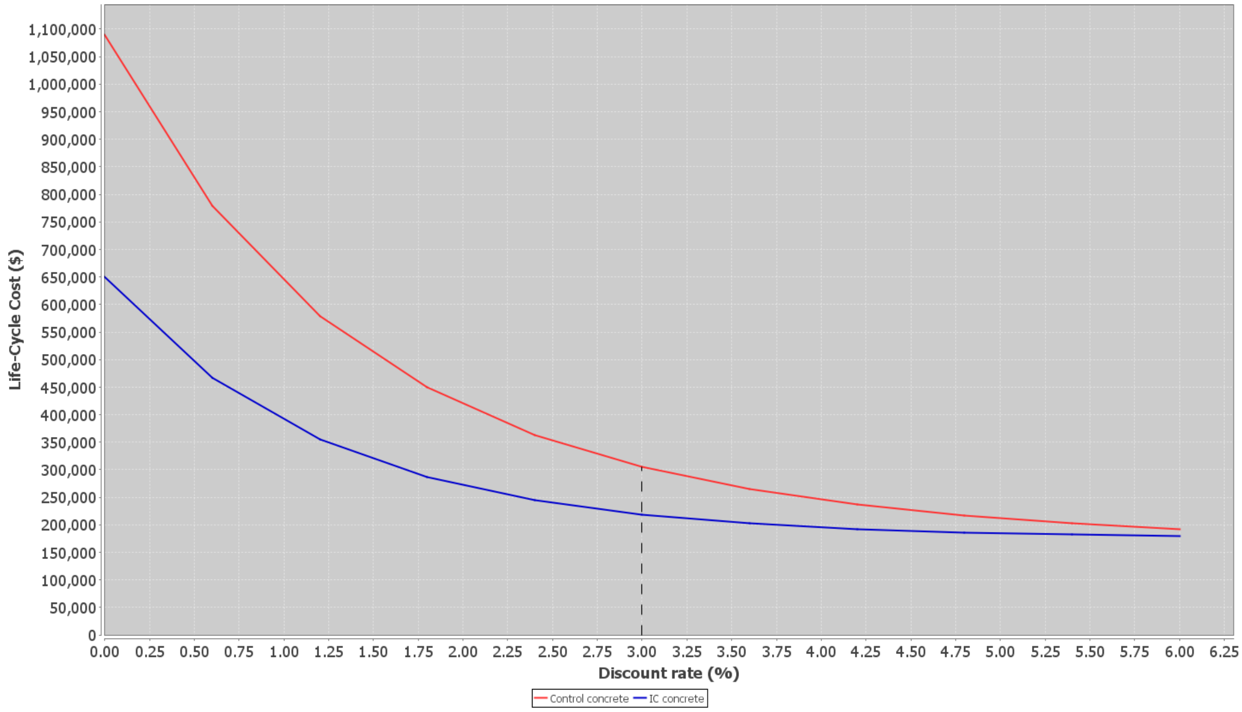


Figure 51. Sensitivity analysis to discount rate (%)

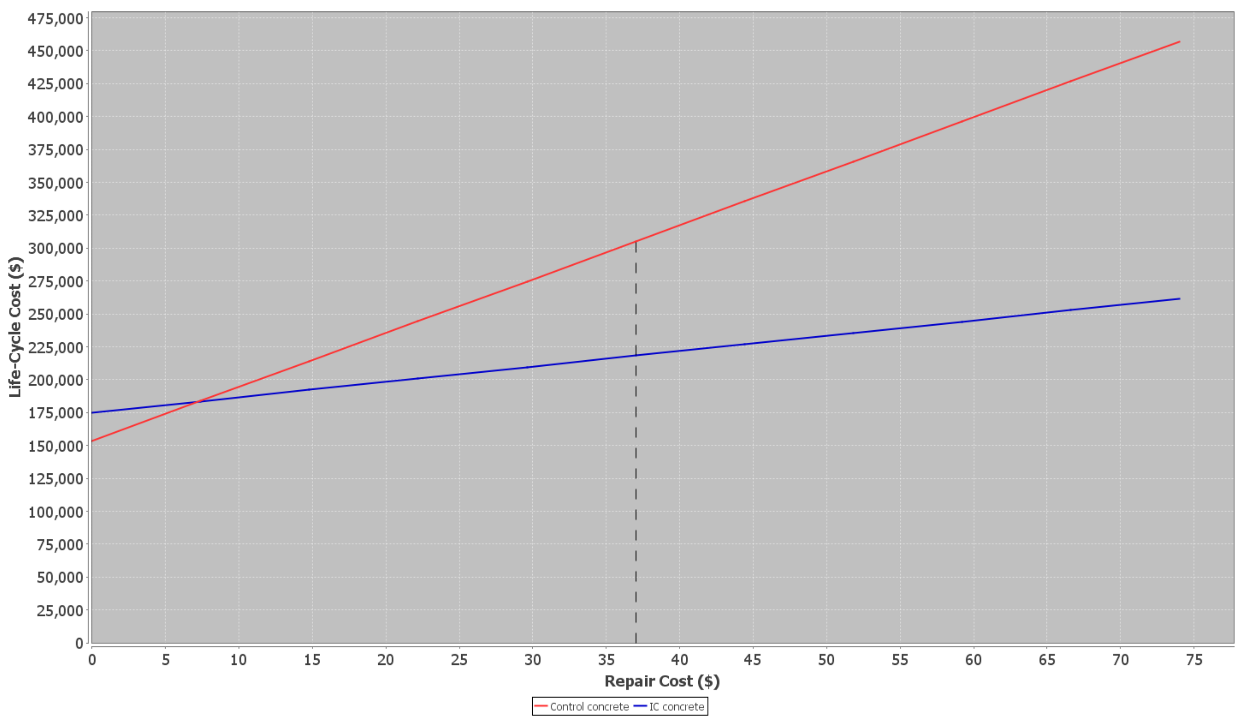


Figure 52. Sensitivity analysis to repair cost ($)

As illustrated, the savings in the life-cycle cost of using internally cured concrete is more significant considering low discount rate. However, any discount rates lower than 5% shows a remarkable benefit for using internally cured concrete. Repair cost is a major expense over the analysis period and using internally cured concrete can help to decrease and postpone this expense; therefore, increasing repair cost results in making the internal curing option more beneficial.

Figure 53 summarizes the results of the LCCA.

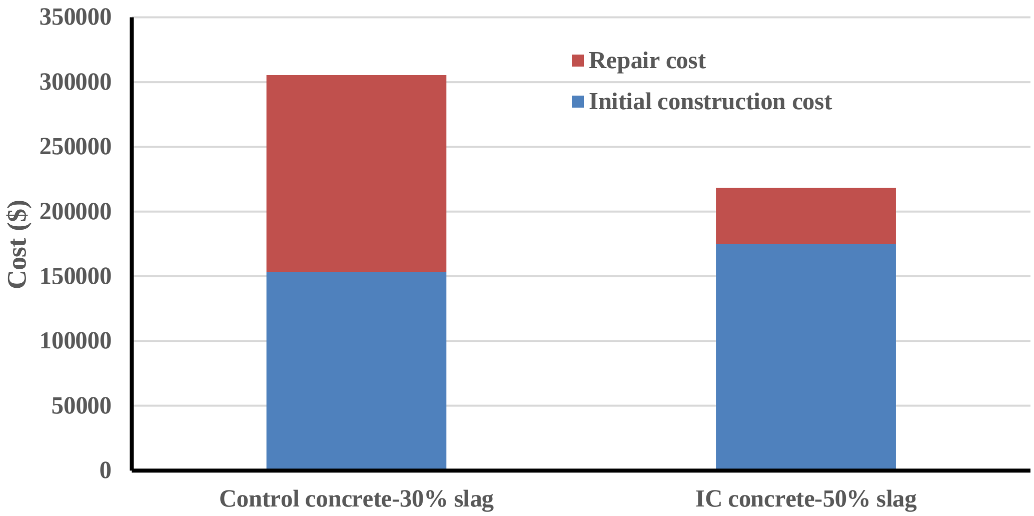


Figure 53. Initial construction and repair costs of the alternatives

The internally cured concrete mixture containing higher slag substitution had a higher initial construction cost; however, it had a significantly lower current life-cycle cost due to its significantly lower repair cost.

Conclusion

Increasing the substitution percentage of slag and internal curing are two methods that can significantly decrease the chloride ion diffusion coefficient of concrete materials leading to increased service life of the bridge decks. Internal curing leads to around a 60 percent decrease in the reference diffusion coefficient of the mixture, while increasing slag substitution from 30 percent to 50 percent increases the diffusion decay index from 0.37 to 0.49.

On the other hand, the internally cured concrete mixture showed superior performance in resistance against chloride ion penetration and reinforcement corrosion, leading to huge savings in the repair cost. The ultimate life-cycle cost is about 29 percent less than the control concrete mixture.

It should be noted that the life cycle analysis was based on the concrete being crack free. While this has been demonstrated to be true for the test deck, it was not true for the control deck, thus increasing the potential benefit of the recommended mixture.

Implementation

The laboratory and field work have demonstrated that careful selection of w/cm, binder type and dose, and the inclusion of internal curing can all help to reduce the risk of cracking, and increase the longevity of a concrete bridge deck. However some challenges have to be addressed if internal curing is to be implemented more widely. These are discussed below.

Availability.

Reportedly, there are two suppliers of suitable lightweight fine aggregate in neighboring states and two others in New York. This does mean that haulage costs have to be accommodated, as discussed in a following section. The competition does mean that materials costs will be kept in check.

Handling

The biggest barrier to use of internal curing is the effort required to stockpile an additional material, and to keep it moist during batching. However, for a bridge deck, the volume of material is relatively low and enough material can be stored and conditioned in a single bin. Again, the cost of this work will have to be taken into account.

The Unknown

As with any new technology, there is always a concern regarding how dealing with a new material or process will affect the constructability of a project. This concern may be reflected in elevated prices initially. The demonstration discussed above has shown that while some procedures may need adjustment, the added risk to the contractor appears to be limited, thus, as familiarity grows, prices should drop.

Cost

All of the challenges noted above have an impact on cost. For the project discussed above, the reported bid price for the conventional concrete was $470 and for the IC concrete was $550 pcy. It is likely that a large portion of this premium was related to the handling and “unknown” aspects, which will likely drop significantly with increasing familiarity with the materials and processes required. As noted in the lifecycle cost analysis, despite this premium, the benefits still outweigh the costs.

Next Steps

It is suggested that the DOT continue to specify and build decks using this technology, with the aim of increasing familiarity and to gather information on constructability and durability of these systems.

KEY FINDINGS

The following points can be concluded from the work conducted:

- All of the mixtures tested in the laboratory provided adequate strength
- All of the mixtures were sufficiently impermeable
- As the amount of slag in the mixture increased, shrinkage and cracking risk decreased
- Mixtures containing LWFA exhibited lower shrinkage and cracking risk than mixtures without LWFA
- Mixtures containing SRA exhibited lower shrinkage and cracking risk than mixtures without SRA
- There was no consistent difference in structural response between the two bridges
- In terms of cracking after one year, the bridge deck with the internal curing mix design outperformed that of the normal concrete mix in all areas observed by the inspection team
- Life cycle assessment of the recommended approach demonstrated that internal curing will provide long-term savings despite higher costs at the time of construction

Suggested Specification language has been provided in the Appendix.

REFERENCES

ACI. 2008. *Guide for Conducting a Visual Inspection of Concrete in Service*. 201.1 R-08. American Concrete Institute, Farmington Hills, MI.

——. 2013. *Report on Internally Cured Concrete Using Prewetted Absorptive Lightweight Aggregate*. (308-213)R-13. American Concrete Institute, Farmington Hills, MI.

ACI Committee 365. 2017. *Report on Service Life Prediction*. 365.1R-17. American Concrete Institute, Farmington Hills, MI.

Alexander, M. G., J. R. Mackechnie, and Y. Ballim. 2001 (1999; revised 2001). *Guide to the Use of Durability Indexes for Achieving Durability in Concrete Structures*. Research Monograph No. 2. University of Cape Town and University of Witwatersrand, South Africa.

Bae, Jewan. 2004. Admixture Controls Slab Drying Shrinkage. *Concrete International*, Vol. 26, No. 9, pp. 88–89.

Barrett, T., A. Miller, and J. Weiss. 2014. *Internal Curing – Lessons Learned and Inspiration from Walt Disney*. Internal Curing Summit, April 23. http://www.cptechcenter.org/ncc/TTCC-NCC-documents/Sp2014/SP14-22-Weiss-Internal%20Curing-sec.pdf.

Bates, R. T., E. Holck, M. Dee, and M. King. 2012. Design and Construction of an Internally Cured Slab. SP290-5. In *The Economics, Performance, and Sustainability of Internally Cured Concrete*. American Concrete Institute, Farmington Hills, MI.

Bentur, A., S-i. Igarashi, and K. Kovler. 2001. Prevention of Autogenous Shrinkage in High-Strength Concrete by Internal Curing Using Wet Lightweight Aggregates. *Cement and Concrete Research*. Vol. 31, No. 11, pp. 1587–1591.

Bentz, D. P. 2006. Influence of Shrinkage-Reducing Admixtures on Early-Age Properties of Cement Pastes. *Journal of Advanced Concrete Technology*, Vol. 4, No. 3, pp. 423–429.

——. 2007. Internal Curing of High-Performance Blended Cement Mortars. *ACI Materials Journal*, Vol. 104, No. 4, pp. 408–414.

——. 2009. Influence of Internal Curing using Lightweight Aggregates on Interfacial Transition Zone Percolation and Chloride Ingress in Mortars. *Cement and Concrete Composites*, Vol. 31, No. 5, pp. 285–289.

Bentz, D. P., M. R. Geiker, and K. K. Hansen. 2001. Shrinkage-Reducing Admixtures and Early-Age Desiccation in Cement Pastes and Mortars. *Cement and Concrete Research*, Vol. 31, No. 7, pp. 1075–1085.

Bentz, D. P., P. Lura, and J. W. Roberts. 2005. Mixture Proportioning for Internal Curing. *Concrete International*, Vol. 27, No. 2, pp. 35–40.

Bentz, D. P., E. A. B. Koenders, S. Mönnig, H.-W. Reinhardt, K. van Breugel, and G. Ye. 2007. Materials Science-Based Models. In *Internal Curing of Concrete - State-of-the-Art Report of RILEM Technical Committee 196-ICC*, pp. 29–43.

Bentz, D. and J. Weiss. 2011. *Internal Curing: A 2010 State-of-the-Art Review.* NISTIR 7765. U.S. Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.

Bentz, D. P., J. M. Davis, M. A. Peltz, and K. A. Snyder. 2014. Influence of Internal Curing and Viscosity Modifiers on Resistance to Sulfate Attack. *Materials and Structures,* Vol. 47, No. 4. pp. 581–589.

Breit, W. 1998. Critical Chloride Content-Investigations of Steel in Alkaline Chloride Solutions. *Materials and Corrosion*, Vol. 49, No. 8, pp. 539–550.

Brooks, J. J., M. A. M. Johari, and M. Mazloom. 2000. Effect of Admixtures on the Setting Times of High-Strength Concrete. *Cement and Concrete Composites*, Vol. 22, No. 4, pp. 293–301.

Byard, B. E. and J. Ries. 2012. Optimizing the Sustainability of Concrete through Internal Curing. SP 290-1. In *The Economics, Performance, and Sustainability of Internally Cured Concrete*. American Concrete Institute, Farmington Hills, MI.

Byard, B. and A. Schindler. 2010. *Cracking Tendency of Lightweight Concrete*. Highway Research Center, Auburn University, Auburn, AL, and The Expanded Shale, Clay, and Slate Institute, Chicago, IL.

Byard, B. E., A. K. Schindler, and R. W. Barnes. 2012. Early-Age Cracking Tendency and Ultimate Degree of Hydration of Internally Cured Concrete. *Journal of Materials in Civil Engineering*, Vol. 24, No. 8, pp. 1025–1033.

Castro, J., I. De la Varga, M. Golias, and W. J. Weiss. 2010. Extending Internal Curing Concepts (Using Fine LWA) to Mixtures Containing High Volumes of Fly Ash. Paper presented at 2010 Concrete Bridge Conference: Achieving Safe, Smart & Sustainable Bridges, February 24–26, Phoenix, AZ.

Castro, J., L. Keiser, M. Golias, and J. Weiss. 2011. Absorption and Desorption Properties of Fine Lightweight Aggregate for Application to Internally Cured Concrete Mixtures. *Cement and Concrete Composites*, Vol. 33, No. 10, pp. 1001–1008.

Castrodale, R. W. 2014. Internal Curing with Lightweight Aggregate for Transportation Structures and Pavements. Paper presented at the North Eastern States' Materials Engineers Association (NESMEA) Conference, October 21, Framingham, MA. http://www.nesmea.uconn.edu/2014-conference-presentations-framingham-ma.

Craeye, B., M. Geirnaert, and G. De Schutter. 2011. Super Absorbing Polymers as an Internal Curing Agent for Mitigation of Early-Age Cracking of High-Performance Concrete Bridge Decks. *Construction and Building Materials*, Vol. 25, No. 1, pp. 1–13.

Cusson, D. and T. Hoogeveen. 2008. Internal Curing of High-Performance Concrete with Pre-Soaked Fine Lightweight Aggregate for Prevention of Autogenous Shrinkage Cracking. *Cement and Concrete Research*, Vol. 38, No. 6, pp. 757–765.

Cusson, D., Z. Lounis, and L. Daigle. 2010. Benefits of Internal Curing on Service Life and Life-Cycle Cost of High-Performance Concrete Bridge Decks – A Case Study. *Cement and Concrete Composites*, Vol. 32, No. 5, pp. 339–350.

Cusson, D. and J. Margeson. 2010. Development of Low-Shrinkage High-Performance Concrete with Improved Durability. In *Concrete under Severe Conditions*. Taylor & Francis Group, London.

Daigle, L., D. Cusson, and Z. Lounis. 2008. Extending Service Life of High Performance Concrete Bridge Decks with Internal Curing. Paper presented at the 8th International Conference on Creep, Shrinkage and Durability of Concrete and Concrete Structures (CONCREEP 8), September 30–October 2, ISE-Shima, Japan.

D'Ambrosia, M. D., J. Slater, and T. Van Dam. 2013. *High-Performance Concrete for Bridge Decks*. CTL Group, Skokie, IL. https://www.illinoistollway.com/documents/20184/239395/High-Performance+Concrete+for+Bridge+Decks-Final+Report.pdf/75102f31-f874-4f95-98d3-a626bfaad247?version=1.0.

Darwin, D. 2014. Control of Cracking in Concrete. Paper presented at the 50th Annual Concrete Conference, South Dakota School of Mines & Technology, March 7, Rapid City, SD.

Darwin, D., J. Browning, W. Lindquist, H. McLeod, J. Yuan, M. Toledo, and D. Reynolds. 2010. Low-Cracking, High-Performance Concrete Bridge Decks: Case Studies over First 6 Years. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2202, pp. 61–69.

Darwin, D., J. Browning, H. A. K. McLeod, W. Lindquist, and J. Yuan. 2011. Implementing Lessons Learned From Twenty Years of Bridge-Deck Crack Surveys. ACI SP 284-8. In *Proceedings of the Andy Scanlon Symposium on Serviceability and Safety of Concrete Structures: From Research to Practice 2011*, October 16–20, Cincinnati, OH.

De la Varga, I., J. Castro, D. Bentz, and J. Weiss. 2012. Application of Internal Curing for Mixtures Containing High Volumes of Fly Ash. *Cement and Concrete Composites*, Vol. 34, No. 9, pp. 1001–1008.

ESCSI. 2012. ESCSI Guide Specifications for Internally Cured Concrete. Information Sheet 4001.1. Expanded Shale, Clay and Slate Institute, Chicago, IL. https://www.utelite.com/wp-content/uploads/2018/06/ESCSIinternal.pdf.

ESCSI. 2018. Applications / Internal Curing Team Approach. Expanded Shale, Clay and Slate Institute, Chicago, IL. https://www.escsi.org/internal-curing/ic-team-approach/.

Folliard, K. J. and N. S. Berke. 1997. Properties of High-Performance Concrete Containing Shrinkage-Reducing Admixture. *Cement and Concrete Research*, Vol. 27, No. 9. pp. 1357–1364.

Geiker, M. R., D. P. Bentz, and O. M. Jensen. 2004. Mitigating Autogeneous Shrinkage by Internal Curing. SP 218-9. In *High-Performance Structural Lightweight Concrete*. American Concrete Institute, Farmington Hills, MI.

Golias, M., J. Castro, and J. Weiss. 2012. The Influence of the Initial Moisture Content of Lightweight Aggregate on Internal Curing. *Construction and Building Materials,* Vol. 35, pp. 52–62.

Henkensiefken, R., J. Castro, D. Bentz, T. Nantung, and J. Weiss. 2009a. Water Absorption in Internally Cured Mortar made with Water-Filled Lightweight Aggregate. *Cement and Concrete Research*. Vol. 39, No. 10, pp. 883–892.

Henkensiefken, R., D. Bentz, T. Nantung, and J. Weiss. 2009b. Volume Change and Cracking in Internally Cured Mixtures Made with Saturated Lightweight Aggregate under Sealed and Unsealed Conditions. *Cement and Concrete Composites*, Vol. 31, No. 7, pp. 427–437.

Hoff, George C. 2002. *The Use of Lightweight Fines for the Internal Curing of Concrete*. Northeast Solite Corporation. Richmond, VA.

Hossain, A. B. and J. Weiss. 2004. Assessing Residual Stress Development and Stress Relaxation in Restrained Concrete Ring Specimens. *Cement and Concrete Composites*, Vol. 26, No. 5, pp. 531–540.

Hossain, A. B. and J. Weiss. 2006. The Role of Specimen Geometry and Boundary Conditions on Stress Development and Cracking in the Restrained Ring Test. *Cement and Concrete Research*, Vol. 36, No. 1, pp. 189–199.

Jongvisuttisun, P., C. Negrello, and K. E. Kurtis. 2012. Efficiency of Eucalyptus Pulps for Internal Curing. SP290-3. In *The Economics, Performance, and Sustainability of Internally Cured Concrete*. American Concrete Institute, Farmington Hills, MI.

Klieger, P. 1957. Early High-Strength Concrete for Prestressing. In *Proceedings of the World Conference on Prestressed Concrete*, San Francisco, CA. https://archive.org/details/EarlyHigh-strengthConcreteForPrestressing/page/n0.

Kosmatka, S. and M. L. Wilson. 2016. *Design and Control of Concrete Mixtures: The Guide to Applications, Methods, and Materials*. 16th Edition. Portland Cement Association, Skokie, IL.

Kovler, K., A. Souslikov, and A. Bentur. 2004. Pre-Soaked Lightweight Aggregates as Additives for Internal Curing of High-Strength Concretes. *Cement, Concrete, and Aggregates*. Vol. 26, No. 2, pp. 1–8.

Ley, T., D. Cook, and G. Fick. 2012. *Concrete Pavement Mixture Design and Analysis (MDA): Effect of Aggregate Systems on Concrete Properties*. National Concrete Pavement Technology Center, Iowa State University, Ames, IA.

Lindquist, W. D., D. Darwin, and J. Browning. 2005. *Cracking and Chloride Contents in Reinforced Concrete Bridge Decks*. University of Kansas Center for Research, Inc. Lawrence, KS.

Liu, X. and M.-H. Zhang. 2010. Permeability of High-Performance Concrete Incorporating Presoaked Lightweight Aggregates for Internal Curing. *Magazine of Concrete Research*. Vol. 62, No. 2, pp. 79–89.

Liu, Y., F. J. Presuel-Moreno, and M. A. Paredes. 2015. Determination of Chloride Diffusion Coefficients in Concrete by Electrical Resistivity Method. *ACI Materials Journal*. Vol. 112, No. 5, pp. 631–640.

Lopes, A. N. M., E. F. Silva, D. C. C. Dal Molin, and R. D. Toledo Filho. 2013. Shrinkage-Reducing Admixture: Effects on Durability of High-Strength Concrete. *ACI Materials Journal*. Vol. 110, No. 4, pp. 365–374.

Maggenti, R., C. Knapp, and S. Fereira. 2013. Controlling Shrinkage Cracking. *Concrete International*, pp. 36–41. http://149.136.20.42/hq/esc/construction/manuals/OSCCompleteManuals/ControlShrinkCracking-ACI_CT.pdf.

Masten, M. 2015. *NCC State Reports Spring 2015–Curing Survey*. National Concrete Consortium Spring Meeting, April 21, Reno, NV. http://www.cptechcenter.org/ncc/Sp2015%20NCC/03%20Masten%20Spring%202015%20-%20NCC%20State%20Reports%20Curing2.pdf.

Mechtcherine, V. and H-W. Reinhardt, editors. 2012. *Application of Super Absorbent Polymers (SAP) in Concrete Construction*. Report by Technical Committee 225-SAP. RILEM. Bagneux, France.

Mora-Ruacho, J., R. Gettu, and A. Aguado. 2009. Influence of Shrinkage-Reducing Admixtures on the Reduction of Plastic Shrinkage Cracking in Concrete. *Cement and Concrete Research*, Vol. 39, No. 3, pp. 141–146.

Nmai, C. K., R. Tomita, F. Hondo, and J. Buffenbarger. 1998. Shrinkage-Reducing Admixtures. *Concrete International*, Vol. 20, No. 4, pp. 31–37.

NYSDOT. 2018. Revision to Standard Specifications Section 557–High Performance Internal Curing Concrete. EB 18-003, New York State Department of Transportation, Albany, NY.

Ozawa, M. and H. Morimoto. 2012. Reduction of Autogenous Shrinkage in Cement Paste by IC Using Jute Fiber. SP290-4. In *The Economics, Performance, and Sustainability of Internally Cured Concrete*. American Concrete Institute, Farmington Hills, MI.

Paul, Á. and M. Lopez. 2011. Assessing Lightweight Aggregate Efficiency for Maximizing Internal Curing Performance. *ACI Materials Journal*. Vol. 108, No. 4, pp. 385–393.

Rajabipour, F., G. Sant, and J. Weiss. 2008. Interactions between Shrinkage Reducing Admixtures (SRA) and Cement Paste's Pore Solution. *Cement and Concrete Research*. Vol. 38, No. 5, pp. 606–615.

Rao, C. and M. I. Darter. 2013. *Evaluation of Internally Cured Concrete for Paving Applications*. Applied Research Associates, Inc., Champaign, IL.

Şahmaran, M., M. Lachemi, K. M. A. Hossain, and V. C. Li. 2009. Internal Curing of Engineered Cementitious Composites for Prevention of Early Age Autogenous Shrinkage Cracking. *Cement and Concrete Research*. Vol. 39, No. 10, pp. 893–901.

Sant, G., A. Eberhardt, D. Bentz, and J. Weiss. 2010. Influence of Shrinkage-Reducing Admixtures on Moisture Absorption in Cementitious Materials at Early Ages. *Journal of Materials in Civil Engineering*, Vol. 22, No. 3, pp. 277–286.

Schlitter, J., R. Henkensiefken, J. Castro, K. Raoufi, J. Weiss, and T. Nantung. 2010. *Development of Internally Cured Concrete for Increased Service Life*. Indiana Department of Transportation, Division of Research, and Purdue University Joint Transportation Research Program, West Lafayette, IN.

See, H. T., E. K. Attiogbe, and M. A. Miltenberger. 2003. Shrinkage Cracking Characteristics of Concrete using Ring Specimens. *ACI Materials Journal*, Vol. 100, No. 3, pp. 239–245.

Siriwatwechakul, W., J. Siramanont, and W. Vichit-Vadakan. 2012. *Ion Filtration Effect of Superabsorbent Polymers for Internal Curing*. ACI SP289. American Concrete Institute, Farmington Hills, MI.

Streeter, D. A., W. H. Wolfe, and R. E. Vaughn. 2012. Field Performance of Internally Cured Concrete Bridge Decks in New York State. SP290-7. In *The Economics, Performance, and Sustainability of Internally Cured Concrete.* American Concrete Institute, Farmington Hills, MI.

Suzuki, M., M. S. Meddah, and R. Sato. 2009. Use of Porous Ceramic Waste Aggregates for Internal Curing of High-Performance Concrete. *Cement and Concrete Research*. Vol. 39, No. 5, pp. 373–381.

Taylor, P., T. Hosteng, X. Wang, and B. Phares. 2016. *Evaluation and Testing of a Lightweight Fine Aggregate Concrete Bridge Deck in Buchanan County, Iowa*. National Concrete Pavement Technology Center and Bridge Engineering Center, Iowa State University. Ames, IA.

Taylor, P., P. Tikalsky, K. Wang, G. Fick, and X. Wang. 2012. *Development of Performance Properties of Ternary Mixtures: Field Demonstrations and Project Summary*. National Concrete Pavement Technology Center, Iowa State University, Ames, IA.

Taylor, P. and X. Wang. 2014. *Concrete Pavement Mixture Design and Analysis (MDA): Factors Influencing Drying Shrinkage Literature Review*. National Concrete Pavement Technology Center, Iowa State University, Ames, IA.

Taylor, P., E. Yurdakul, X. Wang, and X. Wang. 2015. *Concrete Pavement Mixture Design and Analysis (MDA): An Innovative Approach to Proportioning Concrete Mixtures Technical Report.* National Concrete Pavement Technology Center, Iowa State University, Ames, IA.

Wang, X., K. Wang, F. Bektas, and P. Taylor. 2012. Drying Shrinkage of Ternary Blend Concrete in Transportation Structures. *Journal of Sustainable Cement-Based Materials*, Vol. 1, Nos. 1–2, pp. 56–66.

Wang, X., P. Taylor, K. Wang, and M. Lim. 2016. Monitoring of Setting Time of Self-Consolidating Concrete Using Ultrasonic Wave Propagation Method and Other Tools. *Magazine of Concrete Research*, Vol. 68, No. 3, pp. 151–162.

Wei, Y., Y. Xiang, and Q. Zhang. 2014. Internal Curing Efficiency of Prewetted LWFAs on Concrete Humidity and Autogenous Shrinkage Development. *Journal of Materials in Civil Engineering*, Vol. 26, No. 5, pp. 947–954.

Weiss, W. J., W. Yang, and S. P. Shah. 1998. Shrinkage Cracking of Restrained Concrete Slabs. *Journal of Engineering Mechanics*, Vol. 124, No.7. pp. 765–773.

Weiss, W. J. and L. Montanari. 2017. *Guide Specification for Internally Curing Concrete*. National Concrete Pavement Technology Center, Iowa State University, Ames, IA.

Weyers, R. E. 1998. Service Life Model for Concrete Structures in Chloride Laden Environments. *ACI Materials Journal*, Vol. 95. No. 4, pp. 445–453.

Weyers, R. E., B. D. Prowell, M. M. Sprinkel, and M. Vorster. 1993. *Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosion: A Methods Application Manual*. SHRP-S-360. Strategic Highway Research Program, Washington, DC.

Wilson, W. O. 2013. Mitigation of Early Age Cracking of Concrete for Bridge Decks. MS thesis. University of Illinois at Urbana-Champaign, Urbana, IL.

Wyrzykowski, M., P. Lura, and D. Gawin. 2012. Modeling of Internal Curing with SAP at Meso- and Macro-Level. SP290-12. In *The Economics, Performance, and Sustainability of Internally Cured Concrete*. American Concrete Institute, Farmington Hills, MI.

Yurdakul, E. 2010. Optimizing Concrete Mixtures with Minimum Cement Content for Performance and Sustainability. MS thesis. Civil, Construction, and Environmental Engineering, Iowa State University, Ames, IA.

Zhutovsky, S. and K. Kovler. 2012. Effect of Internal Curing on Durability-Related Properties of High Performance Concrete. *Cement and Concrete Research*, Vol. 42, No. 1, pp. 20–26.

APPENDIX: SUGGESTED SPECIFICATION LANGUAGE

STATE OF OHIO

DEPARTMENT OF TRANSPORTATION

SUPPLEMENTAL SPECIFICATION YYY INTERNALLY CURING CONCRETE WITH LIGHTWEIGHT AGGREGATE FOR BRIDGE DECKS

2018

Description. This item shall consist of the materials and mix design for internally curing concrete with lightweight aggregate according to the requirements of Section XXX of the Standard Specifications, and the following.

Materials. The Contractor shall replace a portion of the normal weight fine aggregate with pre-wetted lightweight fine aggregate to provide water for internally curing the concrete as specified herein. The lightweight aggregate shall be an expanded shale, expanded blast furnace slag, expanded slate, or expanded clay product according to ASTM C1761. The lightweight fine aggregate shall be Gradation XXX. Storage of lightweight aggregate shall be according to Article XXX, except the stockpile shall be on a sloped surface. Lightweight aggregate stockpiles shall be uniformly wetted with a sprinkler system for a minimum 72 hours, and then allowed to drain for 22 ± 2 hours immediately prior to use. Lightweight aggregate from different sources shall not be mixed without permission of the Engineer.

Proportioning and Mix Design. Proportioning and mix design shall be for Class XXX concrete and as follows.

- Water/Cement Ratio. The water/cement ratio shall not be less than 0.36.
- Paste Content. The total cement plus finely divided minerals and water content shall not exceed 26% by volume of the mix design. The minimum cement factor may be reduced to 5.80 cwt/cu yd (345 kg/cu m).
- Volume of Lightweight Aggregate. The pre-wetted lightweight aggregate shall replace a minimum 30 percent, by volume, of the normal weight fine aggregate.
- Batching. Immediately prior to batching, the pre-wetted and drained lightweight aggregate shall have a field absorbed moisture content value not less than 15 percent. The field absorbed moisture content shall be determined. Stockpiles that do not achieve the minimum degree of absorption shall receive additional wetting and be allowed to drain for a minimum 12 hours prior to determining field absorbed moisture content again.

Trial Batch. For a new mix design to be verified, the Engineer will require the Contractor to provide a trial batch at no cost to the Department. The trial batch shall be scheduled a minimum 30 calendar days prior to anticipated use and shall be performed in the presence of the Engineer. A minimum of 2 cu yd (1.5 cu m) trial batch shall be produced and placed offsite. The trial batch shall be produced with the equipment, materials, and methods intended for construction. The trial batch will be evaluated and tested by the Engineer according to the "Portland Cement Concrete Level III Technician" course manual. The Engineer may require the Contractor to provide a sample of the lightweight aggregate, at no cost to the Department, to verify the specific gravity, absorbed moisture content, and desorption of the material.

Verification of the mix design will include trial batch test results and other criteria as determined by the Engineer. The Contractor will be notified in writing of verification. Verification of a mix design shall in no manner be construed as acceptance of any mixture produced. Tests performed at the jobsite will determine if a mix design can meet specifications.

Quality Control Sampling and Testing of Lightweight Aggregate by the Contractor. The Contractor shall sample and test the lightweight aggregate as follows.

- Gradation. The gradation shall be tested a minimum once per day prior to pouring, unless the stockpile has not received additional aggregate material since the previous test. The gradation shall be determined according to XXX.

- Moisture. The field absorbed moisture content and surface moisture of the lightweight aggregate stockpile shall be determined daily at the start of production for that day, and then as needed to control production throughout the day, according to XXX.

Quality Assurance Sampling and Testing of Lightweight Aggregate by the Engineer. The Engineer reserves the right to perform quality assurance tests on independent and split samples of the lightweight aggregate. An independent sample is a field sample obtained and tested by only one party. A split sample is one of two equal portions of a field sample, where two parties each receive one portion for testing. The Engineer may request the Contractor to obtain a split sample. The results of all quality assurance tests by the Engineer will be made available to the Contractor. However, Contractor split sample test results shall be provided to the Engineer before Department test results are revealed. The Engineer's quality assurance independent sample and split sample testing for placement or acceptance will be as follows:

- Gradation. One independent or split sample test at the beginning of the project. Thereafter, independent testing frequency will be as determined by the Engineer, and split testing frequency will be a minimum of 10 percent of the total tests required of the Contractor.

- Moisture. One independent or split sample test at the beginning of the project, and as determined by the Engineer thereafter.

Comparing Lightweight Aggregate Test Results. Differences between the Engineer's and the Contractor's split sample test results will be considered reasonable if within the following limits:

Test Parameter	**Acceptable Limits of Precision**
Gradation	See "Guideline for Sample Comparison" in Appendix A of the Manual of Test Procedures for Materials.
Moisture	0.5%

Action shall be taken when either the Engineer's or the Contractor's test results are not within specification limits. Action may include, but is not limited to, immediate retests on a split sample; investigation of the sampling method, test procedure, equipment condition, equipment calibration, and other factors; or the Contractor being required to replace or repair test equipment as determined by the Engineer.