Evaluation and Testing of a Lightweight Fine Aggregate Concrete Bridge Deck in Buchanan County, Iowa

Final Report
May 2016

BRIDGE ENGINEERING CENTER

National Concrete Pavement Technology Center
Tech Center





IOWA STATE UNIVERSITY
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16. Abstract

Internal curing is a relatively new technique being used to promote hydration of portland cement concretes. The fundamental concept is to provide reservoirs of water within the matrix such that the water does not increase the initial water/cementitious materials ratio to the mixture, but is available to help continue hydration once the system starts to dry out. The reservoirs used in the US are typically in the form of lightweight fine aggregate (LWFA) that is saturated prior to batching.

Considerable work has been conducted both in the laboratory and in the field to confirm that this approach is fundamentally sound and yet practical for construction purposes. A number of bridge decks have been successfully constructed around the US, including one in Iowa in 2013.

It is reported that inclusion of about 20% to 30% LWFA will not only improve strength development and potential durability, but, more importantly, will significantly reduce shrinking, thus reducing cracking risk. The aim of this work was to investigate the feasibility of such an approach in a bridge deck.

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EVALUATION AND TESTING OF A LIGHTWEIGHT FINE AGGREGATE CONCRETE BRIDGE DECK IN BUCHANAN COUNTY, IOWA

Final Report
May 2016

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In addition, the authors would like to acknowledge the support of the Iowa DOT Office of Bridges and Structures staff, who continually provide great insight, guidance, and motivation for practical and implementable research like this, and to thank the Buchanan County engineer for his support and assistance on this project.

EXECUTIVE SUMMARY

Recent research has indicated several benefits to using lightweight fine aggregate (LWFA) in concrete mixtures for concrete bridge decks. The LWFA particles act as reservoirs that provide curing water to the hydrating mixture from within the system. This is particularly beneficial in low water-to-cement (w/cm) ratio concrete mix designs, in which it is unlikely that the water will be sufficient to hydrate all of the cement at the time of mixing. The LWFA improves the properties of the concrete and reduces the risk of cracking.

To ensure that these concrete material alternatives offer the greatest benefit to bridge owners, both in Iowa and nationwide, this research consisted of field testing and evaluating a demonstration bridge designed to utilize a LWFA concrete mixture in the concrete deck of a composite steel girder bridge system. The research was performed through a cooperative effort between the Iowa State University Institute for Transportation's National Concrete Pavement Technology (CP Tech) Center and Bridge Engineering Center (BEC), the Iowa Department of Transportation, and Buchanan County Engineer/Secondary Roads Department.

The objectives of this work were to perform laboratory and field testing and evaluation of a concrete bridge deck constructed with LWFA concrete. The National CP Tech Center conducted material tests on the LWFA and concrete mixtures used in the bridge deck, both in the laboratory and during construction. The BEC conducted live load field tests to evaluate the performance and condition of the LWFA deck and the control deck both at the time of placement and about a year after construction.

Evaluation of performance was accomplished through comparisons with design assumptions and previous research and comparisons between the performance of the LWFA deck and the control deck. A lifecycle cost and service life prediction was also conducted.

INTRODUCTION

Research reported by others has indicated that there are several benefits associated with the use of lightweight fine aggregate (LWFA) in concrete mixtures for the purposes of internal curing (IC). The LWFA particles act as reservoirs that provide water to promote hydration without influencing water-to-cement (w/cm) ratio. The particles are uniformly distributed in the mixture, thus providing a benefit to the whole volume of concrete and not just the surface, as occurs with conventional curing. This benefit is most marked in low w/cm mixtures, in which there is typically insufficient water in the batch to hydrate all of the cement. This improved hydration can improve durability while reducing the effects of dimensional change due to shrinkage. These benefits are most useful in bridge decks that are exposed to aggressive environments and are at a high risk of cracking.

A test bridge was constructed in Iowa using internal curing in half of the deck in order to evaluate the benefits of LWFA in a typical structure.

The objectives of this work were as follows:

- Evaluate the performance of the material through laboratory and field testing
- Evaluate the structural performance through live load tests of the finished structure at the time of construction and after one year and two years of service
- Conduct a lifecycle cost and service life prediction

BACKGROUND

In 1991, Philleo (1991) discussed the use of saturated lightweight fine aggregate in concrete mixtures to replenish water that is depleted during cement hydration. This process is now known as internal curing. It has been demonstrated that internal curing has the potential to contribute to a more sustainable infrastructure in a variety of ways, as discussed below.

Research has shown that concrete mixtures with internal curing result in members with improved material properties, such as reduced autogenous shrinkage, reduced stiffness, increased strength, and lower permeability (Bentur et al. 2001, Delatte and Cleary 2008, Bentz 2009, Schlitter et al. 2010, Cusson and Margeson 2010). In addition, internal curing mixtures are beneficial for enhancing the performance of supplementary cementitious materials (SCMs), such as fly ash and slag cement, by promoting more efficient hydration of the cementitious system.

As stated by Henkensiefken et al. (2009):

"The use of internal curing can substantially reduce transport properties, such as diffusion and sorptivity, thereby increasing the service life of concrete structures. Finally, the enhanced hydration and increased strengths provided by internal curing may allow for small but significant reductions in cement content in many concrete mixtures, thereby significantly reducing the carbon footprint of each cubic yard of concrete used throughout the world."

The Buchanan County Engineer expressed an interest in evaluating the performance of an LWFA concrete bridge deck and offered to use a three-span bridge that was scheduled for rehabilitation in 2012. Buildex, Inc. donated LWFA material. Funding for research was provided by the Iowa Highway Research Board (IHRB) and the Expanded Shale, Clay, and Slate Institute (ESCSI).

This report describes the work that was conducted and the data collected from the work conducted before, during, and after construction of the demonstration deck. The bridge is a three-span, two-lane road bridge across a small river. Half of the deck (1.5 spans) was constructed using a mixture designed for internal curing.

WORK CONDUCTED

Laboratory Testing

A load of lightweight fine aggregate was delivered to the Buchanan County engineer by the manufacturer. Samples of all the concrete ingredients were taken to the PCC Laboratory at Iowa State University and to the contractor for trial batch testing.

Mix proportions were based on replacing 20% by mass of the fine aggregate with LWFA in the test mixture. Both the control and test mixtures were to comply with the normal specifications used by the county.

Six mixtures were prepared in the laboratory: three control mixtures, two mixtures with 20% by mass of the fine aggregate replaced with LWFA, and one mixture with 30% LWFA replacement. Table 1 shows the breakdown of the three sets of mixtures.

Table 1. Mixture proportions

Material (lb./yd^3)	Control	20% IC	30% IC
Portland cement	356	356	356
Slag cement	119	119	119
Fly Ash (Class C)	119	119	119
Coarse Aggregate	1504	1504	1504
Sand Fine Aggregate	1512	1072	968
LWFA	-	311	415
Water	255	222	208
Total Materials	3865	3736	3689

The first pair (Control 1, Test 1) were prepared using the proportions planned for use on site before construction in 2012. The same mixtures were also prepared and tested by the contractor. The mixtures were repeated (Control 2, Test 2) several months later to verify the findings. However, the laboratory temperature at the time of batching was significantly lower than for the first pair, which influenced the results. A third test set was prepared with 30% LWFA (Control 3, Test 3).

The gradations of aggregate used in both the control and IC mixes are shown in Figure 1.

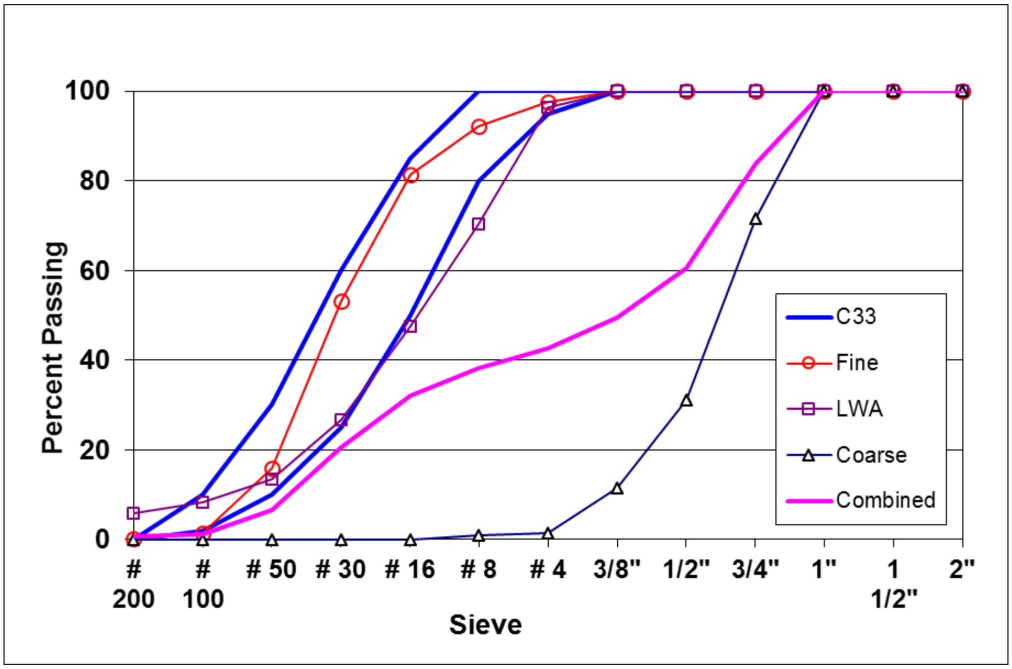


Figure 1. Aggregate gradations

A haystack chart of the combined aggregate gradations for the control and IC mixtures is shown in Figure 2.

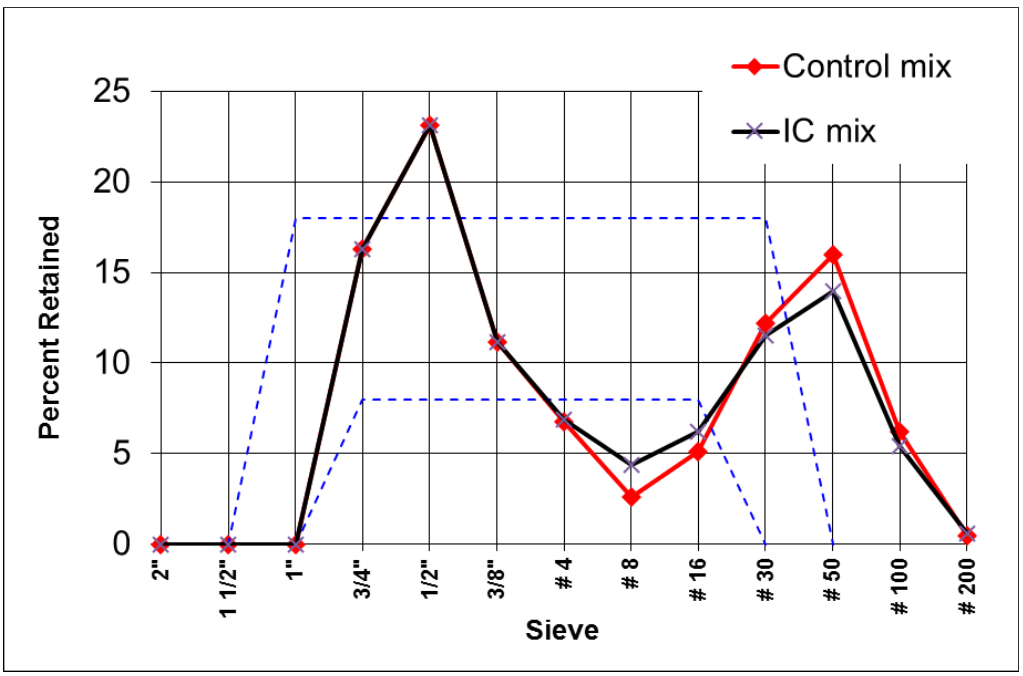


Figure 2. Haystack chart of combined aggregate gradations for control and IC mixes

The following laboratory tests were conducted:

- Slump, unit weight, temperature, and air content of fresh concrete (ASTM C143, ASTM C138, ASTM C231)
- Semi-adiabatic calorimetry (ASTM C1679)
- Compressive strength, splitting tensile strength, static modulus of elasticity: 4 in. x 8 in. cylinders at 3, 7, 28, and 56 days (ASTM C39, ASTM C496, ASTM C469)
- Surface resistivity (LADOTD 2011) method
- Rapid chloride permeability at 28 days (ASTM C1202)
- University of Cape Town Air Permeability Method at 28 days (Alexander et al. 1999)
- Restrained shrinkage test (ASTM C1581)

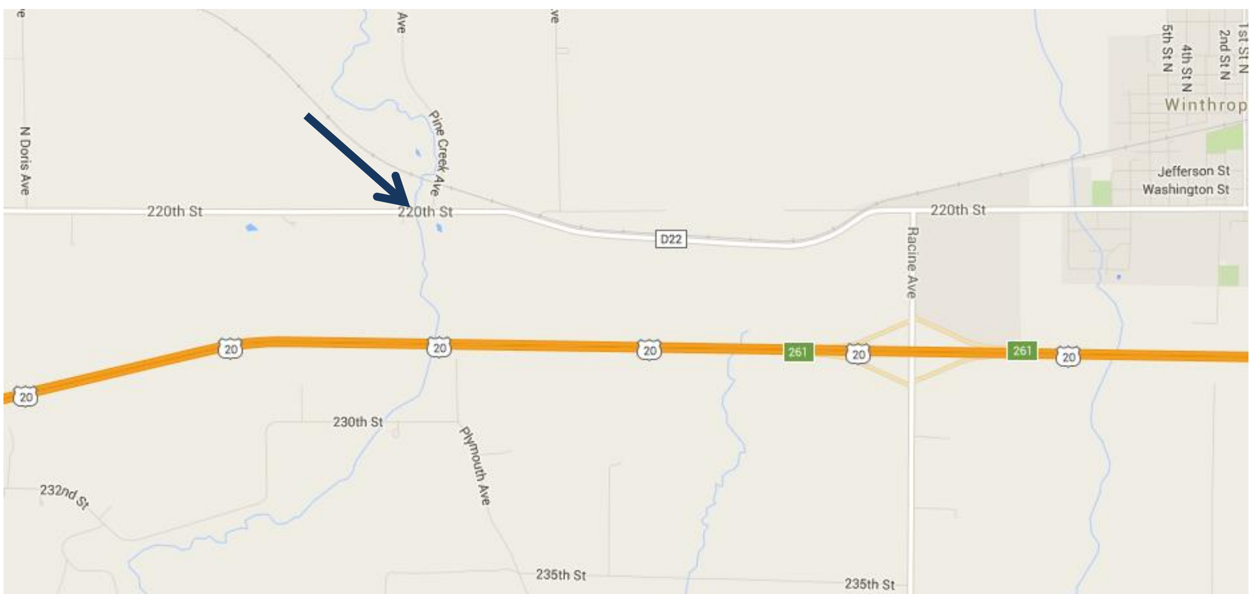
Field Testing

The details of the field testing portion of the project are as follows:

- Owner: Buchanan County, Iowa
- Contractor: Jim Schroeder Construction, Inc.
- Location: 220th Street at Pine Creek
- Structure: Three-span bridge 176 ft long and 30 ft wide, one half (east end, both lanes) conventional mixture and the other half (west end) using internal curing concrete (Figure 3)
- The deck was 8 in. thick with a nominal 2.5-in. cover to the top layer of reinforcement and a nominal 1-in. cover to the bottom layer of reinforcement.
- The specified air content was 6.5% with a maximum of 8.5% and a minimum of 5.5%. The slump ranged from 1 in. to 3 in. with a maximum of 4 in.

Figure 3. Three-span bridge at Pine Creek during concrete placement

The location of the project site is shown in Figure 4.



Map data ©2016 Google

Figure 4. Project location

Concrete placement and sampling took place on June 28, 2013. Hardened samples were transported to Iowa State University on June 28, 2013 for further testing. The same suite of tests was conducted on all of the materials.

Six wireless corrosion sensors were installed 0.5, 1.0, and 1.5 in. below the concrete surface in both the control and test sections. The sensors are passive devices that, when read by a scanner, indicate whether chlorides have penetrated to the depth at which they are buried (Ley 2013). Installation and interrogation of a sensor are shown in Figure 5.

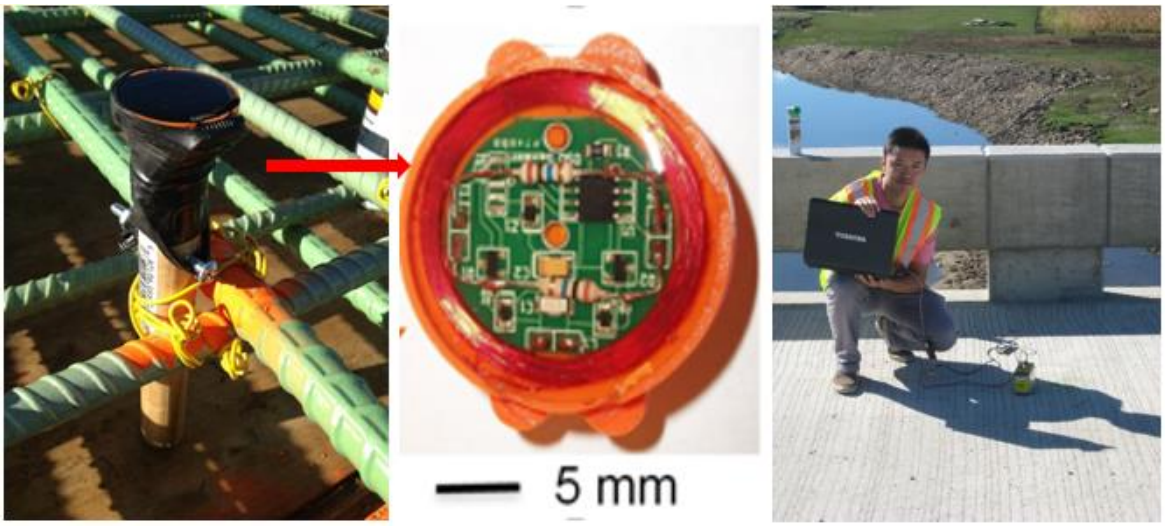
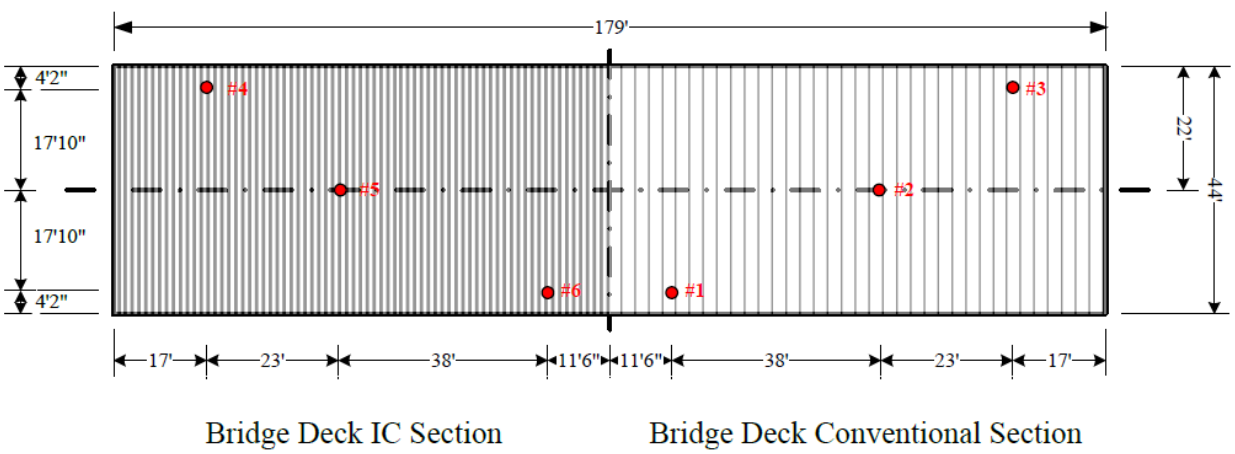


Figure 5. Installation of a corrosion measurement sensor (left) and reading the sensor data on the completed bridge deck (right)

Sensor locations are shown in Figure 6.



#1 and #6 sensors: 1/2 in. below concrete surface
#1 and #6 sensors: 1 in. below concrete surface
#1 and #6 sensors: 1 1/2 in. below concrete surface

Figure 6. Corrosion sensor locations

Field Observations during Construction

The following observations were made during the field work:

- Concrete placement was by means of a form riding bridge deck paver.
- All concrete came from a fixed batch plant and was delivered to the job site in ready-mix trucks. Concrete was discharged from two ready-mix trucks simultaneously to a pump truck.
- The LWFAs were soaked for 48 hours and drained overnight before mixing. The LWFA pile at the central mix plant is shown in Figure 7.
- The mean temperature at the construction site was 71°F. The average humidity and wind speed were 57% and 12 mph (NW), respectively.
- Fresh concrete tests included slump, unit weight, and concrete temperature. Tests were conducted by Buchanan County Engineer/Secondary Roads staff, and the results are shown in Table 2.

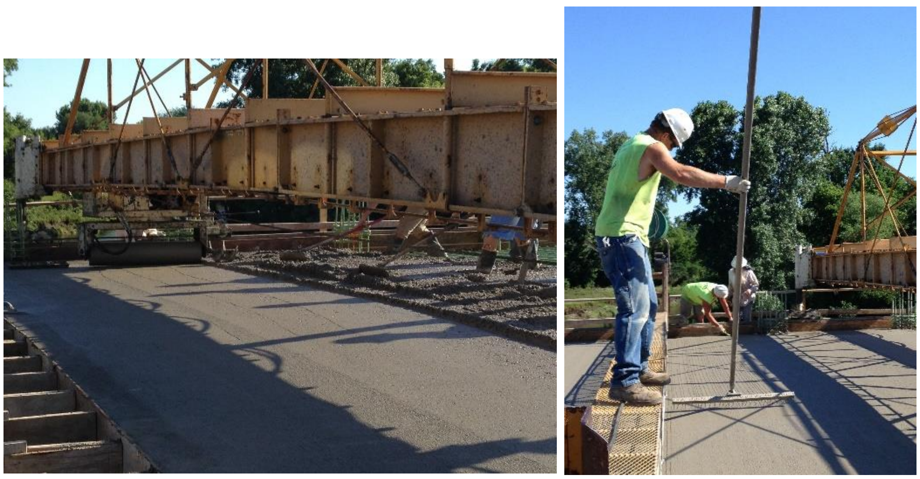
Figure 7. LWFA pile before mixing at central mix plant

Table 2. Fresh properties of control and IC concrete mixes on site

Sample Information and Identification							Fresh Concrete Workability Properties			Pressure Air
Sample Date	Time of Mixing	Truck Loading Number	Time of Discharge	Sample Comments	Water Add @ Project Site (gallon)	w/c after Adjustment	Slump (in.)	Unit Weight (lb/ft^3)	Temperature (°F)	% Air Content
06/28/13	6:20 AM	1	7:02 AM	Internal curing section – 5 yards truck	20	0.402	1.25	-	76.00	6.2
06/28/13	6:40 AM	2	7:02 AM	Internal curing section – 10 yards truck	20	0.346	1.75	-	76.00	6.2
06/28/13	7:01 AM	3	7:23 AM	Internal curing section – 10 yards truck	25	0.371	3.00	-	76.00	6.2
06/28/13	7:18 AM	5	7:45 AM	Internal curing section – 10 yards truck	23	0.387	3.25	138.5	76.00	6.8
06/28/13	8:04 AM	10	8:30 AM	Internal curing section – 10 yards truck	8	0.388	3.25	-	77.00	6.8
06/28/13	8:12 AM	11	8:40 AM	Internal curing section – 10 yards truck	10	0.386	3.25	-	77.00	6.6
06/28/13	8:19 AM	12	8:47 AM	Internal curing section – 10 yards truck	10	0.386	3.25	-	77.00	6.7
06/28/13	8:29 AM	13	9:00 AM	Internal curing section – 10 yards truck	15	0.386	3.25	-	78.00	6.8
06/28/13	8:38 AM	14	9:07 AM	Internal curing section – 10 yards truck	15	0.382	3.25	-	78.00	6.4
06/28/13	9:18 AM	2	9:35 AM	Control mix section – 5 yards truck	10	0.427	3.00	-	77.00	7.5
06/28/13	9:24 AM	3	9:42 AM	Control mix section – 10 yards truck	5	0.416	-	-	77.00	-
06/28/13	10:02 AM	5	10:33 AM	Control mix section – 10 yards truck	12	0.430	-	-	78.00	7.4
06/28/13	10:20 AM	6	10:40 AM	Control mix section – 10 yards truck	10	-	3.25	-	78.00	-
06/28/13	10:58 AM	11	11:27 AM	Control mix section – 10 yards truck	10	0.430	-	141.2	78.00	8.0
06/28/13	11:15 AM	13	11:51 AM	Control mix section – 10 yards truck	3	-	-	142.8	79.00	7.4
06/28/13	11:24 AM	14	11:54 AM	Control mix section – 10 yards truck	10	-	3.00	-	79.00	7.5
06/28/13	11:29 AM	15	12:10 PM	Control mix section – 10 yards truck	8	0.428	3.25	-	78.00	-
06/28/13	11:37 AM	16	12:17 PM	Control mix section – 10 yards truck	15	0.428	3.25	-	79.00	-

Figures 8 through 11 illustrate some activities during the placing process.

Figure 8. Concrete being discharged to a pump truck and then to the deck



Figures 9. Concrete being finished

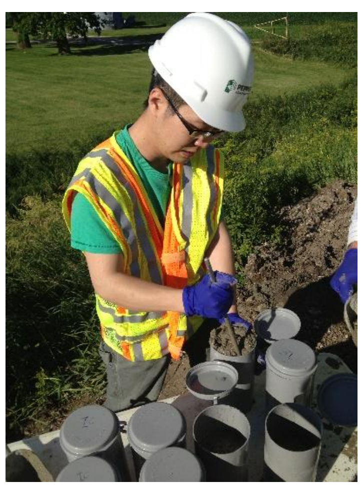


Figure 10. Concrete sample preparation

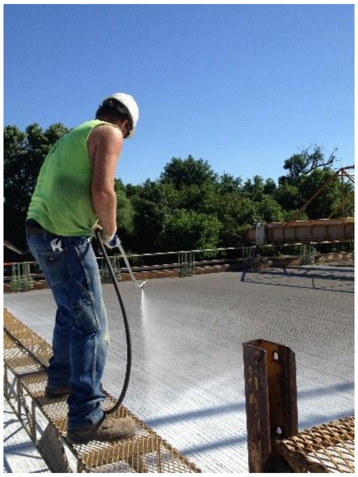


Figure 11. Curing compound being applied

Field Testing after Construction

Following construction of the LWFA bridge, the Bridge Engineering Center (BEC) conducted live load tests of the bridge on three separate occasions: (1) immediately following construction just before the bridge was open to traffic (August 2013), (2) approximately one year after the initial load test (August 2014), and (3) approximately two years after the initial load test (November 2015). All three load tests involved instrumenting the bridge girders and deck with strain transducers and conducting static load tests using a fully loaded tandem-axle dump truck of known weight (see Figure 12).

Figure 12. Live load testing of LWFA bridge, 2013

Bridge Diagnostics, Inc. (BDI) strain transducers and structural testing system were utilized for the testing, and the load truck was provided by the Buchanan County Secondary Roads Department.

Focus for the testing of the three-span bridge was on the end spans, with the west end span having the IC deck and the east end span having the conventional concrete deck. There were three main structural characteristics of interest during the load testing: the strain magnitudes in the deck, transverse load distribution, and composite action. To best evaluate these characteristics, strain gauges were installed midway between the bridge girders on the underside of the deck at midspan of both end spans and on the top and bottom flanges of each girder at midspan of both spans. Figure 13 illustrates a typical cross-section at midspan and the location of the strain gauges. (This is representative of the instrumentation plan for all three load tests.)

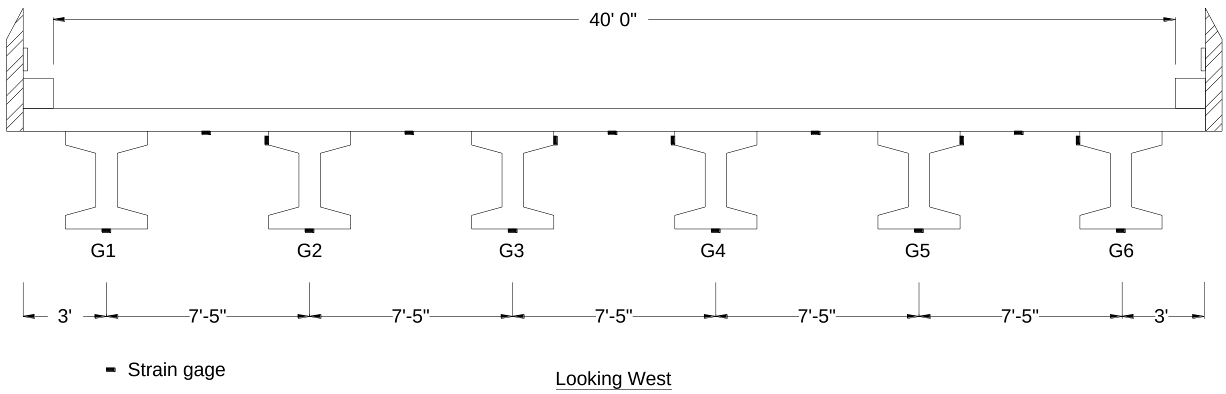


Figure 13. Typical instrumentation layout at midspan for both LWFA bridge end spans

Figure 14 illustrates a typical strain gauge setup during testing, showing the placement of the top and bottom flange girder gauges and the deck gauge.

Figure 14. Typical instrumentation setup for testing of LWFA bridge

Equation 1 is used to calculate the load fractions for a bridge and evaluate the transverse load distribution characteristics for a particular load case (LC).

$$LF = \frac{\dfrac{(E_i I_i \varepsilon_i)}{y_i}}{\sum_{i=1}^{n} \dfrac{(E_i I_i \varepsilon_i)}{y_i}}, \text{ decimal percentage of a single truck} \qquad \textbf{(1)}$$

where,
E_i = modulus of elasticity of the ith girder
Ii = composite moment of inertia of the ith girder
ε_i = maximum measured bottom flange strain from the ith girder
y_i = distance from the neutral axis to the bottom flange gauge location

For the LWFA bridge, the girders were assumed to be of equal stiffness and all of the same material; therefore, Equation 1 was modified to that shown in Equation 2 for calculation of the load fractions for the LWFA bridge.

$$LF = \frac{(\varepsilon_i)}{\sum_{i=1}^{n} (\varepsilon_i)}, \text{ decimal percentage of a single truck} \qquad \textbf{(2)}$$

Equation 3 was used to obtain distribution factors for the LWFA bridge by using superposition of two individual load cases.

$$DF = LF_A + LF_B \quad \textbf{(3)}$$

where,
LFA = load fraction from first load case of the pair
LFB = load fraction from second load case of the pair

For the two-lane loaded case, the individual load cases were offset 4 ft, the minimum transverse distance between two trucks per the AASHTO specification. Lastly, measured strains from the top and bottom flanges at midspan were used to calculate the neutral axis and evaluate the composite action for each girder.

For all three live load tests, six load cases were evaluated to obtain a comprehensive understanding of the bridge's behavior, evaluate symmetry, and best quantify the three previously outlined metrics of interest. Figure 15 shows a cross-section of the bridge showing the location of each of the six load cases.

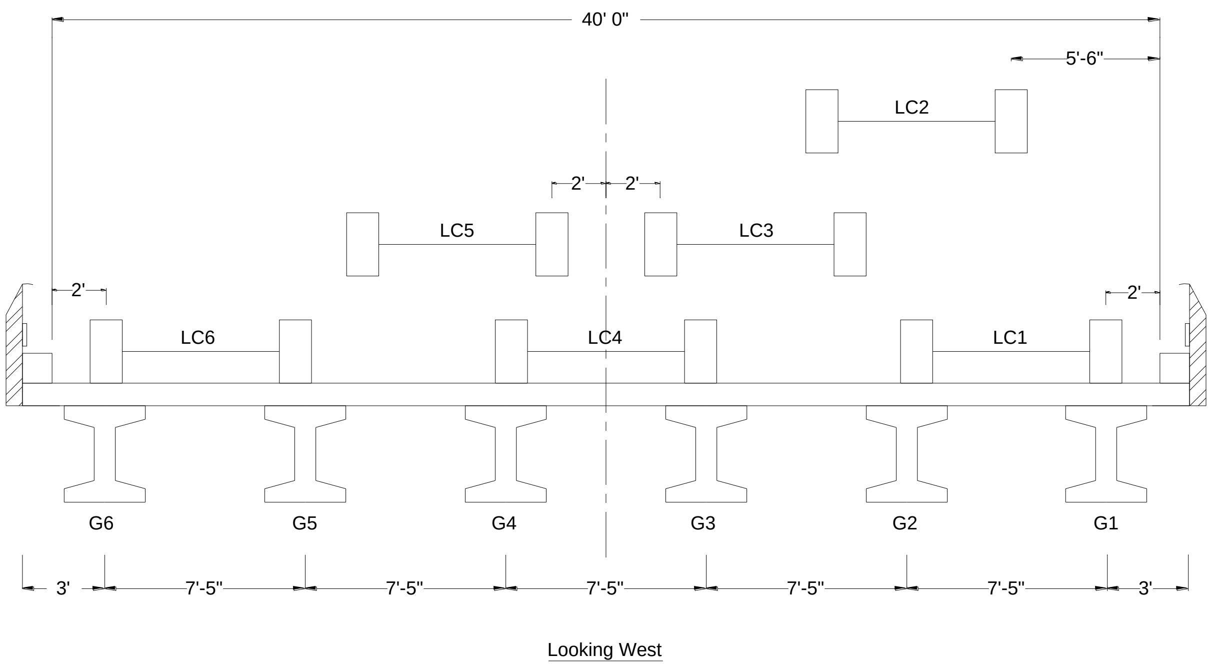


Figure 15. Load cases for testing of the LWFA bridge

Load Cases 1 and 6 placed the load truck as close to the guardrail as allowed by the design specification, 2 ft. Load Cases 3 and 5 each had their interior axle line located 2 ft away from the longitudinal centerline of the bridge; this placed the load trucks 4 ft apart from each other when the load cases were superimposed, resulting in a case in which two lanes were loaded. Load Case 4 was simply the load truck centered on the centerline of the bridge, which allowed for an evaluation of symmetry in transverse load distribution and general bridge behavior. Lastly, Load Case 2 was situated such that the axle closest to the guardrail was centered between Girder Lines 1 and 2, thereby maximizing the strain in the deck between those two girders.

As mentioned previously, the load truck for all tests was a fully loaded tandem-axle dump truck. The weight of each truck (gross and per axle) is listed in Table 3.

Table 3. Load truck weights

Year	Gross Weight (lbs)	Front Axle (lbs)	Front Tandem (lbs)	Rear Tandem (lbs)
2013	51,340	16,730	17,305	17,305
2014	50,760	16,300	17,800	16,660
2015	51,420	15,740	17,840	17,840

For all load cases, the load truck was driven at a crawl speed (~5 mph), traveling from east to west.

RESULTS AND DISCUSSION

Table 4 summarizes the fresh properties, mechanical properties, and permeability of both the field and laboratory samples. A set of results from a commercial laboratory employed by the contractor are included.

Table 4. Laboratory and field test results

Test	Age (days)	Field Samples Control	Field Samples 20% IC	Laboratory Samples Control	Laboratory Samples 20% IC	Laboratory Samples 30% IC	Contractor Samples Control	Contractor Samples 20% IC
Slump, in.		3.3	3.3	4.3	4.0	4.0	4.3	5.0
Air Content, %		7.5	6.5	6.5	7.0	6.8	6.5	6.8
Unit Weight, pcf		141.0	138.5	143.1	137.4	131.5	143.1	138.2
Initial Set, mins				214	220		276	272
Compressive Strength, psi	3	2848	2909	2410	3525	2562	3810	3530
	7	4027	3969	3865	4618	3796	4760	4820
	28	6653	6672	5968	6746	5639	5910	7070
	56	7707	7838	6986	7781	6317		
Splitting Tensile Strength, psi	3	339	336	298	329	319		
	7	454	435	412	458	434		
	28	556	486	522	517	509		
	56	637	582	602	570	532		
Modulus of Elasticity, ksi	3	3520	2990	3350	3500	3150		
	7	3590	3150	3420	3750	3300		
	28	4300	3950	4120	4300	3900		
	56	4600	4350	4550	4860	4600		
Rapid Chloride Permeability Testing, coulomb	28	2407	2166	1986	1459	1058	1239	1090
Air Permeability Index	28	10.13	10.18	10.35	10.63	10.98		
Average Stress Rate, psi/day	28			12.01	14.85	14.04		

Surface resistivity data are shown in Table 5.

Table 5. Surface resistivity test results for laboratory and field samples

Surface Resistivity, kΩcm							
	Laboratory					Field	
	Field samples		Laboratory samples				
Age (days)	Control	20% IC	Control	20% IC	30% IC	Control	20% IC
3	7	6.6	6.4	7.3	8.7		
7	9.8	9.2	9.2	10.6	12.6		
28	22.6	22.8	20.9	38.3	51.1		
56	33.3	32.2	31.2	57	74.4		
86	41.1	45.6	42.6			50.3	52.7
365						70.7	73.9

Calorimetry results for both laboratory and field samples are shown in Figure 16.

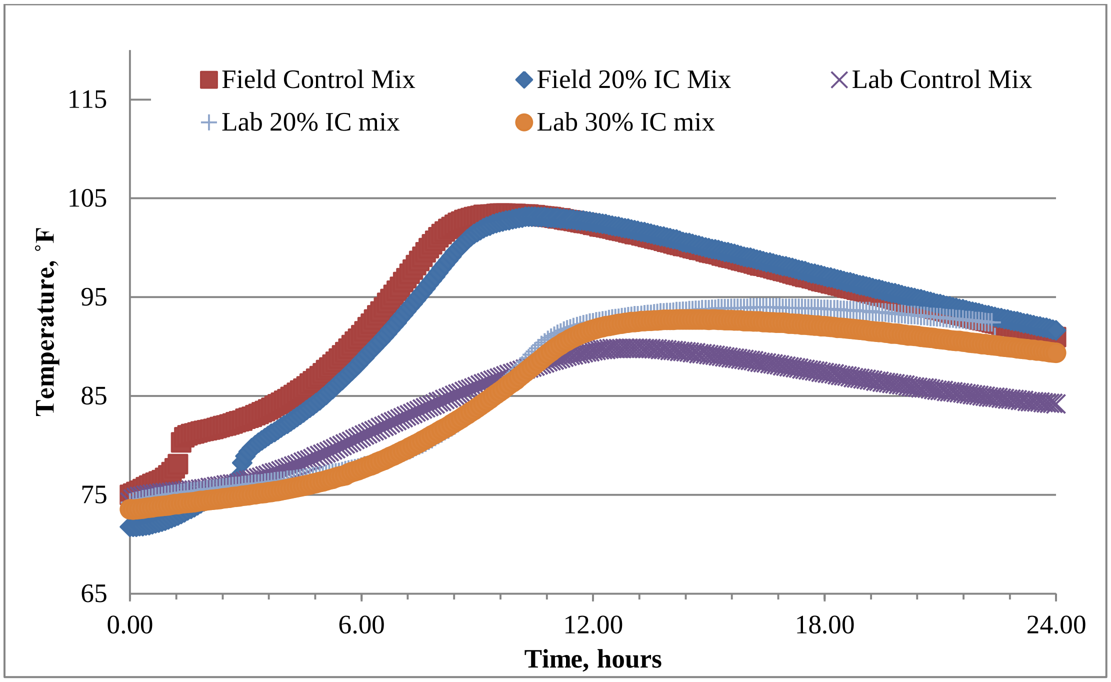
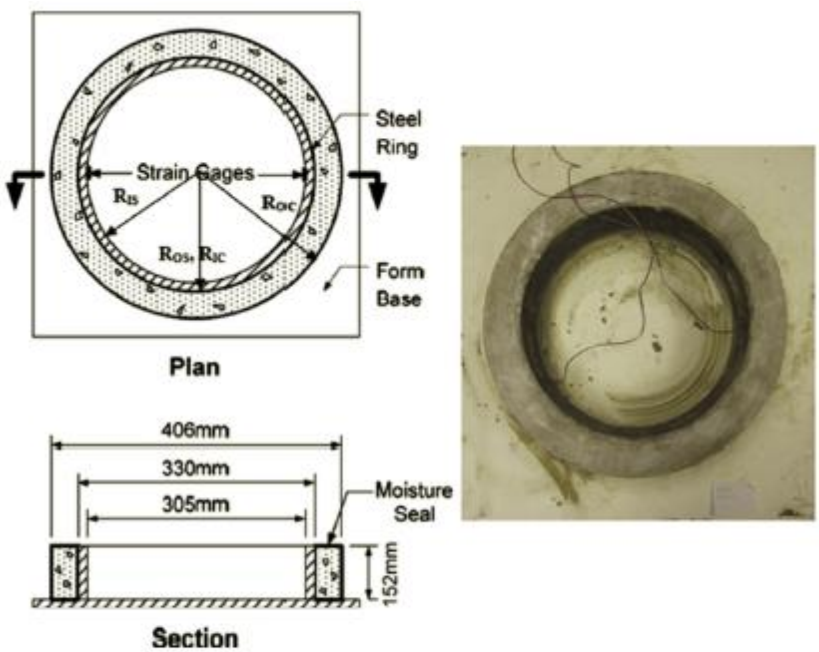


Figure 16. Calorimetry results for laboratory and field samples

A restrained shrinkage test was conducted in accordance with ASTM C1581. The configuration is shown in Figure 17.



Wang et al. 2014

Figure 17. Configuration of restrained concrete ring samples

Figure 18 shows the development of the shrinkage strain of three laboratory samples with elapsed time.

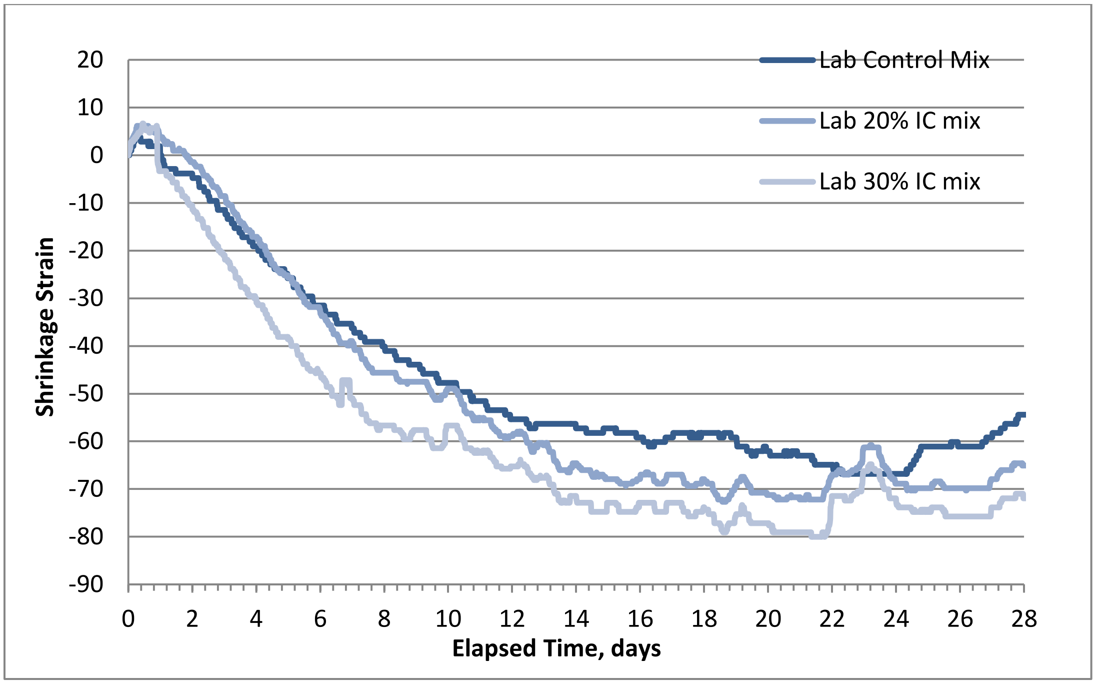


Figure 18. Restrained shrinkage test results

The slope of the regression line of shrinkage strain versus the square root of elapsed time is the shrinkage strain rate factor. This factor can be used to calculate the average stress rate, q, of each mix based on Equation 4, and test results are summarized earlier in Table 4.

$$q = \frac{G|\alpha_{avg}|}{2\sqrt{t_r}} \quad \textbf{(4)}$$

where,
G = a constant number of 72.2 GPa
$|\alpha_{avg}|$ = the absolute value of the average strain rate factor for each test specimen, (in./in.)/day1/2
t_r = the elapsed time at cracking or elapsed time when the test is terminated for each test specimen, days

The following observations can be made from the data:

- The unit weight and stiffness of the IC mixture were slightly lower than those of the control, as expected.
- Both the 20% and 30% IC mixes did not dramatically affect the mechanical properties, but the LWFA dosage improved the permeability.

- The effect of the IC mixes on the elapsed time to reach peak temperature was small.
- Based on the average stress rate values, all three mixtures were classified as having a “low” potential for cracking.

SITE INSPECTIONS

On September 12, 2013, the research team revisited the construction site in order to evaluate the IC and control sections of bridge deck after concrete had been in place for about three months.

The following observations were made:

- No cracks were found.
- The curing compound still appeared to be in place.
- Some traffic was using the bridge, although barriers were in place.
- Little observable difference was apparent between the IC and control sections.

Surface resistivity was measured on the bridge deck. Ten readings were taken on surfaces that had been soaked for 30 minutes. The average resistivity for the IC and control sections was 52.7 and 50.3 kΩcm, respectively.

Four out of six corrosion sensors were located using the scanner and marked with green paint and indents drilled into the surface for future reference. Sensors #3 and #4 (Figure 6), located 1.5 in. below the surface, could not be detected by the scanner. No evidence of corrosion was detected.

Figures 19 and 20 illustrate some activities and observations at the site:

Figure 19. Control (left) and IC (right) section surfaces

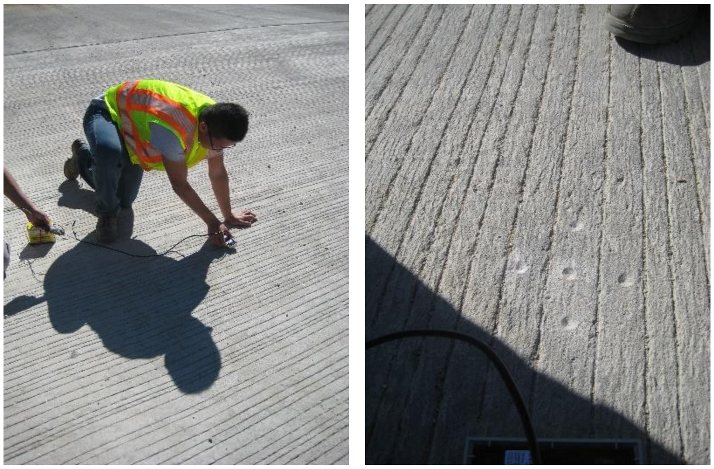


Figure 20. Detecting the corrosion sensors (left) and drilled indents in the surface for locating sensors in the future (right)

On August 8, 2014, the research team revisited the site to evaluate the IC and control sections after the concrete had been in place for about one year.

The following observations were made:

- No cracks were found.
- Slight scaling was observed on both control and IC sections near drains (Figure 21).
- Little observable difference was found between the IC and control sections.

Figure 21. Limited scaling observed on the surface near drains

Surface resistivity readings were repeated on the bridge deck. The resistivity readings for the IC and control sections were 73.9 and 70.7 kΩcm, respectively.

LIEFCYCLE ANALYSIS

A lifecycle cost and service life prediction was performed using the Life-365 Service Life Prediction Model software (Ehlen 2014). Four modules were defined in the program: project, exposure, concrete mixtures, and individual costs. The critical input parameters within each module are listed in the following four sections.

Project

- Type of structure: slabs and walls (1-D) for both the control and IC sections
- Thickness: 8.0 in.
- Reinforcement depth: 2.5 in.
- Area: 2640 ft^2
- Base year: 2013
- Analysis period: 150 years
- Inflation rate: 1.80%
- Real discount rate: 2.00%

Exposure

- Chloride exposure: the default parameters for both the IC and control sections were selected. The structure is a rural highway bridge located in the Dubuque area, Iowa. The default maximum concentration at the bridge surface is 0.560% by weight, and the time to reach the maximum surface concentration is 13.3 years.
- Temperature cycle: based upon the temperature history in the Dubuque area

Concrete Mixtures

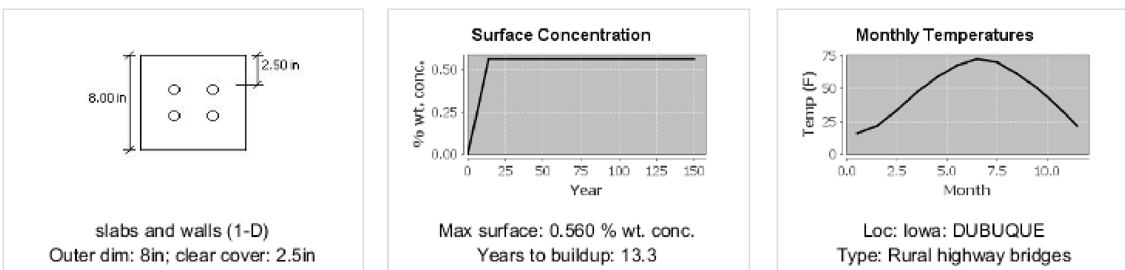
- Mixtures: both mixtures have the same w/cm ratio of 0.43, a fly ash content of 20%, a slag cement content of 20%, and an epoxy coated rebar steel type. No barriers or inhibitors were used.
- Diffusion rate: based on ASTM C1556, extrapolated from ASTM C1202. The IC section has a 10% lower rapid chloride permeability testing result than the control section, as shown in Table 3. Therefore, the diffusion rate at 28 days for the IC section was assumed to be 10% lower than that of the control section, i.e., 1.3079E-8 $in.^2$/sec and 1.4532E-8 $in.^2$/sec, respectively.
- "m" term: models the ability of chlorides to transfer through the concrete over time. This term is a function of the amount of fly ash and slag cement in the mixture. The higher the "m" value, the lower the diffusivity of the concrete over time. The calculated "m" term for the control section was 0.47, while that of the IC section was 0.52, which was extrapolated to be 10% higher than that of the control section.
- Hydration: similar to "m" term, the hydration periods for the IC and control sections are 27.5 and 25 years, respectively.

- Ct (% wt. conc.): the concentration of chloride on the surface of the reinforcing steel necessary to initiate corrosion was assumed to be the same for both the IC and control sections, i.e., 0.05.
- Propagation period: the estimated time between initial corrosion of the steel to the time at which the concrete is considered to be in need of repair. The propagation period for both sections was assumed to be 20 years.

Individual Costs

- Concrete cost: project bid price for the structural concrete of the control section was \$386.00/yd^3, while that of the IC section was calculated to be \$440.00/yd^3, in consideration of LWFA cost.
- Concrete repair cost: default numbers were selected, including \$37.16/ft^2 for repair, 10.00% area to be repaired, and a 10 year repair interval.

Figure 22 shows the service lives that can be predicted through the model for the two concrete mixes, and the lifecycle cost is shown in Figure 23.



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Control	yes	n/a	n/a	n/a		Epoxy Coated
IC	yes	n/a	n/a	n/a		Epoxy Coated

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Control	-> 1.45E-8 in*in/sec	-> 0.47	-> 0.05 % wt. conc.	37.9 yrs	-> 20 yrs	**57.9 yrs**
IC	-> 1.31E-8 in*in/sec	-> 0.52	-> 0.05 % wt. conc.	56.6 yrs	-> 20 yrs	**76.6 yrs**

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

Figure 22. Concrete mixes and service lives

Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Control	$9.53 per sq. ft	$0.00 per sq. ft	$30.47 per sq. ft	$40.00 per sq. ft
IC	$10.86 per sq. ft	$0.00 per sq. ft	$23.93 per sq. ft	$34.80 per sq. ft

Graphs

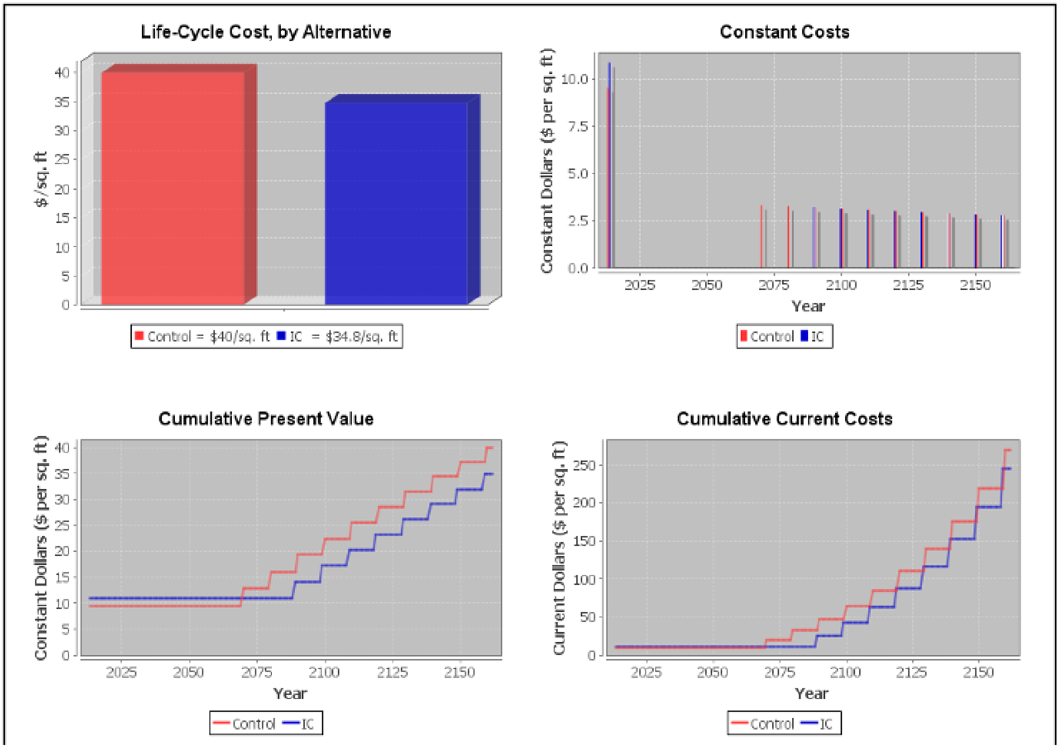


Figure 23. Lifecycle cost

The service life for the IC section is predicted to be 18.7 years longer than that of the control section. The initial cumulative present value is higher for the IC section, but it will be lower after about the year 2060.

LIVE LOAD TESTING RESULTS

As mentioned previously, three live load tests were conducted on the LWFA bridge following its construction in 2013. The first load test was completed in August of 2013 just before the bridge was opened to traffic, the second test was completed in August of 2014, and the third test was completed in November of 2015. There were three main structural characteristics of interest during the load testing, including transverse load distribution, strain magnitudes in the deck, and composite action, all of which were evaluated based on the girder and deck strains measured during the live load testing.

Transverse Load Distribution

Transverse load distribution represents the fraction of the load truck, presented as a fraction of a lane, distributed laterally to each girder for any given load case. Figure 24 shows the six load cases evaluated during each of the three load tests for quick reference.

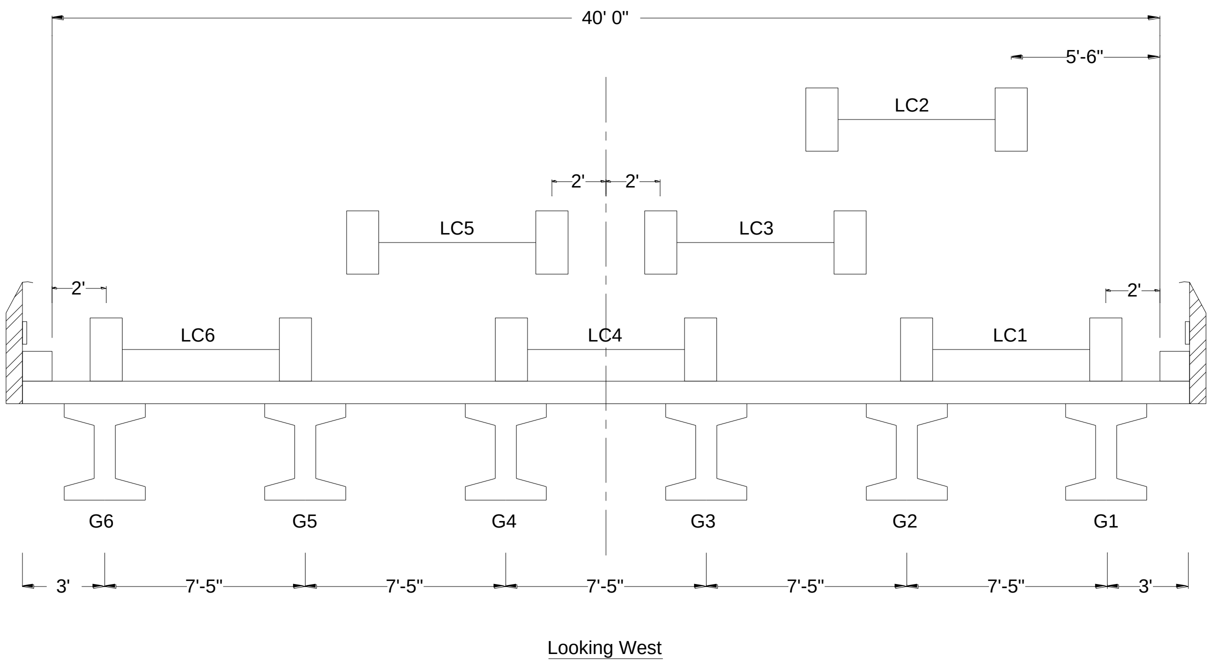


Figure 24. Load paths for testing of the LWFA bridge

Each load case was evaluated individually as a single-lane loaded case; additionally, Load Cases 3 and 5 were superimposed to create a two-lane loaded case, as described previously.

First, to obtain a general understanding of the comparative transverse live load distribution characteristics of the two end spans (west = LWFA deck, east = typical concrete deck) of the LWFA bridge, single-lane transverse distribution factor plots were generated. For brevity, transverse load distribution plots for only three load cases are presented, namely Load Cases 1, 4, and 6 in Figures 25, 26, and 27, respectively, because there was little evidence of variation throughout the testing.

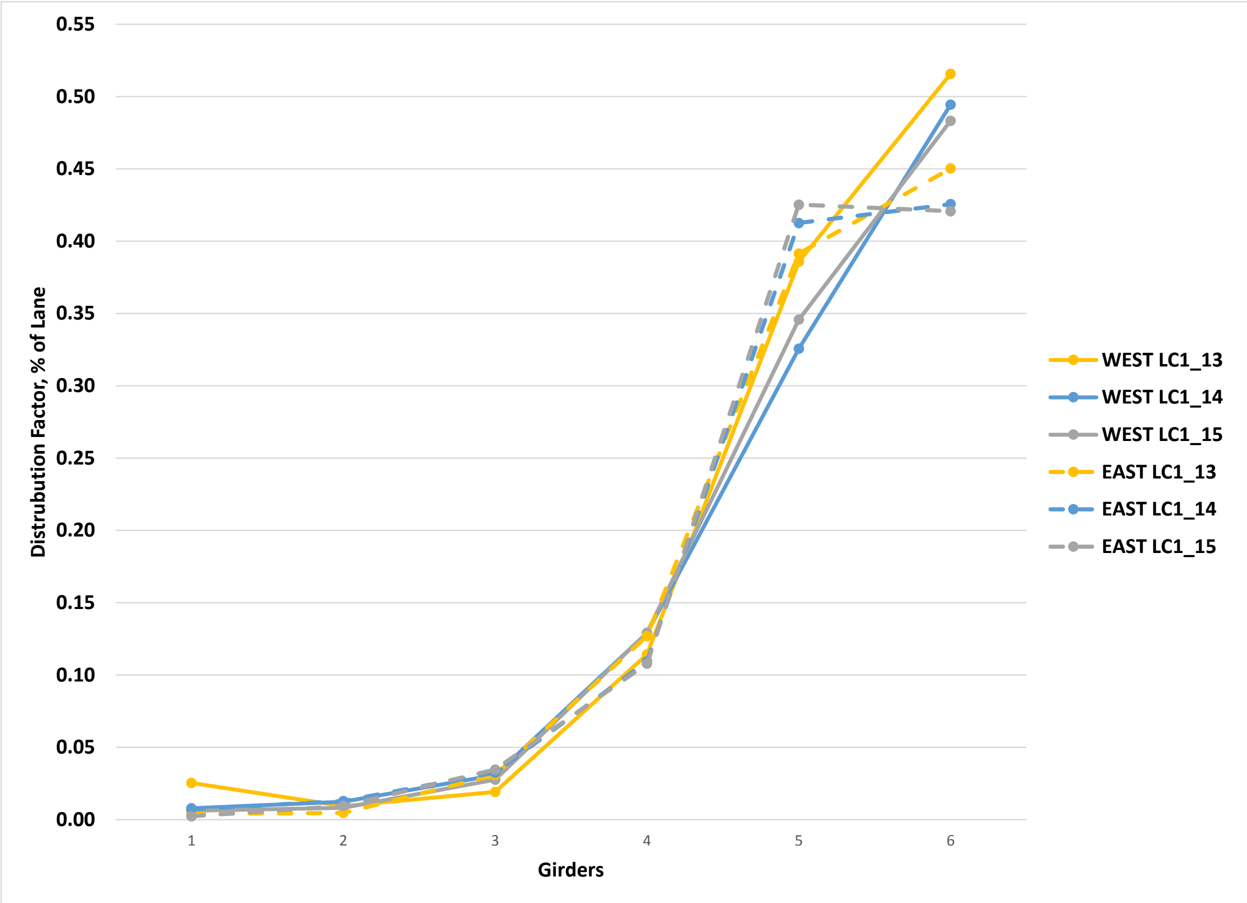


Figure 25. Transverse load distribution, Load Case 1

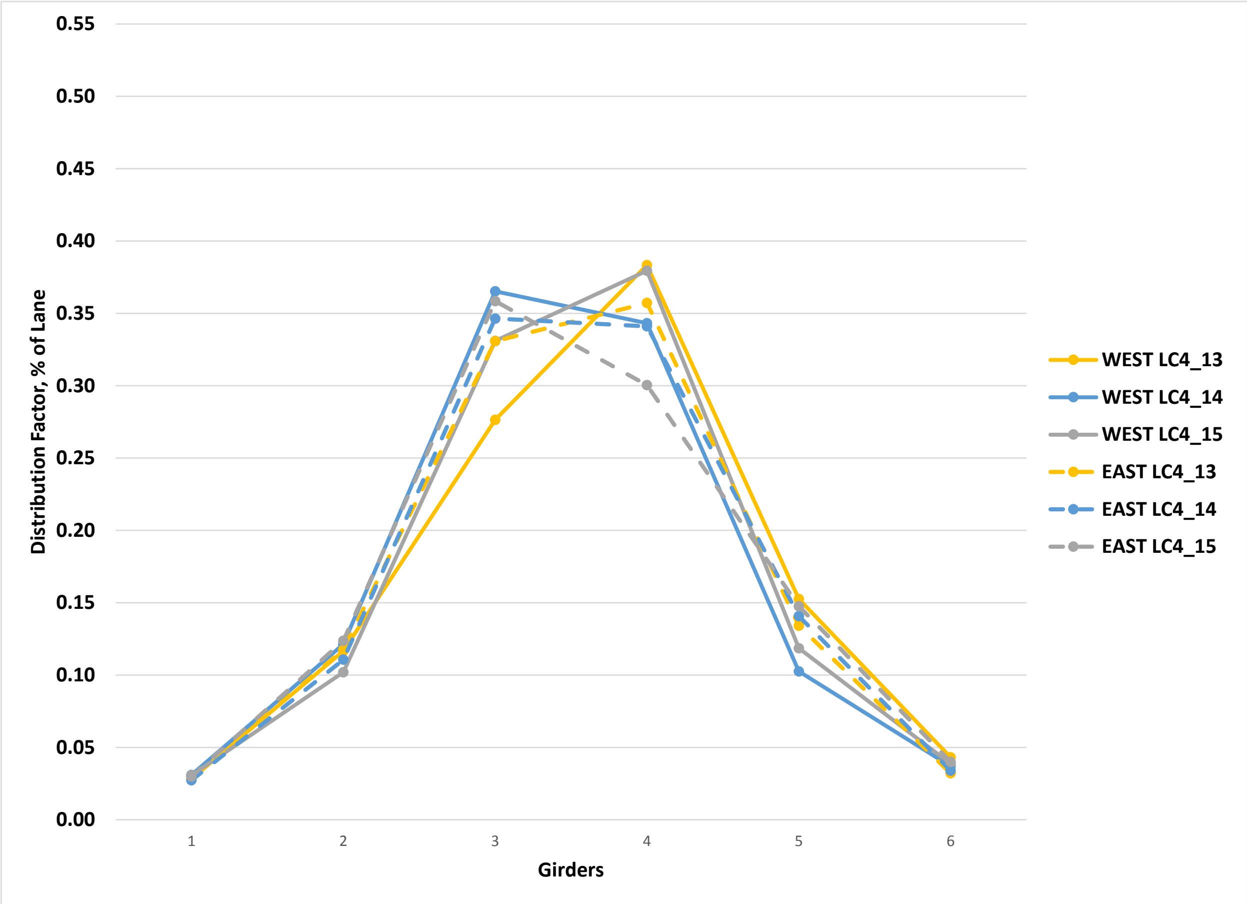


Figure 26. Transverse load distribution, Load Case 4

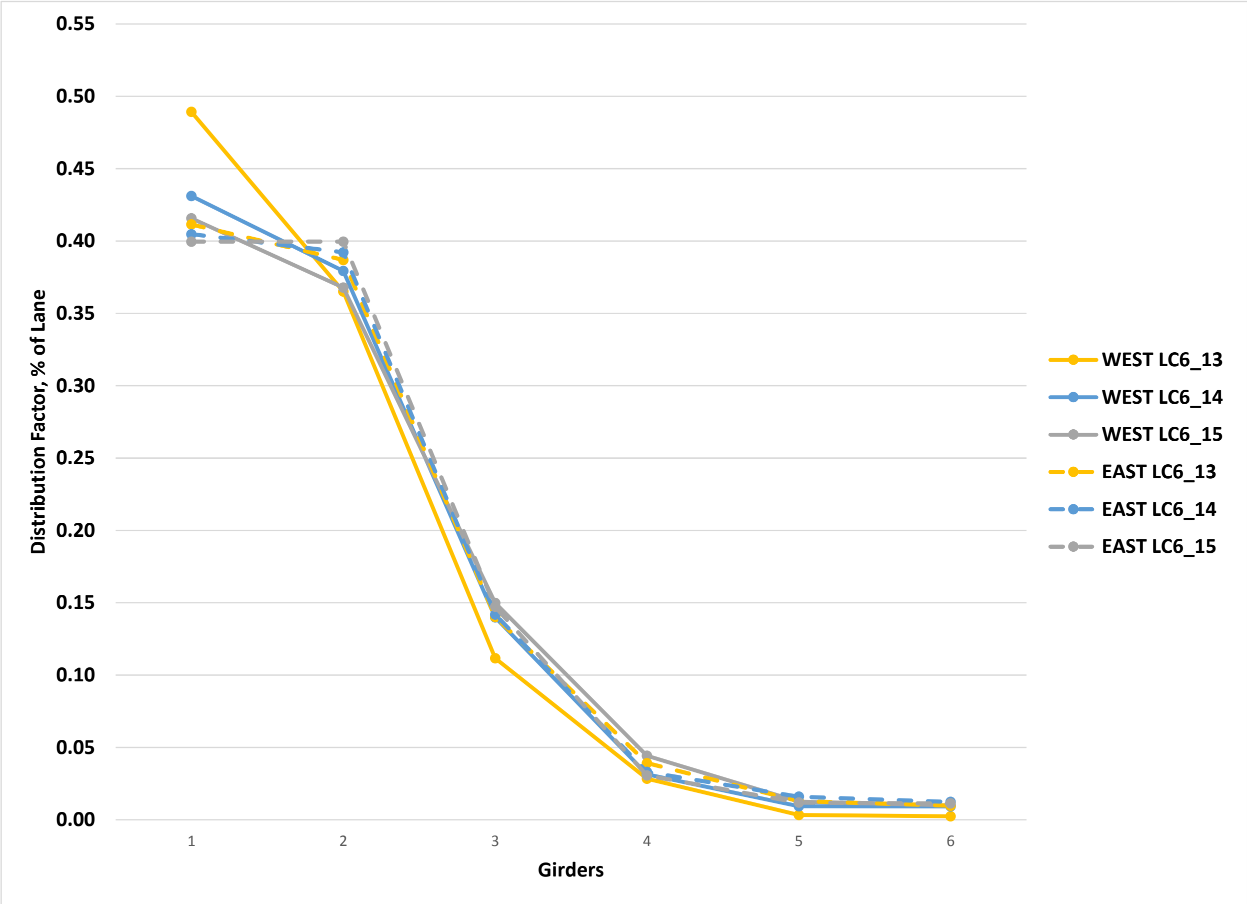


Figure 27. Transverse load distribution, Load Case 6

In all three graphs, the west span is represented by the solid colored lines, and the east span is represented by the dashed colored lines. In general, the transverse load distribution pattern for both the west and east spans were relatively similar when compared to each other for a given load case and from year to year, though some slight variances were evident. For instance, the east span appears to distribute more load to the girders nearest the truck, which could be a result of a number of factors or combination of factors, including but not limited to variances in girder stiffness, composite action, deck stiffness, girder end restraint, and the exact transverse placement of the load truck for a given load case. Further analysis of Figures 25 through 27 suggest, however, that in addition to the bridge possessing adequate transverse load distribution characteristics in both spans, the distribution of load is symmetric, as can be seen when comparing Figure 25 and 27 (they should be mirror images of each other), or by simply looking at Figure 26.

For the case in which two lanes were loaded, which involved superposition of Load Cases 3 and 5, some conclusions may be made about the performance of the west and east spans by inspecting the 2013, 2014, and 2015 graphs in Figures 28, 29, and 30, respectively.

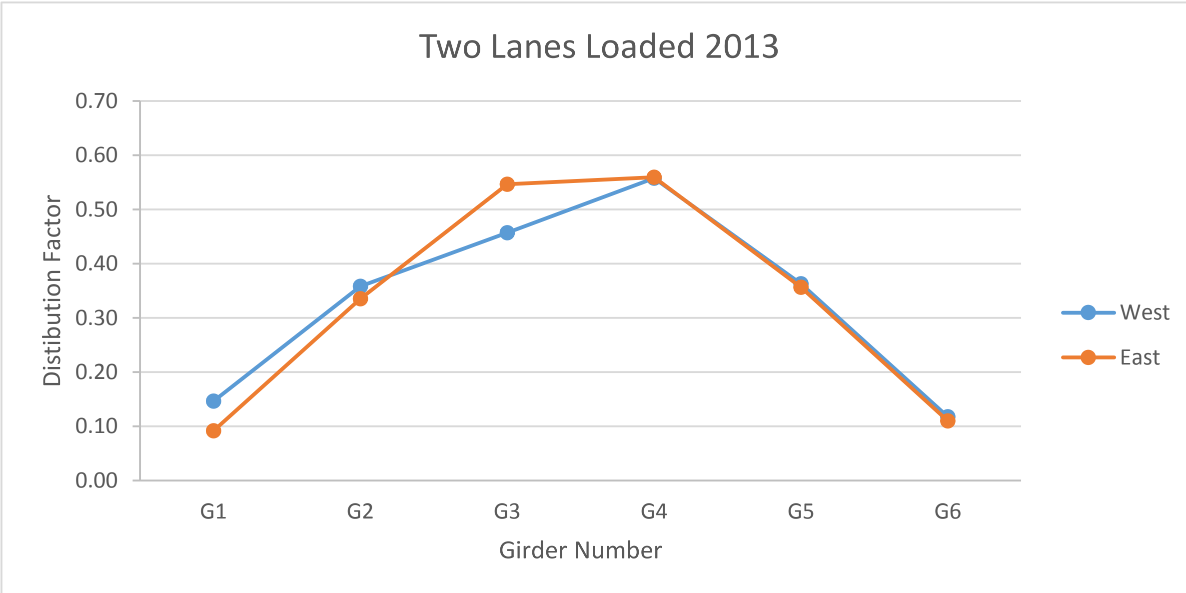


Figure 28. Distribution factors, two lanes loaded, 2013

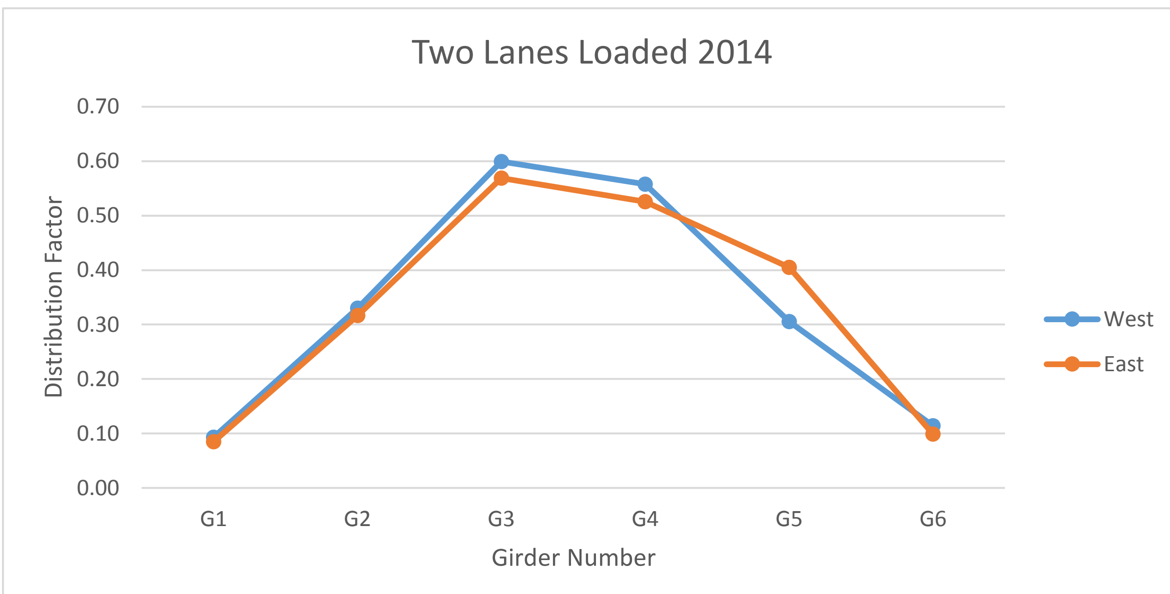


Figure 29. Distribution factors, two lanes loaded, 2014

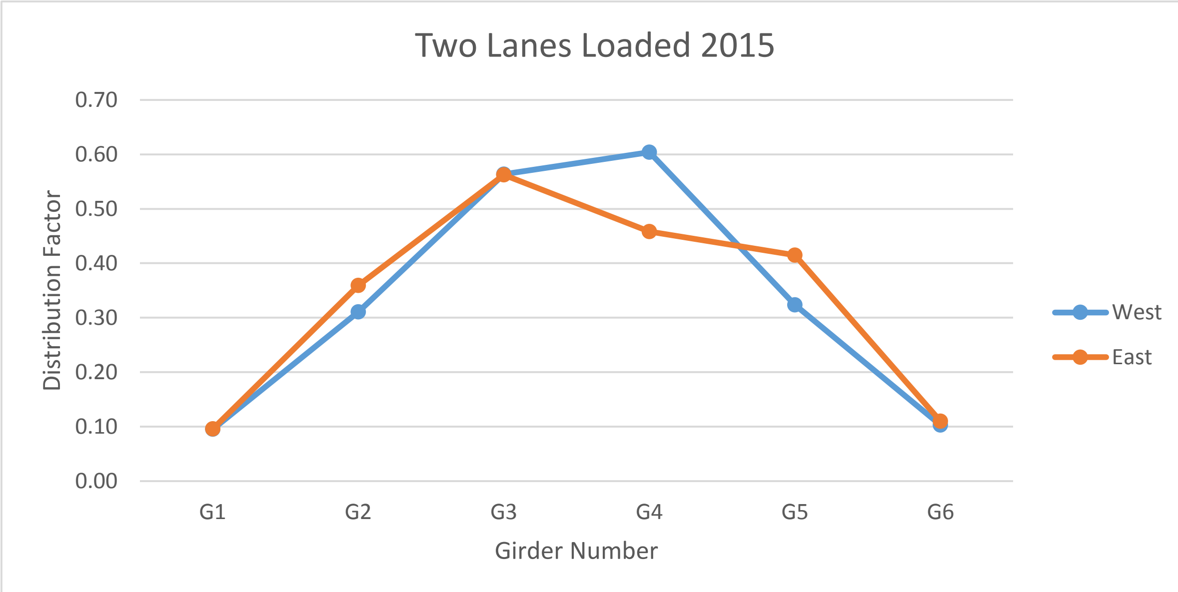


Figure 30. Distribution factors, two lanes loaded, 2015

In general, the west and east spans exhibit similar levels of transverse load distribution for any given year; additionally, though minor variances are evident, the performance of both spans remains relatively consistent from year to year, indicating little change in structural performance over the course of the three-year project.

Deck Strain Magnitudes

Deck strain magnitudes were evaluated using strain gauges mounted on the underside of the deck midway between each of the girders at midspan of both spans. To obtain a worst case scenario, Load Case 2 positioned the load truck such that its outside wheel line was directly over the deck gauge between girders G1 and G2. Figure 31 and 32 illustrate representative plots of peak girder and deck strains for both the west and east spans, respectively, for Load Case 2.

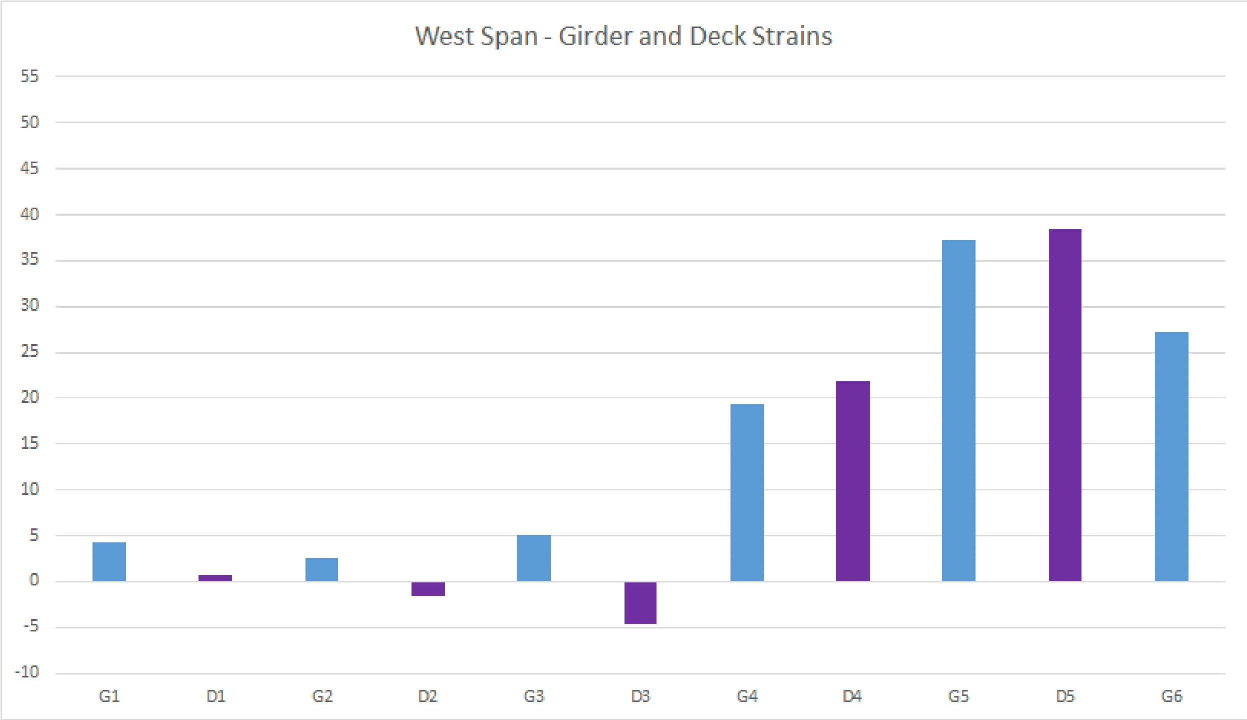


Figure 31. Typical girder and deck strain distribution for the west span

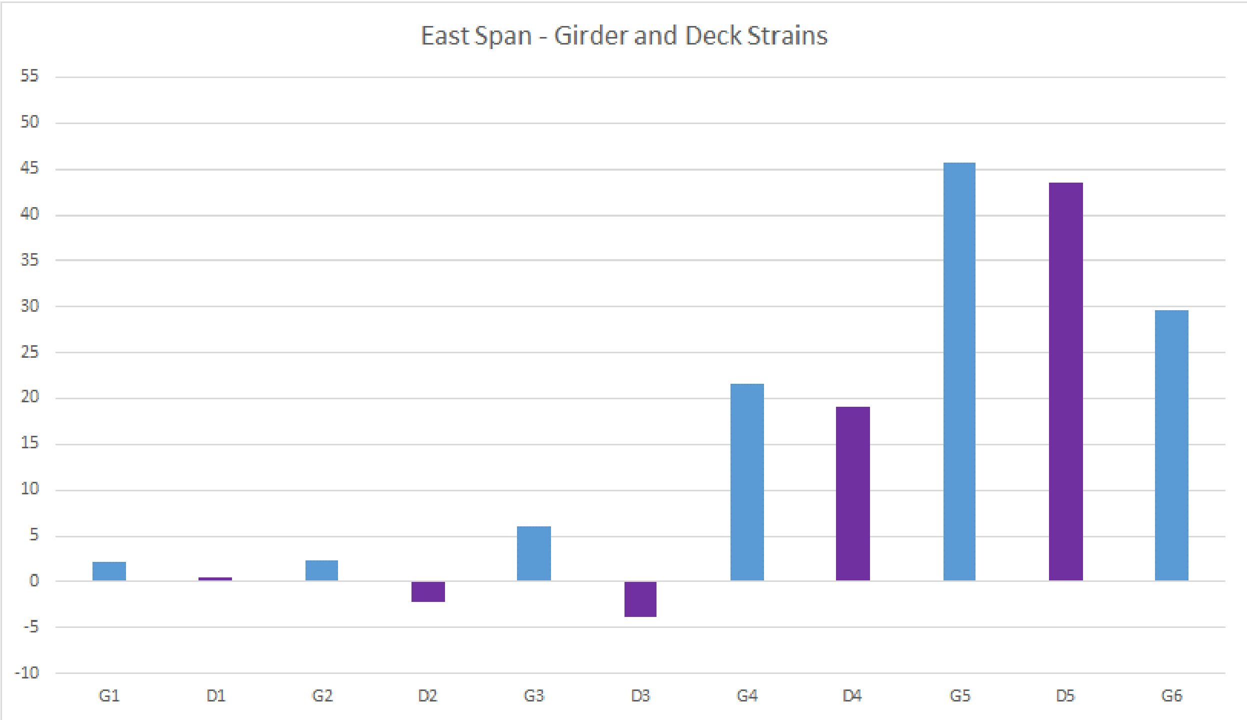


Figure 32. Typical girder and deck strain distribution for the east span

Peak strains in the deck gauges measured during Load Case 2 were in the range of 35–45 microstrain, which is relatively the same magnitude of strain measured in girders G1 and G2 during this load case. This suggests that the deck is adequately stiff, i.e., not flexing excessively between the girders under load. In addition, there was only minor variance in the magnitude of the deck strains from the west span to the east span, and, furthermore, little variance in the deck strain over the three years of test data. In general, there was no noticeable difference between the performance of the west and east spans under load.

Composite Action

Evaluation of the composite action of the bridge girders and deck was accomplished by evaluating the top and bottom flange strains of each girder, under load, for each load case. Figures 33 through 35 show the top and bottom flange strains for several girders versus truck position, which are representative of all tests conducted.

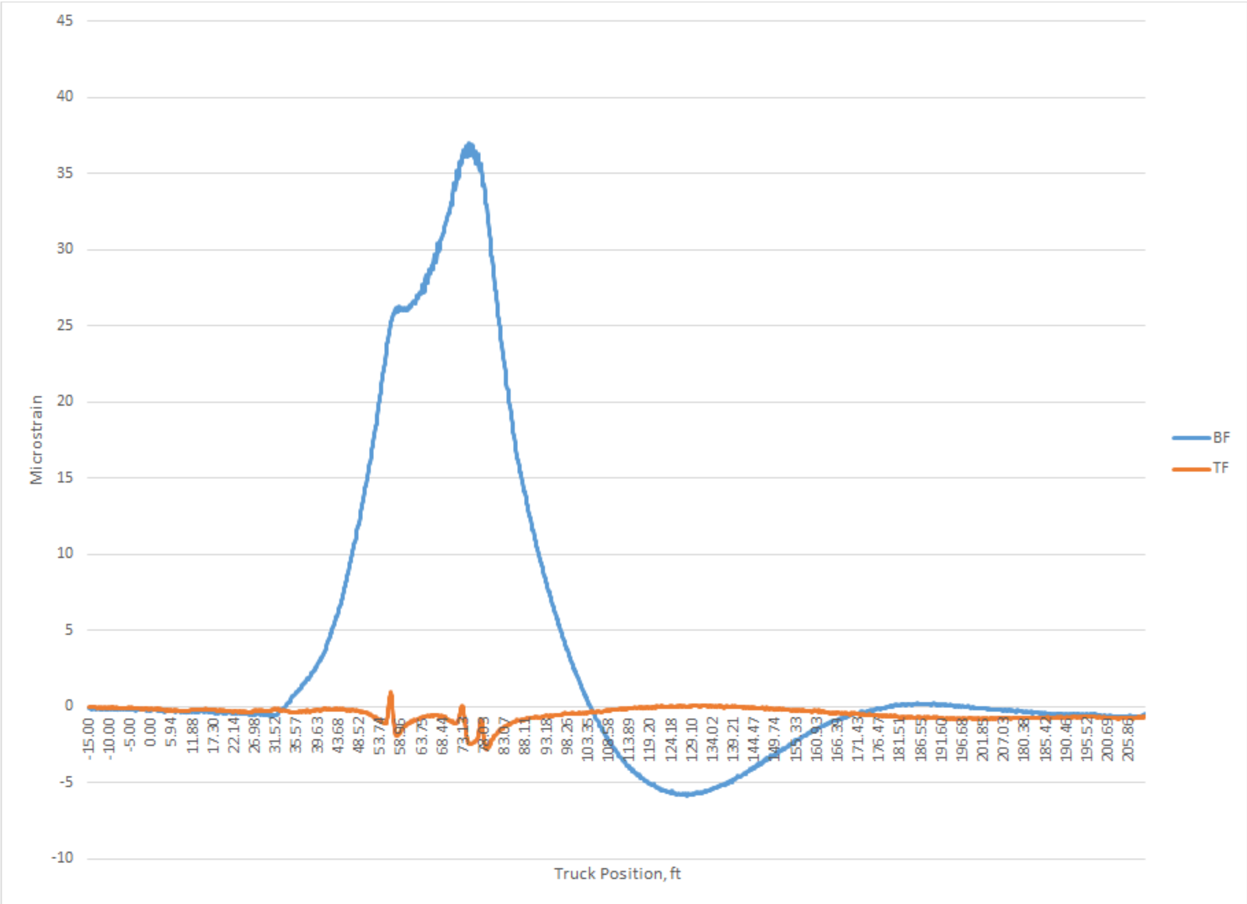


Figure 33. Top and bottom flange strains for girder G2, east span, Load Case 6, 2013

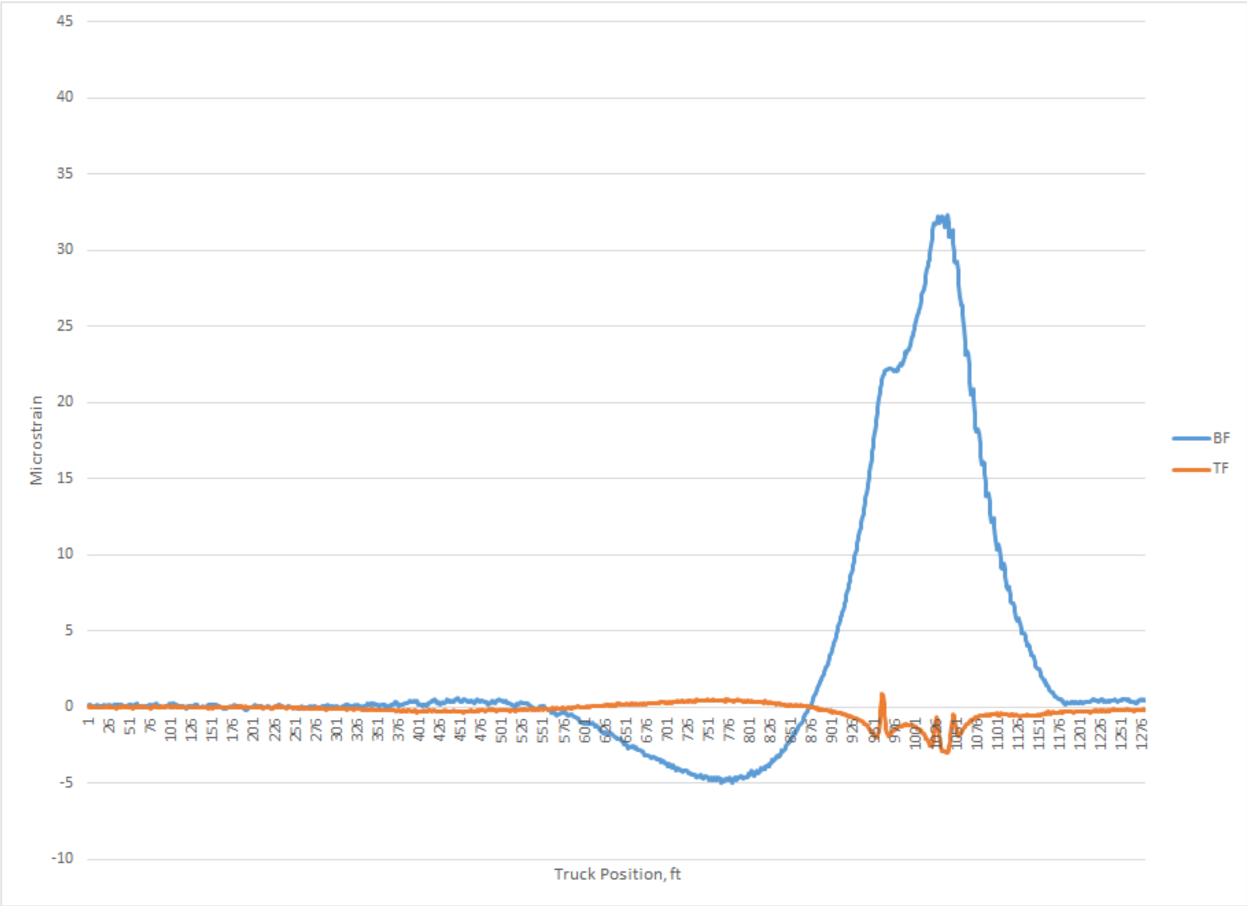


Figure 34. Top and bottom flange strains for girder G4, west span, Load Case 4, 2014

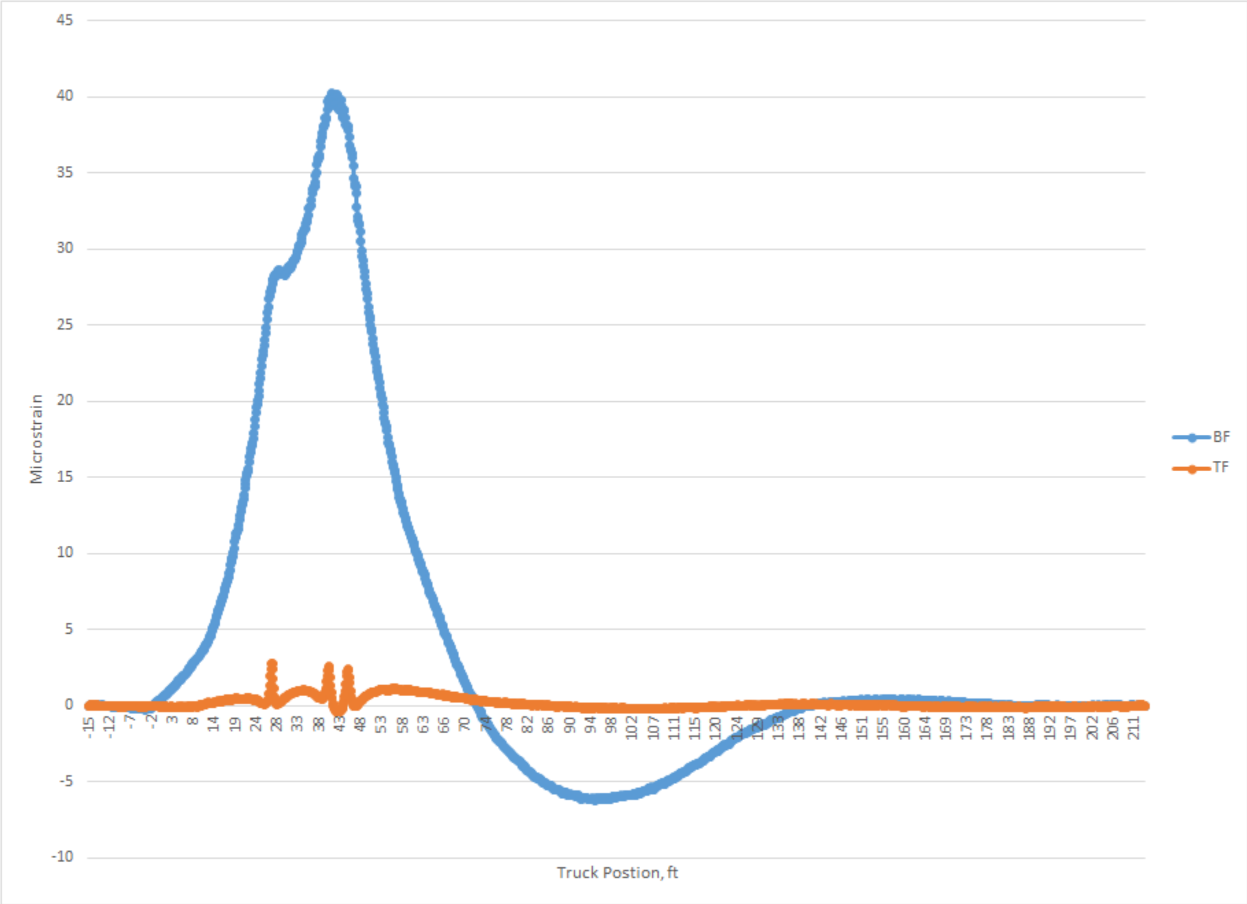


Figure 35. Top and bottom flange strains for girder G6, east span, Load Case 1, 2015

In each case, the bottom flange strains are relatively large compared to their corresponding top flange strains, indicating that the neutral axis of the section is located in the area of the top flange; in other words, the section is acting compositely.

To further verify and quantify the composite action behavior of the bridge cross-section, Equation 5 was used to calculate the location of the composite section neutral axis assuming a linear-elastic behavior.

$$\bar{y} = 37.5 in. \frac{(\varepsilon_{BF})}{(\varepsilon_{BF} - \varepsilon_{TF})}, \text{ composite section neutral axis} \qquad \textbf{(5)}$$

where,

$\bar{y}$ = distance to neutral axis measured from bottom of bottom flange

ε_{TF} = measured top flange strain

ε_{BF} = measured bottom flange strain

37.5 in. = distance between bottom flange gauge and top flange gauge

The total girder depth of the prestressed concrete girders was measured at approximately 39 in., and the deck thickness is 8 in. For all three load tests conducted on the bridge, the bottom flange strain gauges were installed directly to the underside of the girder at the mid-width of the bottom flange (or bell, as this area is often called on pretensioned prestressed concrete beams), and the top flange strain gauges were installed approximately 1.5 in. down from the deck on the side of the top flange. This gives a distance of 37.5 in. between locations of strain measurement.

Using Equation 5 and the measured strains from the three load tests conducted on the bridge, the average composite neutral axis of the exterior and interior bridge girders for both spans were calculated. For load cases where the load was near the guardrail, as with Load Cases 1 and 6, the calculated composite neutral axis for the exterior girders was approximately 37 in., which is near the middle of the top flange. When the load was located towards the center of the bridge, as with Load Cases 3 through 5, the calculated composite neutral axes for the exterior and interior girders were approximately 30 in. and 35 in., respectively, or just below the top flange of the girder. All of these calculated neutral axis values indicate adequate composite section behavior for both spans of the LWFA bridge. Note that the slightly higher neutral axis for the exterior girders when the truck is near the curb is possibly the result of added stiffness from the concrete guardrail.

CONCLUSIONS

The objectives of this work were as follows:

- Evaluate the performance of the LWFA material through laboratory and field testing
- Evaluate the structural performance through live load tests of the finished structure at the time of construction and after one year and two years of service
- Conduct a lifecycle cost and service life prediction

The data showed that replacement of about 20% of the fine aggregate with lightweight material for the purposes of providing internal curing resulted in the following:

- The unit weight and stiffness of the IC mixture were slightly lower than those of the control, as expected.
- The effect of the IC mixes on the elapsed time to reach peak temperature was small.
- Both the 20% and 30% IC mixes did not dramatically affect the mechanical properties, but the increased LWFA dosage seemed to improve the permeability.
- Based on the average stress rate values, all three mixtures were classified as having a “low” potential for cracking.
- Surface resistivity readings on the bridge deck indicated a marked benefit in the IC section compared to the control.
- Little difference could be detected in the structural performance of the test and control sections after two years.
- The service life for the IC section is predicted to be about 20 years longer than that of the control section.

Structural testing of the bridge after construction revealed similar levels of performance for the two end spans in terms of transverse load distribution, strain magnitudes in the deck, and composite action. No significant differences were evident in the data to suggest that the LWFA deck behaved any differently than the normal weight concrete deck.

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