Report No. K-TRAN: KU-11-8 ▪ FINAL REPORT ▪ November 2013

Use of Innovative Concrete Mixes for Improved Constructability and Sustainability of Bridge Decks

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The University of Kansas

A cooperative transportation research program between
Kansas Department of Transportation,
Kansas State University Transportation Center, and
The University of Kansas

Kansas
Department of Transportation

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1 **Report No.** K-TRAN: KU-11-8	2 **Government Accession No.**	3 **Recipient Catalog No.**
4 **Title and Subtitle** Use of Innovative Concrete Mixes for Improved Constructability and Sustainability of Bridge Decks		5 **Report Date** November 2013
		6 **Performing Organization Code**
7 **Author(s)** Pankaj Narayan Shrestha; Amber Harley; Benjamin Pendergrass, Ph.D., P.E.; David Darwin, Ph.D., P.E.; JoAnn Browning, Ph.D., P.E.		8 **Performing Organization Report No.**
9 **Performing Organization Name and Address** The University of Kansas Civil, Environmental & Architectural Engineering Department 1530 West 15th Street Lawrence, Kansas 66045-7609		10 **Work Unit No. (TRAIS)**
		11 **Contract or Grant No.** C1872
12 **Sponsoring Agency Name and Address** Kansas Department of Transportation Bureau of Research 2300 SW Van Buren Street Topeka, Kansas 66611-1195		13 **Type of Report and Period Covered** Final Report July 2010–May 2013
		14 **Sponsoring Agency Code** RE-0559-01
15 **Supplementary Notes** For more information, write to address in block 9..		

16 **Abstract**

Bridge deck crack surveys were performed on twelve bridges on US-59 south of Lawrence, Kansas, to determine the effects of mixture proportions, concrete properties, deck type, and girder type on the crack density of reinforced concrete bridge decks. Of the twelve decks surveyed, eight are supported by prestressed concrete girders and four are supported by steel girders. Four of the decks supported by prestressed girders are cast on partial-depth precast deck panels, two are monolithic with synthetic fibers, and two have overlays. Of the four decks supported by steel girders, two have silica fume overlays (SFO) and two are monolithic. One of two decks with a silica fume overlay contains synthetic fibers in the overlay. Following the surveys, crack maps were plotted and analyzed and cracking trends were observed. The results for the US-59 bridge decks are compared with crack densities obtained in a study of low-cracking high-performance concrete (LC-HPC) bridge decks.

The monolithic concrete bridge decks supported by prestressed concrete girders within this study exhibit less cracking than decks supported by steel girders. At an age of approximately three and a half years, the US-59 monolithic decks supported by prestressed girders with deck panels are not displaying significant cracking; most of the cracks are short transverse cracks aligned with the joints between the deck panels. The US-59 decks supported by prestressed girders with overlays exhibit significantly more cracking than the decks on prestressed girders without overlays. Bridge decks supported by steel girders without overlays have slightly higher crack densities than the decks with overlays. No benefits of using fibers in either the overlay or deck have been observed in this study, the sample size, however, is small. An increase in crack density was observed with an increase in average concrete slump for decks supported by both prestressed and steel girders. Decks with deck panels supported by prestressed girders exhibited an increased crack density with an increase in paste content. .

17 **Key Words** Bridge decks, high-performance concrete, cracking, fibers		18 **Distribution Statement** No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19 **Security Classification (of this report)** Unclassified	20 **Security Classification (of this page)** Unclassified	21 **No. of pages** 113	22 **Price**

Form DOT F 1700.7 (8-72)

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A Report on Research Sponsored by

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

and

THE UNIVERSITY OF KANSAS
LAWRENCE, KANSAS

November 2013

PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Abstract

Bridge deck crack surveys were performed on twelve bridges on US-59 south of Lawrence, Kansas, to determine the effects of mixture proportions, concrete properties, deck type, and girder type on the crack density of reinforced concrete bridge decks. Of the twelve decks surveyed, eight are supported by prestressed concrete girders and four are supported by steel girders. Four of the decks supported by prestressed girders are cast on partial-depth precast deck panels, two are monolithic with synthetic fibers, and two have overlays. Of the four decks supported by steel girders, two have silica fume overlays (SFO) and two are monolithic. One of two decks with a silica fume overlay contains synthetic fibers in the overlay. Following the surveys, crack maps were plotted and analyzed and cracking trends were observed. The results for the US-59 bridge decks are compared with crack densities obtained in a study of low-cracking high-performance concrete (LC-HPC) bridge decks.

The monolithic concrete bridge decks supported by prestressed concrete girders within this study exhibit less cracking than decks supported by steel girders. At an age of approximately three and a half years, the US-59 monolithic decks supported by prestressed girders with deck panels are not displaying significant cracking; most of the cracks are short transverse cracks aligned with the joints between the deck panels. The US-59 decks supported by prestressed girders with overlays exhibit significantly more cracking than the decks on prestressed girders without overlays. Bridge decks supported by steel girders without overlays have slightly higher crack densities than the decks with overlays. No benefits of using fibers in either the overlay or deck have been observed in this study, the sample size, however, is small. An increase in crack density was observed with an increase in average concrete slump for decks supported by both prestressed and steel girders. Decks with deck panels supported by prestressed girders exhibited an increased crack density with an increase in paste content.

Acknowledgements

This report is based on research performed by Pankaj Narayan Shrestha and Amber Harley in partial fulfillment of the requirements for the MSCE degree from the University of Kansas. Support was provided by the Kansas Department of Transportation under K-TRAN Project KU-11-8 with technical oversight by David Meggers and John Jones.

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Chapter 1: Introduction

Cracking is a problem for most bridge decks because cracks provide direct access of deicing chemicals to the reinforcing steel and reduce the freeze-thaw resistance of the concrete. Cracking is affected by a number of factors, including concrete mixture proportions, plastic concrete properties, weather conditions during construction, construction procedures, and the age of the bridge deck. The Kansas Department of Transportation (KDOT) has been working to minimize cracking in bridge decks for several decades. A pooled-fund study is being conducted by the University of Kansas to reach this goal. KDOT is also pursuing other efforts to achieve this goal.

The vehicle for achieving minimal cracking in the pooled-fund study has been through specifications for Low-Cracking High-Performance Concrete (LC-HPC) bridge decks. These specifications address cement and water content, aggregate content, concrete properties, construction methods, and curing requirements. Sixteen bridge decks have been constructed in Kansas in accordance with LC-HPC specifications. As a part of the project, crack surveys of the bridge decks have been conducted annually. A standard procedure has been developed for the surveys so that consistent data are obtained from year to year. The results of that study demonstrate that the LC-HPC bridge decks are performing better than bridge decks constructed per standard KDOT specifications across the state (Lindquist et al. 2008; McLeod et al. 2009; Darwin et al. 2010, 2012; Yuan et al. 2011).

In addition to LC-HPC bridge decks, KDOT has constructed a number of bridge decks using innovative concrete mixtures in an effort to identify other approaches to minimize cracking. This report addresses six pairs of bridges on US-59 south of Lawrence, Kansas, with each pair consisting of a northbound and a southbound bridge at the same location. The concrete mixtures contain different combinations of cementitious materials, aggregates, and fibers. Some of the mixtures have similarities to LC-HPC. As a result, the decks in the two projects are compared in this report. The twelve bridges on US-59 include bridges supported by prestressed and steel girders, a point that is of interest because research dating back over four decades indicates that bridge decks supported by prestressed girders crack less than decks supported by steel girders (PCA 1970). Full-depth cast-in-place decks were used on eight of the bridges, while precast concrete deck panels with reinforced cast-in-place toppings were used on the other four. The crack surveys follow the same procedure as used for the LC-HPC bridge decks (Appendix A) and are conducted annually. This report summarizes the crack surveys performed between the summer of 2010 and the summer of 2012.

1.1 Background

This section provides background information that is used throughout the report. It recognizes problems that have been exhibited in the past by bridges with deck panels (Wenzlick 2005, Sneed et al. 2010). It also highlights past research at the University of Kansas to identify problems with silica fume overlays (Lindquist et al. 2005) and the importance of a 14-day wet cure when using cementitious material blends in bridge-deck concrete (Lindquist et al. 2008).

The Missouri Department of Transportation (MoDOT) has been using partial-depth precast-prestressed concrete panels since the 1980s. Spalling has been observed in some of these bridge decks due to rusting of the embedded steel reinforcement. Cracking at the joints between deck panels has been a problem due to restrained shrinkage of cast-in-place concrete at the joints. Wenzlick (2005) found that cracking nearly doubled for decks supported by prestressed girders with partial-depth precast panels compared to cast-in-place decks. MoDOT is currently investigating solutions to these problems since deck panels are cost-effective for deck construction.

Prior to the LC-HPC study, University of Kansas surveyed 30 bridge decks with silica fume overlays along with 17 monolithic decks. Of these bridges, 13 monolithic and 20 silica fume overlay bridge decks were surveyed two or more times by 2005 (Lindquist et al. 2005). The latter decks include both 5% and 7% silica fume overlay decks. The mean crack densities for the 5% and 7% silica fume overlay decks were essentially the same; therefore, all silica fume overlays were considered as a single deck type. At 42 months, the mean crack density for the monolithic bridge decks was found to be 0.203 m/m^2, which was significantly lower the mean crack density of 0.565 m/m^2 for silica fume overlay decks.

The effect of curing period on mixes with slag, silica fume, and the combination of both slag and silica fume was studied by Yuan et al. (2011). This work included mixes containing

Grade 120 and Grade 100 slag, which provided similar results. Six shrinkage specimens were fabricated for each mix. Half were cured for 7 days and half were cured for 14 days. The use of silica fume, slag, or both in these mixes reduced the shrinkage, but only for the specimens that were cured for 14 days. The study demonstrated that to significantly reduce shrinkage when using silica fume, slag, or both, the curing period must be at least 14 days.

Many of the bridge decks evaluated in this study have concrete properties similar to LC-HPC decks. The LC-HPC study (Lindquist et al. 2008, McLeod et al. 2009, Darwin et al. 2010, 2012, Yuan et al. 2011) includes control decks, most of which have silica fume overlays, and decks following the LC-HPC specifications. The current LC-HPC concrete specification permits cement contents between 500 and 540 lb/yd^3 (297 and 320 kg/m^3), a water/cement (w/c) ratio of 0.44 to 0.45, a concrete slump between 1 and 3½ inches (25 to 90 mm), an air content of 6.5 to 9.5%, 28-day compressive strengths of 3500 to 5500 psi (24.1 to 37.9 MPa), and concrete temperatures at the time of placement between 55 and 70°F (13 and 21°C). The current specifications for LC-HPC bridge decks are given in Appendix B. Since the concrete properties for the US-59 bridges without overlays are closer to LC-HPC bridge decks than most of the other bridge decks in Kansas, the decks on US-59 are compared with the LC-HPC bridges in this report. The bridge decks that have overlays (US-59 5, 6, 9 and 11) are compared with the control decks in the LC-HPC study that have overlays.

The LC-HPC specifications include provisions for aggregates and construction procedures, including requirements for finishing and curing techniques. The specifications require using either a single-drum roller or a vibrating screed for strike off followed by a burlap drag, metal pan, or both for finishing. Tining of plastic concrete is prohibited. Wet burlap must be placed within 10 minutes of strike off with soaker hoses placed over the burlap and covered with a plastic within 12 hours and left in place to provide a 14-day curing period. At the end of the 14-day curing period, the specification stipulates that two coats of an opaque curing membrane must be applied within 30 minutes of burlap removal.

1.2 Bridges

Three contractors were involved in the construction of the US-59 bridge decks. Ames Construction constructed eight decks, while Beachner Construction Co. and Reece Construction Co. constructed two decks each. The bridges consist of monolithic decks on prestressed girders, decks with silica fume overlays on prestressed girders, deck panels topped with monolithic concrete on prestressed girders, monolithic decks on steel girders, and decks with silica fume overlays on steel girders. All of the decks are 8½ inches (216-mm) thick with 3 inches (76-mm) of top cover over the reinforcing steel and have abutments that are integral with the bridge deck, providing a fixed condition at the ends of the girders. The bridge IDs, KDOT bridge numbers, bridge types, contractors, reinforcing bar sizes, reinforcing bar spacing, bridge skews, and bridge lengths are summarized in Table 1.1.

TABLE 1.1
Bridge Properties

Bridge ID	KDOT Bridge No.	Contractor*	Girder and Deck Type**	Bridge Skew	Bridge Length		Total Deck Thickness		Transverse Steel				Angle of Reinf.
									Size		Spacing		
				(deg.)	(ft)	(m)	(in.)	(mm)	No.	(mm)	(in.)	(mm)	(deg.)
US-59 1	59-30-19.92	Ames	Steel - M	45.63	387.9	118.2	8.5	216	5	16	6	152	0
US-59 2	59-30-19.91	Ames	Steel - M	45.63	387.9	118.2	8.5	216	5	16	6	152	0
US-59 3	59-30-20.05	Ames	PS w/ DP	8.43	242.9	74.0	8.5	216	5	16	6	152	0
US-59 4	59-30-20.04	Ames	PS w/ DP	8.43	242.9	74.0	8.5	216	5	16	6	152	0
US-59 5	59-30-21.84	Ames	Steel w/ O[F]	39.17	264.8	80.7	8.5	216	5	16	7	178	0
US-59 6	59-30-21.85	Ames	Steel w/ O	39.17	266.2	81.1	8.5	216	5	16	7	178	0
US-59 7	59-30-18.76	Ames	PS w/ DP	2.3	333.5	101.7	8.5	216	5	16	7	178	0
US-59 8	59-30-18.75	Ames	PS w/ DP	2.3	333.5	101.7	8.5	216	5	16	7	178	0
US-59 9	59-30-24.51	Beachner	PS w/ O	0	225.5	68.7	8.5	216	5	16	6	152	0
US-59 10	59-30-24.50	Beachner	PS -M[F]	0	225.5	68.7	8.5	216	5	16	6	152	0
US-59 11	59-30-24.82	Reece	PS w/ O	0	172.5	52.6	8.5	216	5	16	7	178	0
US-59 12	59-30-24.83	Reece	PS - M[F]	0	172.5	52.6	8.5	216	5	16	7	178	0

*Ames = Ames Construction, Beachner = Beachner Construction Co., Inc., General Contractor, Reece = Reece Construction Company, Inc.
**PS = Prestressed concrete girder, DP = Deck panels, O = Deck with silica fume overlay, M = Monolithic deck
[F]Fibers in the deck or overlay

Bridges US-59 1, 2, 10, and 12 have cast-in-place monolithic decks. Bridges US-59 5, 6, 9 and 11 have 7 inches (178-mm) thick cast-in-place subdecks with 1½ inches (33-mm) thick silica fume overlays. Bridges US-59 3, 4, 7, a nd 8 have 3 inches (76-mm) thick deck panel stopped with 5½ inches (140-mm) cast-in-place reinforced concrete. The panels for US-59 3 and 4 are approximately 7 × 9 ft (2.13 × 2.74 m) and the panels for US-59 7 and 8 are 8 × 9 ft (2.44 × 2.74 m). All deck panels had a design strength of 5000 psi (34.5 MPa) and were manufactured by Core Slab (Kansas), Inc.

1.3 Concrete Properties and Construction Procedures

The mixture proportions for the bridge decks, shown in Table 1.2, vary by type of cementitious material (portland cement, slag cement, and silica fume), type of aggregate (granite, limestone, and river sand), w/c ratio (0.42 to 0.45), and type of fibers (Grace 90/40 Strux and Grace fibers), if used. The Grace 90/40 Strux fibers are 1.55 inches long macro synthetic fibers made with polyolefin. A quantity of 5 lb/yd^3 of the fibers was used in the concrete. The Grace fibers are fibrillated polypropylene micro synthetic fibers. They are ¾ inches long and 3 lb/yd^3 was used in the concrete. The mix designs for the silica fume overlays on US-59 5, US-59 6, US-59 9, and US-59 11 are shown in Table 1.3. The plastic concrete properties, concrete strengths for the decks and subdecks (in the case of decks with overlays), the range of and average of construction day air temperatures, and the average concrete temperature listed in Table 1.4. Concrete slump ranged from 2½ to 5 inches (65 to 115 mm), air content ranged from 6 to 8%, and compressive strength at 28 days ranged from 4100 to 6390 psi (28.3 to 44.0 MPa). Most of the slumps, air contents, and compressive strengths were within or just outside of the LC-HPC specified ranges with a few exceptions. US-59 2 had the highest compressive strength of 6390, and US-59 5 and US-59 11 had the highest slumps of 5 and 4¾ inches (125 and 120 mm), respectively.

TABLE 1.2
Mixture Proportions for Decks or Subdecks of Decks with Silica Fume Overlays

Bridge ID	Date of Placement	Cementitious Material**	Fibers in Deck	Aggregates by Weight***	Water Content		Cementitious Material		w/c Ratio	% Paste by Vol.	Types of Admix.* ***
					(lb/yd^3)	(kg/m^3)	(lb/yd^3)	(kg/m^3)			
US 59-1	11/13/2008	60% C, 35% S., 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	134	540	317	0.42	23.99	AEA, Type A
US 59-2	11/25/2008	60% C, 35% S, 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	133	540	318	0.42	23.99	AEA, Type A
US 59-3	9/30/2008	65% C, 35% S	NA	45% CA-2, 15.2% CA-3, 39.8% FA	241	143	540	317	0.45	24.77	AEA, Type A
US 59-4	9/19/2008	65% C, 35% S	NA	45% CA-2, 15.2% CA-3, 39.8% FA	241	143	540	317	0.45	24.77	AEA, Type A
US 59-5*	5/14/2008	100% C	NA	50% CA-1, 50% FA	274	163	620	369	0.44	27.95	AEA, Type A
US 59-6*	4/30/2008	100% C	NA	50% CA-1, 50% FA	274	163	620	369	0.44	27.95	AEA, Type A
US 59-7	11/1/2008	60% C, 35% S, 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	134	540	317	0.42	23.99	AEA, Type A
US 59-8	10/29/2008	60% C, 35% S, 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	134	540	317	0.42	23.99	AEA, Type A
US 59-9*	10/21/2008	100% C	NA	50% CA-1, 50% FA	259	154	600	358	0.44	26.68	AEA, Type A
US 59-10	12/6/2008	100% C	5 lb/yd^3 WR Grace 90/40 Strux^F	50% CA-1, 50% FA	237	141	560	334	0.42	24.62	AEA, Type A
US 59-11*	10/3/2008	100% C	NA	50% CA-1, 50% FA	274	163	620	369	0.44	27.95	AEA, Type A
US 59-12	1/9/2009	100% C	3 lb/yd^3 Grace Fibers^F	50% CA-1, 50% FA	237	141	560	334	0.42	24.62	AEA, Type A

*Bridges have overlays and proportions are for the subdecks.
**C = Cement, S = Slag, SF = Silica fume
***CA-1= ½ in. Crushed limestone, CA-2 = ¾ in. Crushed granite, CA-3= ½ in. Crushed granite , FA= River sand
**** AEA = Air entraining agent, Type A = Type A water reducer
^{F}WR Grace 90/40 Strux = 1.55 in. long polyolefin macro fibers, Grace Fibers = ¾ in. long fibrillated polypropylene micro fibers

TABLE 1.3
Silica Fume Overlay Mix Designs

Bridge ID	Cementitous Material*	Fibers in Overlay	Aggregates by Weight**	Water Content (lb/yd^3)	Water Content (kg/m^3)	Cementitious Material (lb/yd^3)	Cementitious Material (kg/m^3)	w/c Ratio	% Paste by Volume	Types of Admix. ***
US-59 5	66% C, 30.1 S, 3.9% SF	5lb/yd^3 WR Grace 90/40 Strux[F]	50% CA-1 50% FA	239	142	645	382	0.37	23.54	AEA, Type A
US-59 6	66% C, 30.1 S, 3.9% SF	NA	50% CA-1 50% FA	239	142	645	382	0.37	23.54	AEA, Type A
US-59 9	92.2% C, 7.8% SF	NA	50% CA-1 50% FA	239	142	645	382	0.37	23.54	NA
US-59 11	92.2% C, 7.8% SF	NA	50% CA-1 50% FA	239	142	645	382	0.37	23.54	AEA, Type A

*C = Cement, S = Slag, SF = Silica fume
**CA-1= ½ in. Crushed limestone, FA= River sand
*** AEA = Air entraining agent, Type A = Type A water reducer
[F]WR Grace 90/40 Strux = 1.55 in. long polyolefin macro fibers

TABLE 1.4
Average Plastic Concrete Properties, Air Temperature at Time of Placement, and Concrete Compressive Strength

Bridge ID	Slump		Air Content	Average Concrete Temp.		Air Temperature								Average Concrete Temp. Minus Average Air Temp.		28-Day Strength (psi)	
						Low		High		Range		Average					
	(in.)	(mm)	(%)	(F)	(C)	(F)	(C)	(F)	(C)	(F)	(C)	(F)	(C)	(F)	(C)	(psi)	(MPa)
US-59 1	4	100	6.50	65.5	18.6	42	5.6	57	14	15	8	50	10	15.5	8.6	5090	35.1
US-59 2	3½	90	6.75	65.3	18.5	27	-2.8	51	11	24	13	39	4	26.3	14.5	6390	44.1
US-59 3	4	100	7.25	76.9	24.9	45	7.2	72	22	27	15	58	14	18.9	10.9	4260	29.4
US-59 4	4	100	6.75	78.7	26.0	57	13.9	79	26	22	12	68	20	10.7	6.0	5000	34.5
US-59 5	5	130	6.75	65.0	18.3	46	7.8	66	19	20	11	55	13	10.0	5.3	5010	34.5
US-59 6	4½	115	6.25	66.0	18.9	48	8.9	79	26	31	17	63	17	3.0	1.9	4850	33.4
US-59 7	3¼	80	6.25	68.3	20.2	46	7.8	71	22	25	14	58	14	10.3	6.2	4720	32.5
US-59 8	2½	65	6.25	66.2	19.0	32	0.0	66	19	34	19	49	9	17.2	10.0	4580	31.6
US-59 9	3¾	95	6.25	71.3	21.8	48	8.9	59	15	11	6	54	12	17.3	9.8	5110	35.2
US-59 10	3	75	7.00	63.7	17.6	21	-6.1	45	7	24	13	34	1	29.7	16.6	5100	35.2
US-59 11	4¾	120	7.75	76.3	24.6	46	7.8	75	24	29	16	60	16	16.3	8.6	4480	30.9
US-59 12	4	100	7.00	61.5	16.4	27	-2.8	59	15	32	18	44	7	17.5	9.4	5740	39.6

*Bridges have overlays and properties listed are for the subdecks.

TABLE 1.5
Average Overlay Plastic Concrete Properties and Compressive Strengths

Bridge ID	Slump		Air Content	Concrete Temp.		28-Day Strength	
	(in.)	(mm)	(%)	(F)	(C)	(psi)	(MPa)
US-59 5	4½	115	6.75	81.0	27.2	6450	44.5
US-59 6	¾	20	7.75	74.0	23.3	7480	51.6
US-59 9	4	100	7.00	58.0	14.4	9100	62.7
US-59 11	3¼	85	7.25	70.0	21.1	5470	37.7

The difference between the average concrete and air temperatures at the time of placement for the decks supported by steel girders ranged from 3.0 to 26.3 °F (1.7 to 14.6 °C), with an average difference of 13.7 °F (7.6 °C). The difference between the average concrete and air temperatures for LC-HPC decks supported by steel girders ranged from -7 to 27.4 °F (-3.9 to 15.2 °C), with an average difference of 6.1 ° F (3.4 °C) (Yuan et al. 2011). The difference between the average concrete and average air temperatures are thus, higher for the decks on US-59. Only US-59 6 ha d a temperature difference below the average LC-HPC temperature difference. Because air temperature serves as a proxy for girder temperature, a higher concrete temperature relative to the air temperature indicates a greater potential for cracking in the US-59 decks due to subsequent contraction of the deck with respect to the girders.

The US-59 bridges were tined, which is prohibited for LC-HPC decks. Taking the time to tine the plastic concrete typically delays the initiation of curing for an hour or more, allowing the concrete to dry prior to initiating curing. The decks were cured for 14 days using wet burlap. It is not known if plastic was used to cover the burlap, as is required for LC- HPC decks.

1.4 Crack Surveys

The crack surveys described in this report were conducted after the bridge decks were opened to traffic. At this writing, half of US-59 bridges were surveyed in 2010 and all twelve bridges were surveyed in 2011 and 2012.

1.4.1 Procedure

To ensure accurate comparisons of crack survey results, a standard procedure has been developed for the surveys. Surveys are conducted on days that are mostly sunny with temperatures of at least 60ºF (16°C). The bridge deck must be completely dry; therefore, if it has rained the night before or if rain is expected, the survey is not performed. Traffic control must be provided to shut down at least one lane of the bridge at a time.

Prior to the survey, a scaled drawing of the bridge deck is prepared at a scale of 1 inch = 10 ft (25 mm = 3.05 m). Two versions of this drawing should be printed: one with a 5 ft × 5 ft

(1.52 ×1.52 m) grid over the bridge and one without the grid. The version with the grid is placed under the version without, so the grid can be seen through the paper.

The survey crew consists of three to five people. The surveyors draw the 5 ft × 5 ft (1.52 ×1.52 m) grid on the deck using sidewalk chalk or lumber crayons to parallel the grid on the scale drawing. Cracks can be identified by bending at the waist but no closer to the deck. The goal is to obtain a consistent measure of cracking, rather than attempting to identify every crack. Cracks are also marked using either sidewalk chalk or lumber crayons. Each part of the bridge is surveyed by at least two individuals using this method. One person transfers the cracks to the scale drawing using a pencil.

After the survey is complete, the scale drawing is scanned into a computer. All lines that are not cracks, such as lines identifying bridge piers or deck boundaries, are erased immediately after the scanned images have been saved. Since the computer program only accounts for straight cracks, curved cracks are broken into straight line sections. This is done by removing single pixels from the curves. The scanned image may need to be enhanced to darken the pixels of the cracks. The scanned images are then converted to a data file that is analyzed using a program that counts the number of dark adjacent pixels to determine individual crack lengths, which are converted to crack density for the bridge deck (Lindquist et al. 2005). Crack densities are calculated for the entire deck as well as by span, placement, and for the end sections of the bridge. The complete procedure for performing crack surveys is described in Appendix B.

1.4.2 Results

The completed crack maps for each of the crack surveys are shown in the following sections in Figures 1.1-2.12. Because the bridges are in pairs, they are considered "twins" and can be used to provide comparisons.

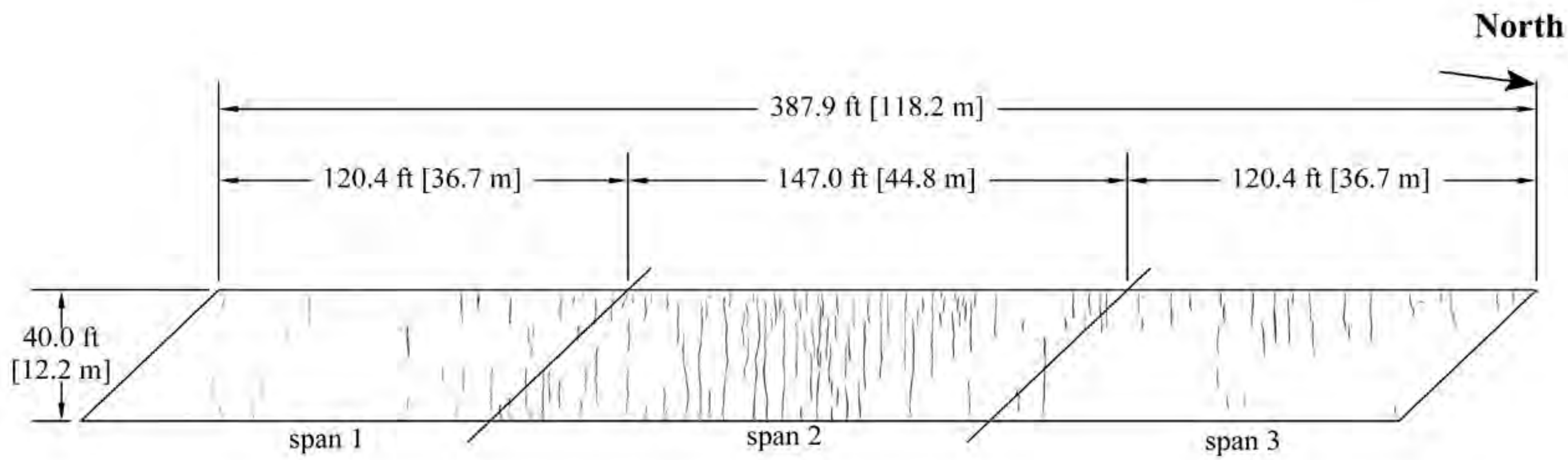
All of the US-59 bridge decks have reached an age of at least 42 months. Since the bridges in the LC-HPC study, as well as this study, range in age, a crack density at an age of 42 months is used for most comparisons. Linear interpolation is used to calculate the 42-month crack density for the bridges. Thirteen LC-HPC bridge decks have been surveyed, but only two of these are supported on prestressed girders. Ten control bridge decks in the LC-HPC study have been surveyed along with the LC-HPC decks, but only one is supported by prestressed girders. At 42 months, crack densities range from 0.008 to 0.324 m/m^2 for the LC-HPC bridge decks with steel girders, with an average of 0.160 m/m^2. Two of the LC-HPC decks are supported by prestressed girders, which have crack densities at 42 months of 0.055 and 0.373 m/m^2, with an average of 0.214 m/m^2. For the control decks on steel girders without a silica flume overlay, the crack densities at 42 months range from 0.127 to 0.782 m/m^2, with an average of 0.420 m/m^2; for the one control deck supported by prestressed girders, the crack density is 0.205 m/m^2 at 42 months. In 2005, Lindquist et al. (2005) also studied older monolithic and silica fume overlay decks supported by steel girders in Kansas. At 42 months, the crack densities for the monolithic decks ranged from 0.008 to 0.355 m/m^2, with an average of 0.220 m/m^2, and for the control decks on steel girders with silica fume overlay, the crack densities ranged from 0.286 to 1.013 m/m^2, with an average of 0.565 m/m^2.

1.4.3 US-59 1

This bridge deck is supported by steel girders. The concrete contains 540 lb/yd^3(320 kg/m^3) of cementitious material with 60% cement, 35% slag, and 5% silica fume, granite coarse aggregate, and has no overlay. The w/c ratio of 0.42 used for this deck is lower than the specified range of 0.44 to 0.45 for LC-HPC decks. The paste content was 23.99 percent. The deck had an average slump of 4 inches (100 mm), an average air content of 6.5 percent, and a compressive strength of 5090 psi (35.1 MPa). The average slump is slightly higher than the specified LC-HPC maximum of 3½ inches (90 mm), while the air content and compressive strength are within the LC-HPC specifications. Ames was the contractor. The average concrete temperature was 15.5°F (8.6°C) higher than the average air temperature on the day of placement.

Three crack surveys were performed, at 22, 31, and 45 months. The crack density at 22 months was 0.280 m/m^2 (Figure 1.1). At 31 months, the crack density increased to 0.385 m/m^2 (Figure 1.2) and at 45 months, the crack density was 0.403 m/m^2 (Figure 1.3). The highest crack density was observed in middle of the center span on each survey. Small longitudinal cracks were present on both the abutments except in survey 1.

The crack density for the US-59 1 bridge deck at 42 months is 0.399 m/m^2, which is higher than highest crack density for LC-HPC decks supported by steel girders at the same age, which is 0.324 m/m^2. This could be due to the differences in curing methods, the lower w/c ratio, and the larger difference between concrete and air temperatures compared to those associated with the LC-HPC decks.

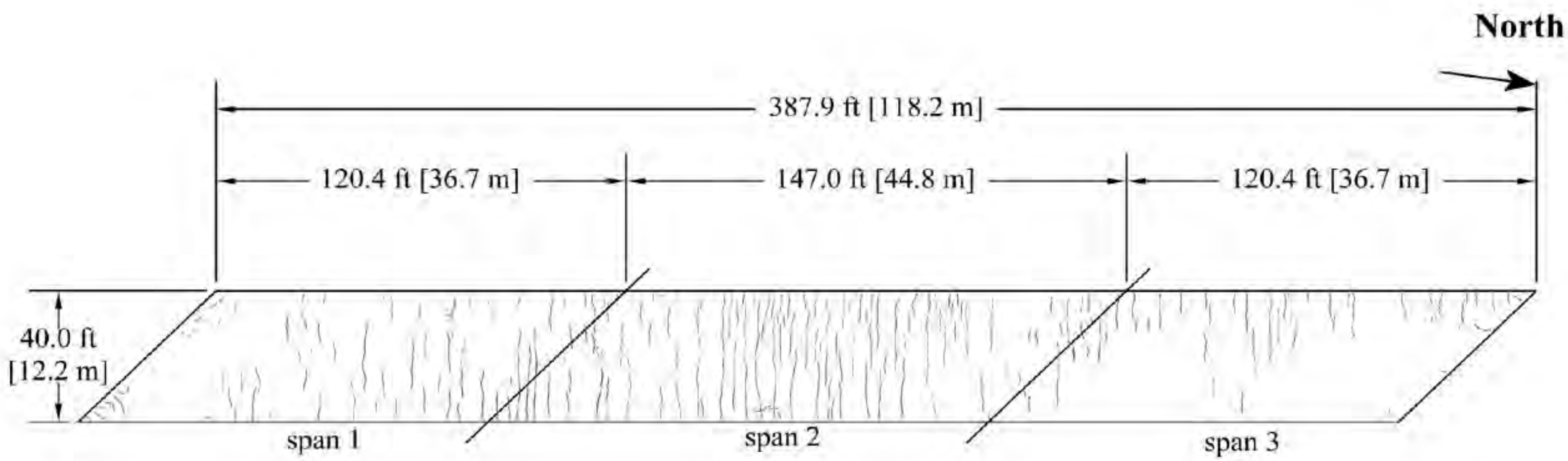


Bridge Number: 59-30-19.92
Bridge Location: SB US-59 over Sand Creek Rd.
Construction Date: 11-25-2008
Crack Survey Date: 8-25-2010

Bridge Length: 387.9 ft (118.2 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -45.37 °
Number of Spans: 3
Span 1: 120.4 ft (36.7 m)
Span 2: 147.0 ft (44.8 m)
Span 3: 120.4 ft (36.7 m)
Number of Placements: 1

Bridge Age: 22 months
Crack Density: 0.280 m/m²
Span 1: 0.110 m/m²
Span 2: 0.530 m/m²
Span 3: 0.150 m/m²

FIGURE 1.1
US-59 1 (Survey 1)

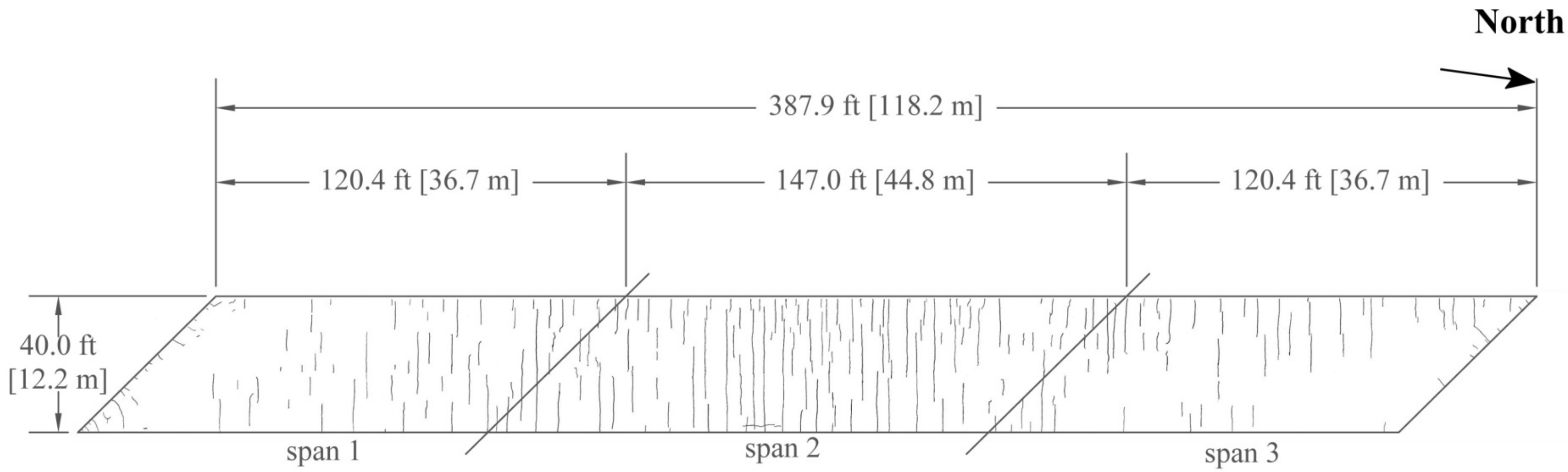


Bridge Number: 59-30-19.92
Bridge Location: SB US-59 over Sand Creek Rd.
Construction Date: 11-25-2008
Crack Survey Date: 6-20-2011

Bridge Length: 387.9 ft (118.2 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -45.37 °
Number of Spans: 3
Span 1: 120.4 ft (36.7 m)
Span 2: 147.0 ft (44.8 m)
Span 3: 120.4 ft (36.7 m)
Number of Placements: 1

Bridge Age: 31 months
Crack Density: 0.385 m/m^2
Span 1: 0.299 m/m^2
Span 2: 0.595 m/m^2
Span 3: 0.214 m/m^2

FIGURE 1.2
US-59 1 (Survey 2)



Bridge Number: 59-30-19.92
Bridge Location: SB US-59 over Sand Creek Rd.
Construction Date: 11-25-2008
Crack Survey Date: 8-17-2012

Bridge Length: 387.9 ft (118.2 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -45.37 ^{o}
Number of Spans: 3
Span 1: 120.4 ft (36.7 m)
Span 2: 147.0 ft (44.8 m)
Span 3: 120.4 ft (36.7 m)
Number of Placements: 1

Bridge Age: 45 months
Crack Density: 0.403 m/m^2
Span 1: 0.296 m/m^2
Span 2: 0.592 m/m^2
Span 3: 0.274 m/m^2

FIGURE 1.3
US-59 1 (Survey 3)

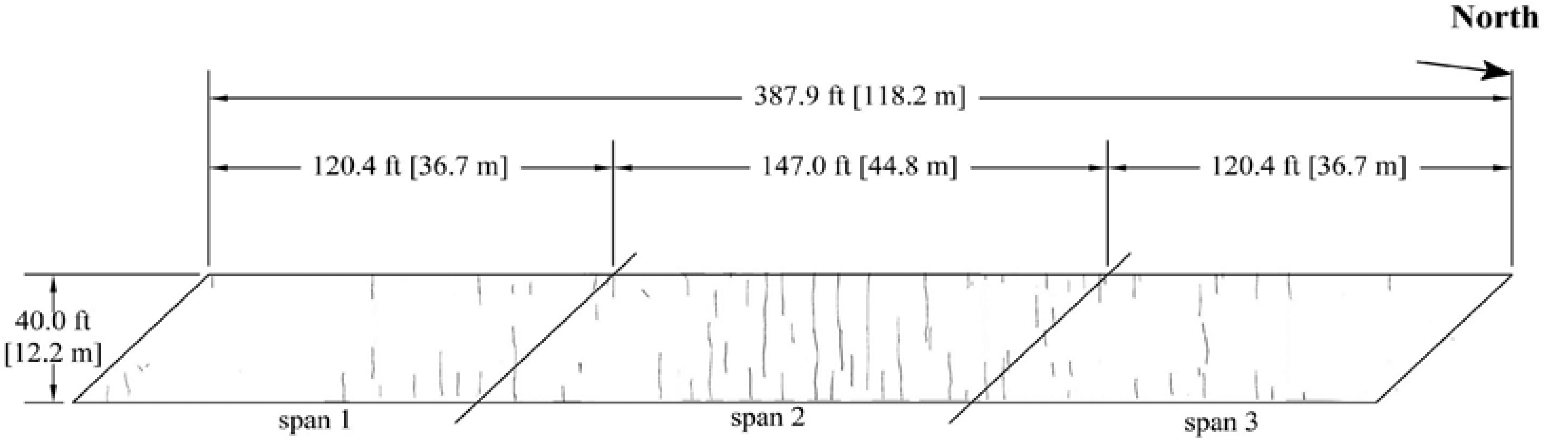
1.4.4 US-59 2

US-59 2 is the twin bridge to US-59 1 and is also supported by steel girders. It was constructed by Ames, has no overlay, and contains the same concrete mixture as US-59 1. The concrete in the deck had an average slump of 3½ inches (90 mm) and an average air content of 6.75 percent. The w/c ratio was 0.42, which is lower than the LC-HPC desired range of 0.44 to 0.45, and the compressive strength of 6390 psi (44.1 MPa) is higher than the LC-HPC specified maximum of 5500 psi (37.9 MPa). The air temperature during concrete placement ranged from 27°F to 51°F (-2.8°C to 11°C) and the average air temperature was 39°F (4°C). The average concrete temperature was 26.3°F (14.5°C) higher than the average air temperature, which is higher than allowable temperature difference (25°F, 14°C) in cold weather placing in accordance with LC-HPC specifications.

Three crack surveys were performed, at 22, 32 and 46 months. The crack density at 22 months was 0.140 m/m^2 (Figure 1.4). At 32 months, the crack density increased to 0.217 m/m^2 (Figure 1.5) and at 46 months, to 0.306 m/m^2 (Figure 1.6). The highest crack density was observed in middle of the center span, with significant growth each year. Small longitudinal cracks at both abutments were most apparent during survey 3.

For US-59 2 bridge, the crack density is 0.281 m/m^2 at 42 month, which is higher than the average of 0.160 m/m^2 for LC-HPC bridges with steel girders. This could be attributed to the low w/c ratio and the high compressive strength. The crack density does fall within the range of crack densities at 42 months for LC-HPC bridge decks on steel girders. It is lower than the average crack density for the old monolithic decks.

Also, the crack density for this deck is lower than for its twin. This could be attributed to the lower average slump, 3½ (90 mm), compared to 4 inches (100 mm) for US-59 1, since a higher slump increases the potential for settlement cracking over the reinforcing bars.

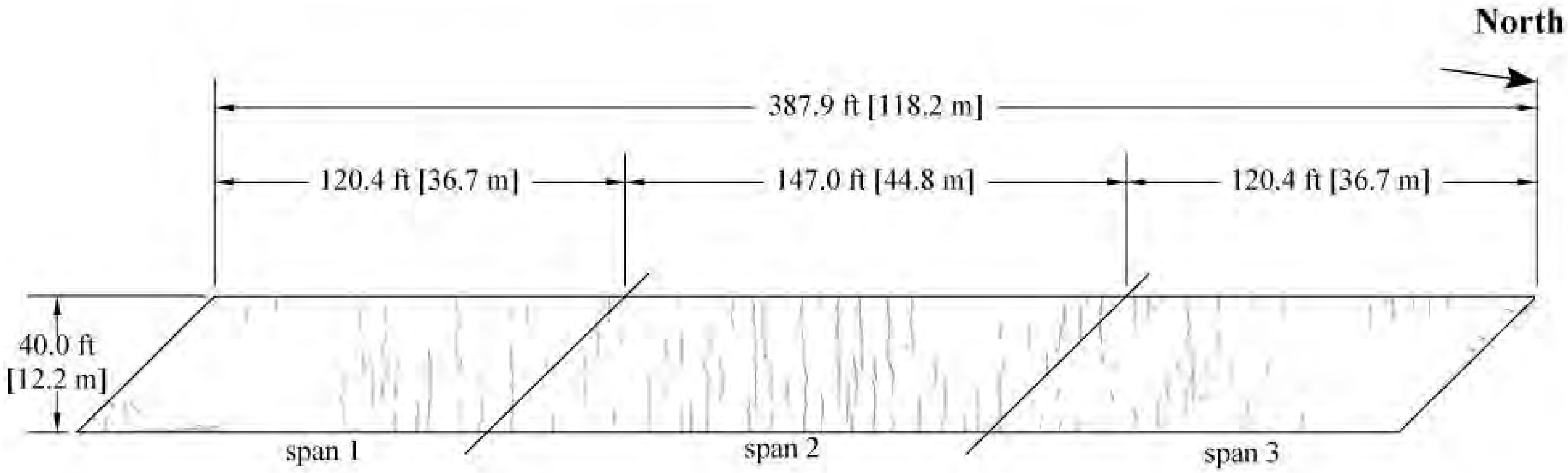


Bridge Number: 59-30-19.91
Bridge Location: NB US-59 over Sand Creek Rd.
Construction Date: 9-19-2008
Crack Survey Date: 9-3-2010

Bridge Length: 387.9 ft (118.2 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -45.37 °
Number of Spans: 3
Span 1: 120.4 ft (36.7 m)
Span 2: 147.0 ft (44.8 m)
Span 3: 120.4 ft (36.7 m)
Number of Placements: 1

Bridge Age: 22 months
Crack Density: 0.140 m/m²
Span 1: 0.082 m/m²
Span 2: 0.250 m/m²
Span 3: 0.094 m/m²

FIGURE 1.4
US-59 2 (Survey 1)

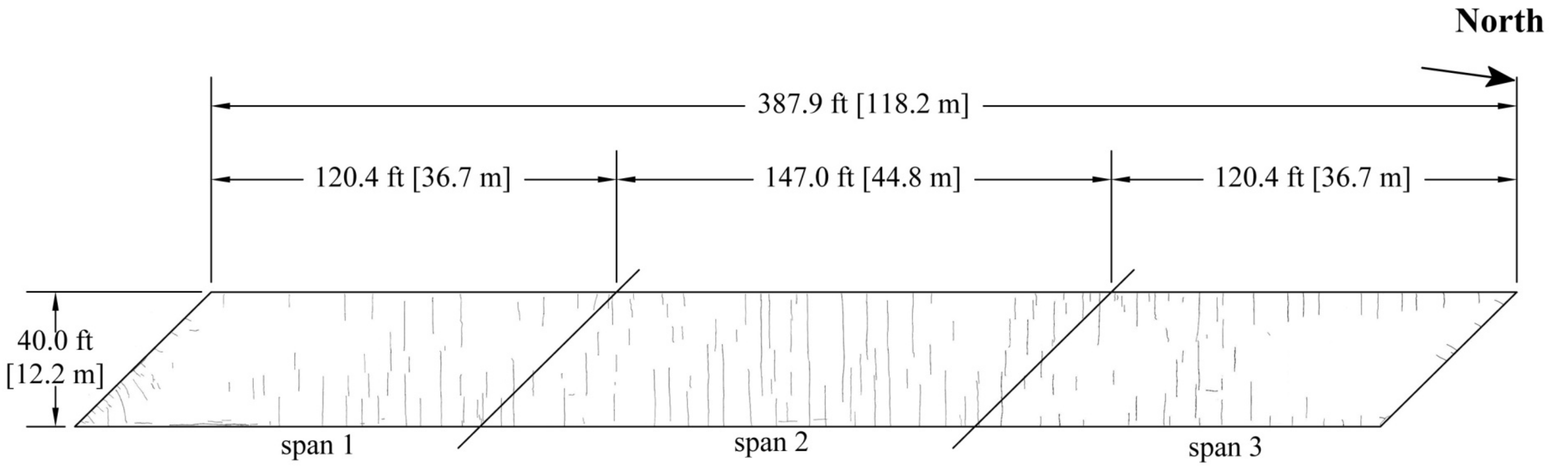


Bridge Number: 59-30-19.91
Bridge Location: NB US-59 over Sand Creek Rd.
Construction Date: 9-19-2008
Crack Survey Date: 6-20-2011

Bridge Length: 387.9 ft (118.2 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -45.37°
Number of Spans: 3
Span 1: 120.4 ft (36.7 m)
Span 2: 147.0 ft (44.8 m)
Span 3: 120.4 ft (36.7 m)
Number of Placements: 1

Bridge Age: 32 months
Crack Density: 0.217 m/m^2
Span 1: 0.181 m/m^2
Span 2: 0.303 m/m^2
Span 3: 0.143 m/m^2

FIGURE 1.5
US-59 2 (Survey 2)



Bridge Number: 59-30-19.91
Bridge Location: NB US-59 over Sand Creek Rd.
Construction Date: 9-19-2008
Crack Survey Date: 8-1-2012

Bridge Length: 387.9 ft (118.2 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -45.37 °
Number of Spans: 3
Span 1: 120.4 ft (36.7 m)
Span 2: 147.0 ft (44.8 m)
Span 3: 120.4 ft (36.7 m)
Number of Placements: 1

Bridge Age: 46 months
Crack Density: 0.306 m/m^2
Span 1: 0.285 m/m^2
Span 2: 0.398 m/m^2
Span 3: 0.215 m/m^2

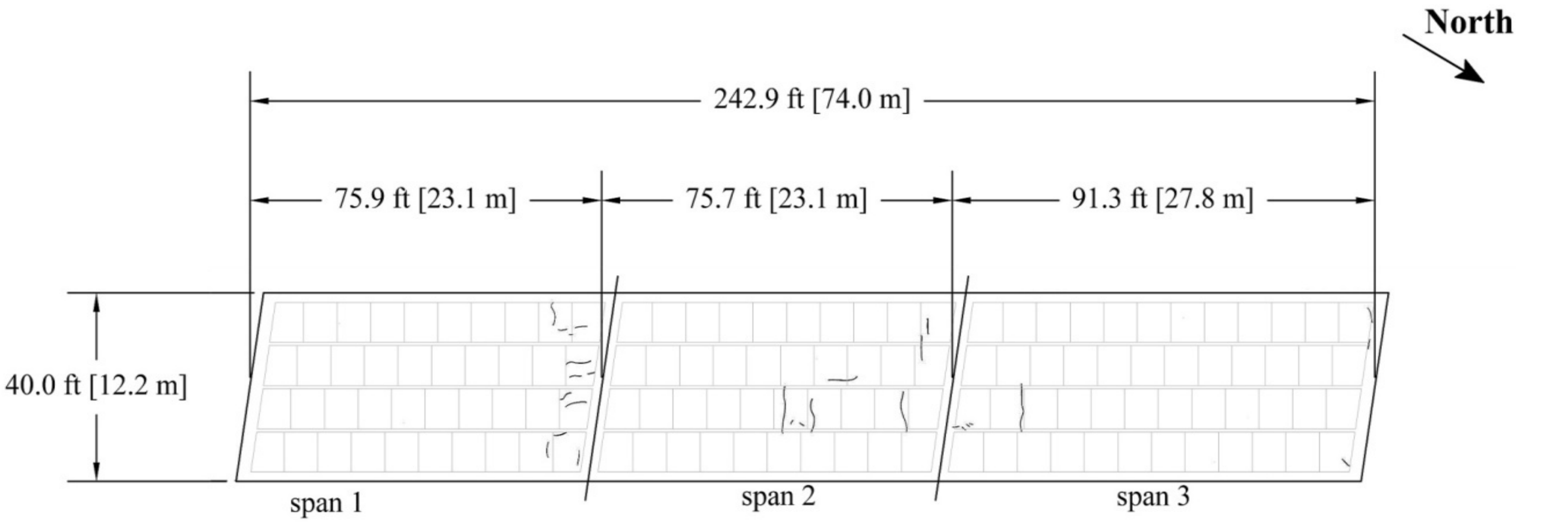
FIGURE 1.6
US-59 2 (Survey 3)

1.4.5 US-59 3

The deck on bridge US-59 3 has precast deck panels supported by prestressed concrete girders. Ames was the contractor. The concrete contains 545 lb/yd^3 (323 kg/m^3) of cementitious material with 65% cement and 35% slag and granite coarse aggregate. The paste content was 24.77 percent. The concrete in the deck had an average slump of 4 inches (100 mm), an average air content of 7.25 percent, and a compressive strength of 4260 psi (29.4 MPa). The w/c ratio was 0.45. The air, w/c ratio, and compressive strength are within the desired range for an LC-HPC deck, while the average slump is slightly higher than the specified maximum of 3½ inches (90 mm) for LC- HPC decks. The average concrete temperature was 18.9°F (10.9°C) higher than the average air temperature on the day of placement.

The deck has been surveyed three times, at 23, 32 and 46 months. The crack density at 23 months was 0.035 m/m^2 (Figure 1.7). At 32 months, the crack density slightly increased to 0.051 m/m^2 (Figure 1.8), and at 45.6 months, to 0.070 m/m^2 (Figure 1.9). Much of the cracking is located over the piers and in the middle span of the deck. The cracks are oriented longitudinally over the pier and transversely in the middle span of the bridge. Transverse cracks appear to be aligned along the joints of the deck panels, as shown on the figures. Cracks seem to be slightly shifted from the joint of the deck panels on the crack maps because the crack survey procedures are not designed to exactly identify crack locations. Cracking seems to be minimal at the joints for the most of the deck panels, but it does appear to be greater in survey 3.

The crack density at 42 months is 0.065 m/m^2, which is lower than average crack density for the LC-HPC decks supported by prestressed girders (0.214 m/m^2). The crack density for this deck is much lower than LC-HPC decks supported by steel girders (0.160 m/m^2).

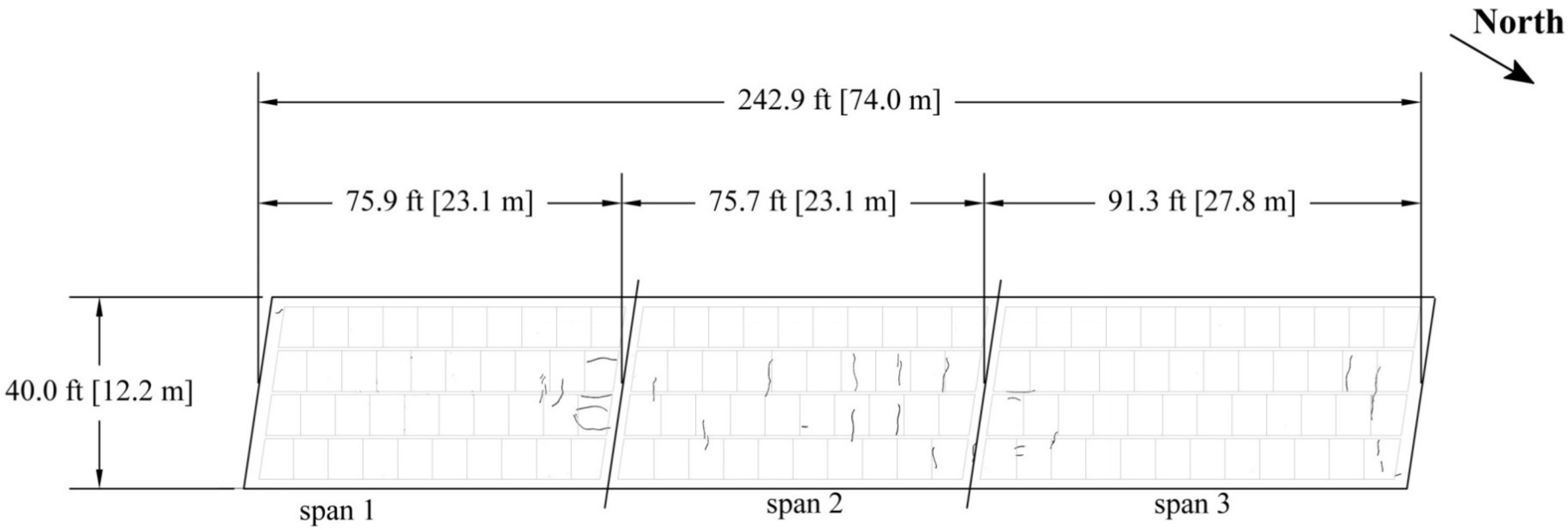


Bridge Number: 59-30-20.05
Bridge Location: SB US-59 over BNSF R.R.
Construction Date: 9-30-08
Crack Survey Date: 8-25-10

Bridge Length: 242.9 ft (74.0m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -8.3°
Number of Spans: 3
Span 1: 75.9 ft (23.1 m)
Span 2: 75.7 ft (23.1 m)
Span 3: 91.3 ft (27.8 m)
Number of Placements: 1

Bridge Age: 23 months
Crack Density: 0.035 m/m^2
Span 1: 0.037 m/m^2
Span 2: 0.047 m/m^2
Span 3: 0.020 m/m^2

FIGURE 1.7
US-59 3 (Survey 1)

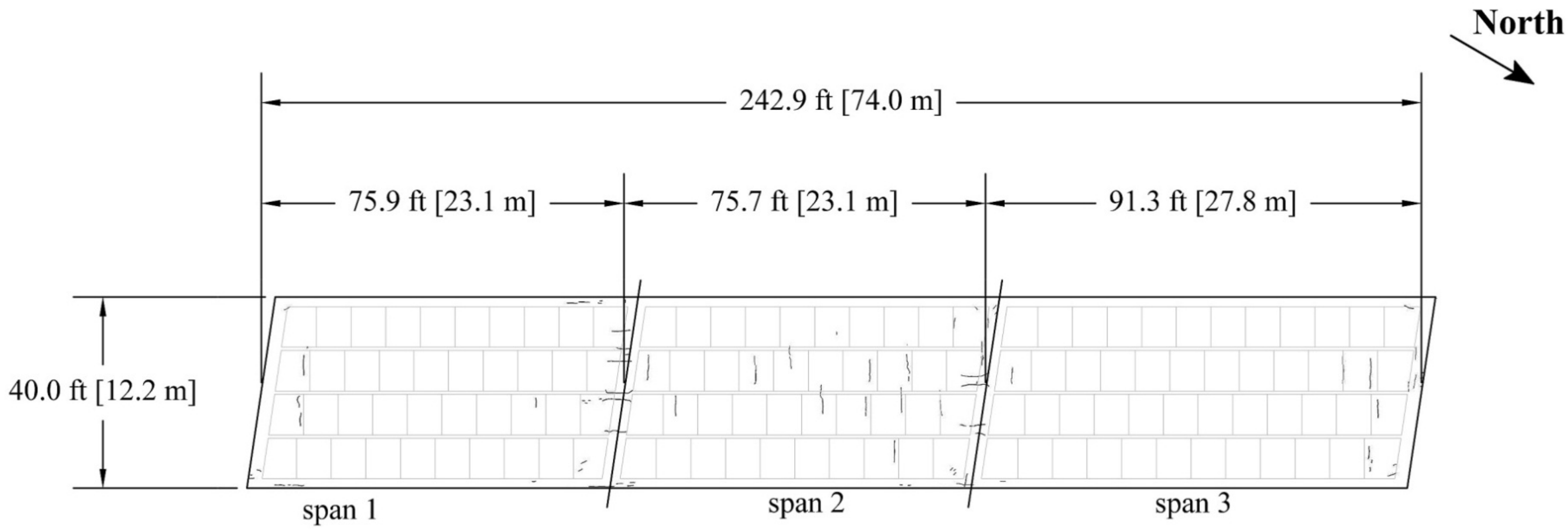


Bridge Number: 59-30-20.05
Bridge Location: SB US-59 over BNSF R.R.
Construction Date: 9-30-08
Crack Survey Date: 6-20-11

Bridge Length: 242.9 ft (74.0m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -8.3°
Number of Spans: 3
Span 1: 75.9 ft (23.1 m)
Span 2: 75.7 ft (23.1 m)
Span 3: 91.3 ft (27.8 m)
Number of Placements: 1

Bridge Age: 32 months
Crack Density: 0.051 m/m^2
Span 1: 0.070 m/m^2
Span 2: 0.066 m/m^2
Span 3: 0.021m/m ^{2}

FIGURE 1.8
US-59 3 (Survey 3)



Bridge Number: 59-30-20.05
Bridge Location: SB US-59 over BNSF R.R.
Construction Date: 9-30-08
Crack Survey Date: 7-18-12

Bridge Length: 242.9 ft (74.0m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -8.3°
Number of Spans: 3
Span 1: 75.9 ft (23.1 m)
Span 2: 75.7 ft (23.1 m)
Span 3: 91.3 ft (27.8 m)
Number of Placements: 1

Bridge Age: 46 months
Crack Density: 0.070 m/m²
Span 1: 0.063 m/m²
Span 2: 0.103 m/m²
Span 3: 0.036 m/m²

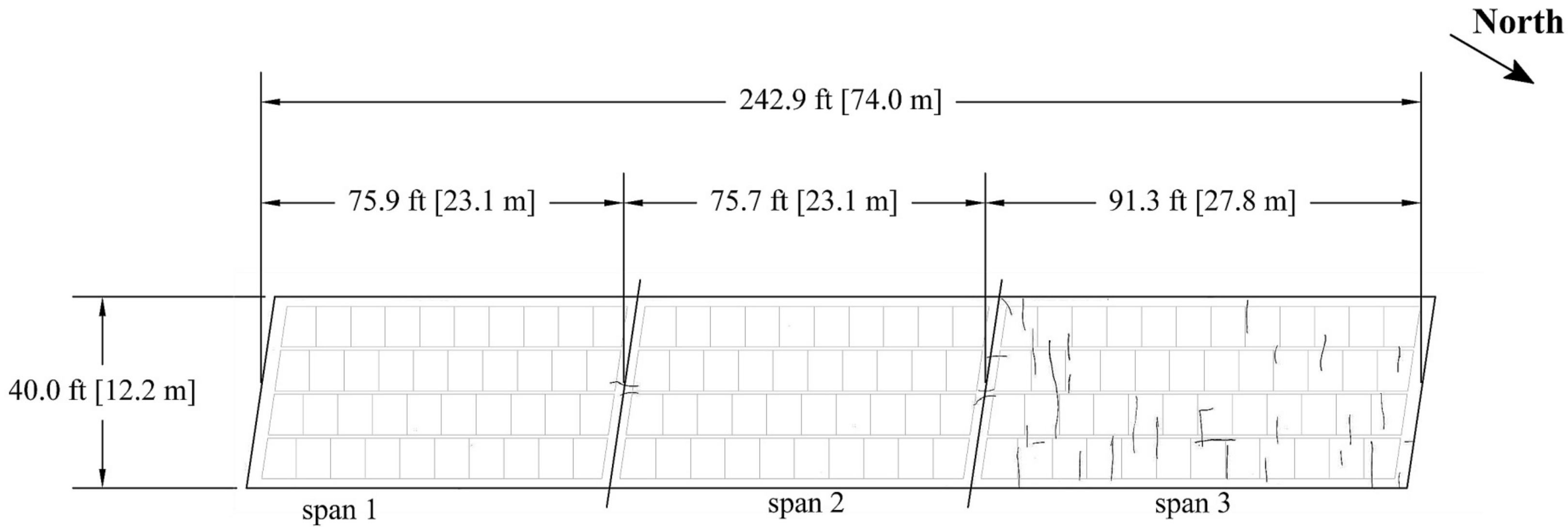
FIGURE 1.9
US-59 3 (Survey 2)

1.4.6 US-59 4

US-59 4 was also constructed by Ames, has deck panels, is supported by prestressed girders, and has the same concrete mixture properties as US-59 3. The average slump of the plastic concrete for this deck was 4 inches (100 mm), the average air content was 6.75 percent, and the compressive strength was 5000 psi (34.5 MPa). The w/c ratio was 0.45. The air, w/c ratio, and compressive strength are in the desired range for LC-HPC deck, while the average slump is slightly higher than the LC- HPC maximum of 3½ inches (90 mm). The average concrete temperature was 10.7°F (6°C) higher than the average air.

US-59 4 and US-59 3 are twin bridges. US-59 4 has been surveyed three times, at 23, 33, and 46 months. At 23 months, the crack density was 0.067 m/m^2 (Figure 1.10). At 33 months, the crack density was 0.056 m/m^2 (Figure 1.11), and at 46 months, the crack density was 0.082 m/m^2 (Figure 1.12). Most of the cracks in the bridge deck are short. They are oriented in both transverse and longitudinal directions. The crack density decreased slightly during survey 2; the value of the decrease can be considered to be within the variation expected between surveys. On survey 3, an increase in crack density was observed. The majority of the cracks are in the north span of the deck. The other two spans exhibit minimal cracking, primarily over the piers. Many of the transverse cracks on the deck appear to be aligned with the joints of the deck panels.

The crack density for US-59 4 bridge at 42 months is 0.074 m/m^2, which is higher than crack density of its twin bridge (0.065 m/m^2). The crack density is lower than the average crack density for the LC-HPC decks supported by prestressed girders and is closer to the crack density of the lowest cracking LC-HPC bridge deck. The crack density for this deck is also significantly lower than the averages for both the old monolithic and LC-HPC decks supported by steel girders.

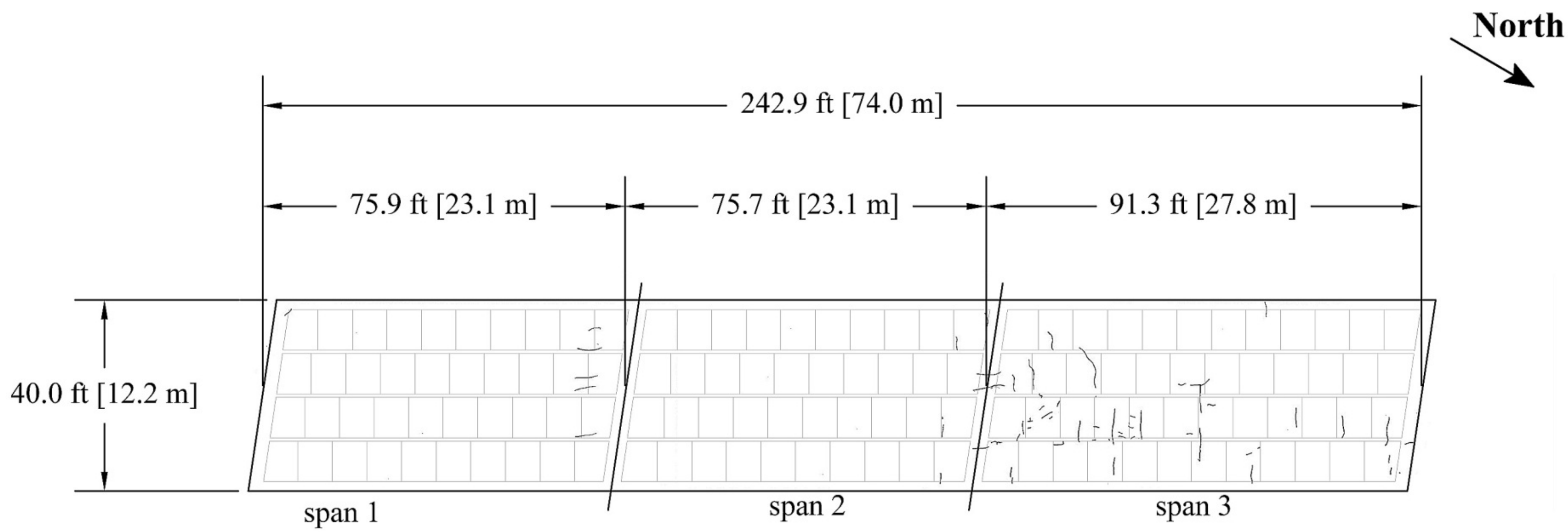


Bridge Number: 59-30-20.04
Bridge Location: NB US-59 over BNSF R.R.
Construction Date: 9-19-08
Crack Survey Date: 9-3-10

Bridge Length: 242.9 ft (74.0m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -8.3°
Number of Spans: 3
Span 1: 75.9 ft (23.1 m)
Span 2: 75.7 ft (23.1 m)
Span 3: 91.3 ft (27.8 m)
Number of Placements: 1

Bridge Age: 23 months
Crack Density: 0.067 m/m^2
Span 1: 0.004 m/m^2
Span 2: 0.012 m/m^2
Span 3: 0.160 m/m^2

FIGURE 1.10
US-59 4 (Survey 1)

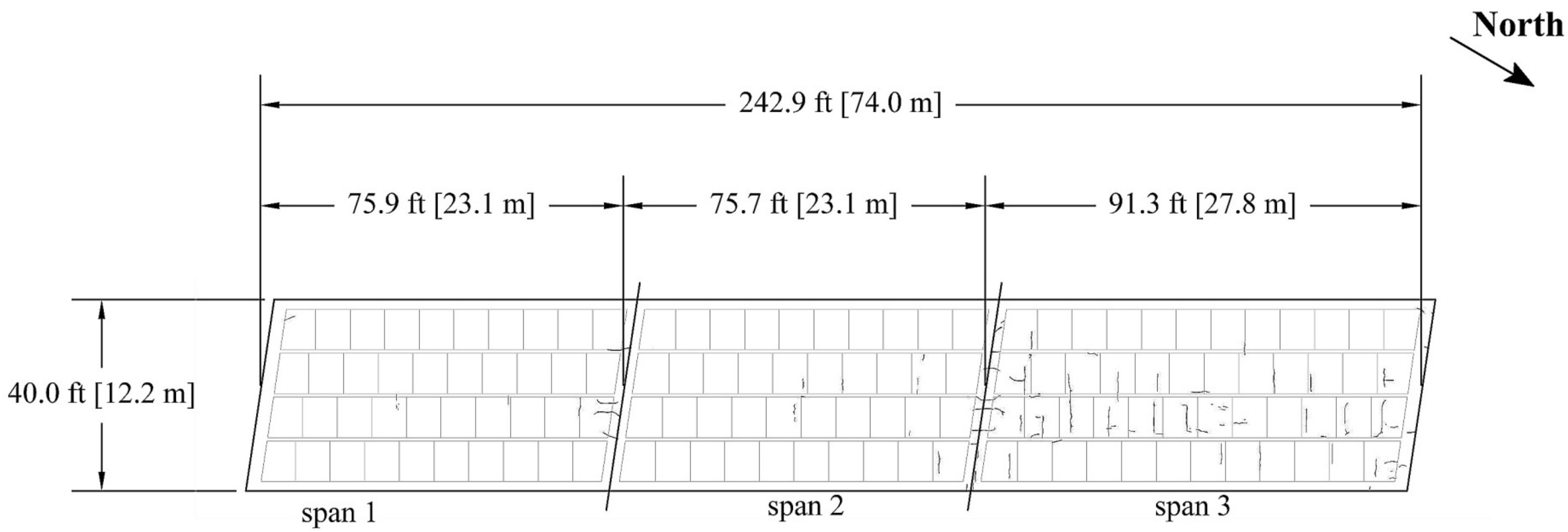


Bridge Number: 59-30-20.04
Bridge Location: NB US-59 over BNSF R.R.
Construction Date: 9-19-08
Crack Survey Date: 6-20-11

Bridge Length: 242.9 ft (74.0m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -8.3°
Number of Spans: 3
Span 1: 75.9 ft (23.1 m)
Span 2: 75.7 ft (23.1 m)
Span 3: 91.3 ft (27.8 m)
Number of Placements: 1

Bridge Age: 33 months
Crack Density: 0.056 m/m²
Span 1: 0.018 m/m²
Span 2: 0.011 m/m²
Span 3: 0.124 m/m²

FIGURE 1.11
US-59 4 (Survey 2)



Bridge Number: 59-30-20.04
Bridge Location: NB US-59 over BNSF R.R.
Construction Date: 9-19-08
Crack Survey Date: 7-18-12

Bridge Length: 242.9 ft (74.0m)
Bridge Width: 40.0 ft(12.2 m)
Skew: -8.3°
Number of Spans: 3
Span 1: 75.9 ft (23.1 m)
Span 2: 75.7 ft (23.1 m)
Span 3: 91.3 ft (27.8 m)
Number of Placements: 1

Bridge Age: 46 months
Crack Density: 0.082 m/m^2
Span 1: 0.029 m/m^2
Span 2: 0.029 m/m^2
Span 3: 0.159 m/m^2

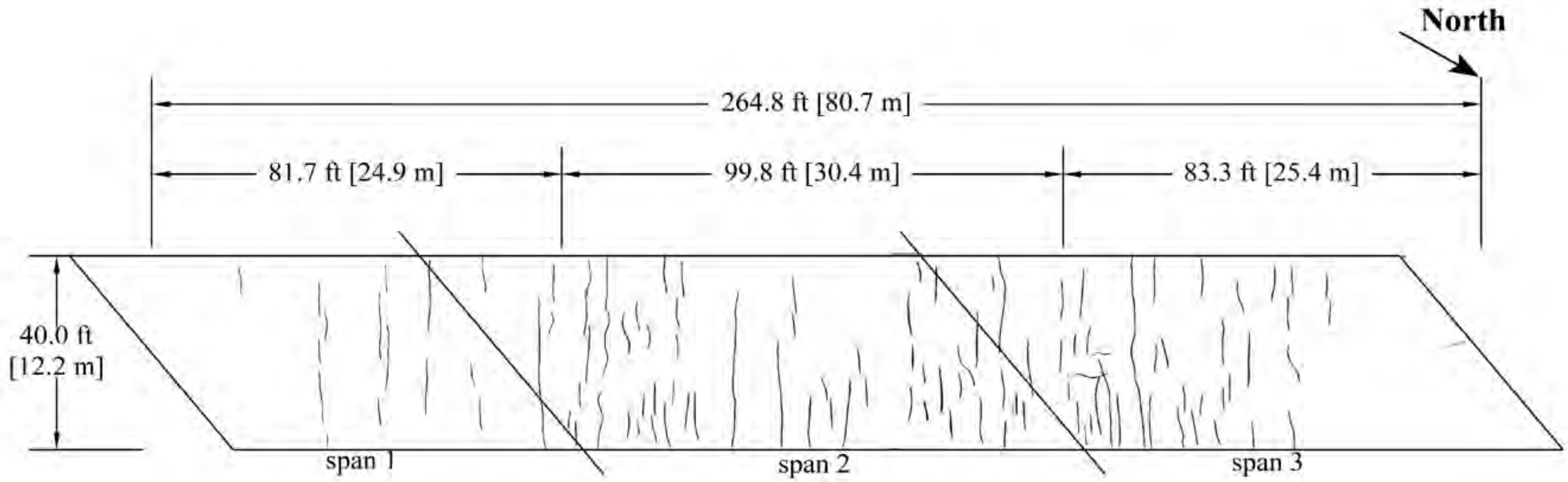
FIGURE 1.12
US-59 4 (Survey 3)

1.4.7 US-59 5

US-59 5 is supported by steel girders and was constructed by Ames. The concrete in the deck contains 630 lb/yd^3 (374 kg/m^3) of cement, limestone coarse aggregate, and has a silica fume overlay with 1.55 inches long synthetic fibers in the overlay (Grace 90/40 Strux). The average slump, paste content and air content of the plastic concrete for the subdeck were, respectively, 5 inches (130 mm), 27.95 percent, and 6.75 percent. The w/c ratio was 0.44, and the compressive strength of the subdeck was 5010 psi (34.5 MPa). The paste content, along with that of US-59 6 and 11, was highest among the US-59 decks or subdecks. The average slump was much higher than the maximum of 3½ inches (90 mm) specified for LC-HPC, while the air content and w/c ratio fall within the desired ranges for LC-HPC. The difference between the average concrete and air temperatures was 10°F (5.3°C). The average slump and compressive strength for the silica fume overlay were 4.5 inches (114 mm) and 6450 psi (44.5 MPa).

Three crack surveys have been performed on this bridge, at 28, 38, and 46 months. At 28 months, the crack density was 0.270 m/m^2 (Figure 1.13). At 38 months, the crack density was 0.320 m/m^2 (Figure 1.14). At 51 months, the crack density increased significantly to 0.465 m/m^2 (Figure 1.15). The cracks on this bridge are evenly distributed over most of the deck, excluding the ends. A longitudinal crack through almost the entire middle span was observed on both second and third surveys.

At 42 months, the crack density on US-59 5 was 0.393 m/m^2 which is lower than the average of 0.565 m/m^2 for control decks with silica fume in the LC-HPC study and the average of 0.420 m/m^2 for the control decks without silica fume overlays supported by steel girders.

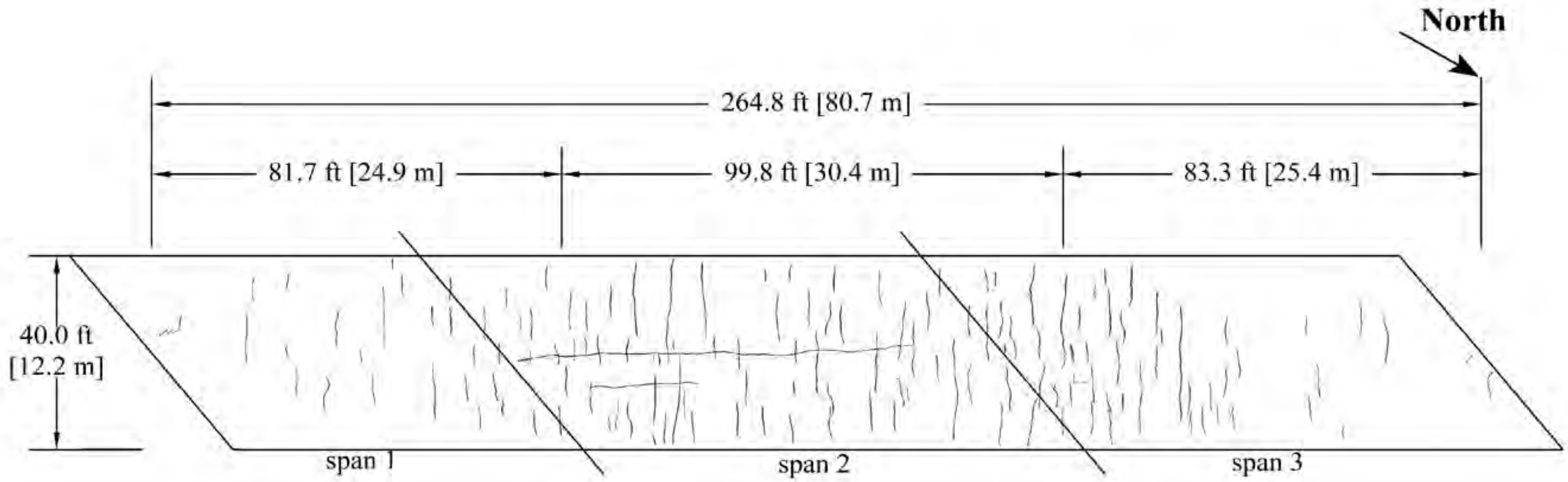


Bridge Number: 59-30-21.84
Bridge Location: SB US-59 over Midland R.R.
Construction Date: 5-14-08
Crack Survey Date: 9-21-10

Bridge Length: 264.8 ft (80.7 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: 39.17°
Number of Spans: 3
Span 1: 81.7 ft (24.9 m)
Span 2: 99.8 ft (30.4 m)
Span 3: 83.3 ft (25.4 m)
Number of Placements: 1

Bridge Age: 28 months
Crack Density: 0.270 m/m^2
Span 1: 0.160 m/m^2
Span 2: 0.380 m/m^2
Span 3: 0.250 m/m^2

FIGURE 1.13
US-59 5 (Survey 1)

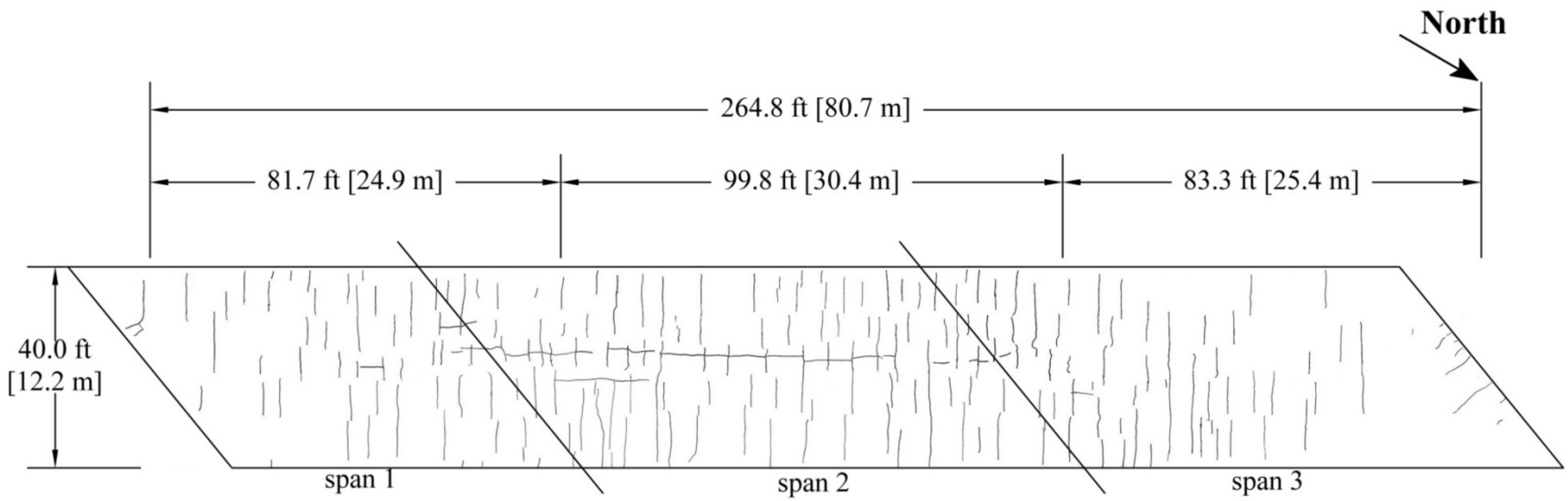


Bridge Number: 59-30-21.84
Bridge Location: SB US-59 over
Midland R.R.
Construction Date: 5-14-08
Crack Survey Date: 7-15-11

Bridge Length: 264.8 ft (80.7 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: 39.17°
Number of Spans: 3
Span 1: 81.7 ft (24.9 m)
Span 2: 99.8 ft (30.4 m)
Span 3: 83.3 ft (25.4 m)
Number of Placements: 1

Bridge Age: 38 months
Crack Density: 0.320 m/m^2
Span 1: 0.195m/m^2
Span 2: 0.458 m/m^2
Span 3: 0.246 m/m^2

FIGURE 1.14
US-59 5 (Survey 2)



Bridge Number: 59-30-21.84
Bridge Location: SB US-59 over Midland R.R.
Construction Date: 5-14-08
Crack Survey Date: 8-9-12

Bridge Length: 264.8 ft (80.7 m)
Bridge Width: 40.0 ft(12.2 m)
Skew: 39.17°
Number of Spans: 3
Span 1: 81.7 ft (24.9 m)
Span 2: 99.8 ft (30.4 m)
Span 3: 83.3 ft (25.4 m)
Number of Placements: 1

Bridge Age: 46 months
Crack Density: 0.465 m/m^2
Span 1: 0.418 m/m^2
Span 2: 0.580 m/m^2
Span 3: 0.324 m/m^2

FIGURE 1.15
US-59 5 (Survey 3)

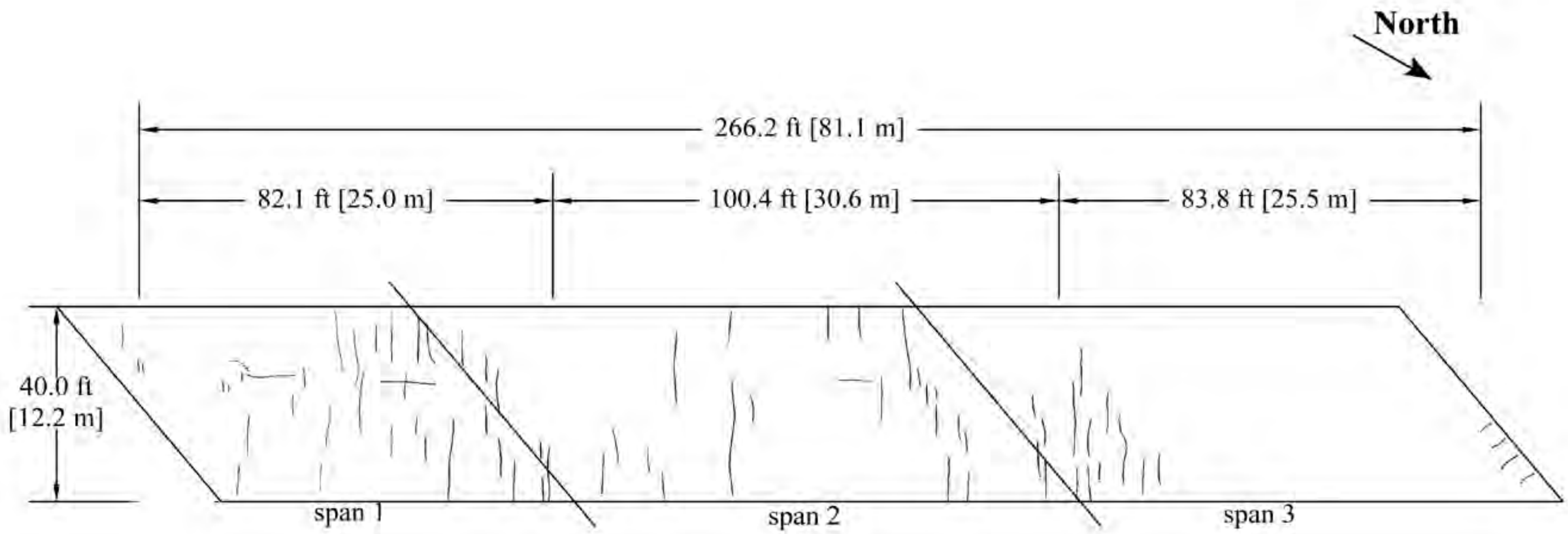
1.4.8 US-59 6

US-59 6 is the twin bridge to US-59 5. As with US-59 5, this bridge deck is supported by steel girders and was constructed by Ames. The concrete contains 630 lb/yd^3 (374 kg/m^3) of cement, limestone coarse aggregate, and has a silica fume overlay without fibers in the overlay. The plastic concrete for the subdeck for US-59 6 had an average slump of 4½ inches (115 mm), paste content of 27.95 percent, and an average air content of 6.25 percent. The w/c ratio is 0.44 and the compressive strength of the subdeck was 4850 psi (33.4 MPa). The average slump and air content are both out of the desired ranges for LC-HPC decks, but the w/c ratio and the compressive strength are within the LC-HPC specifications. The paste content was high. The difference between the average concrete and air temperatures was 3°F (1.9°C). The silica fume overlay had the average slump and compressive strength of ¾ inch (19 mm) and 7480 psi (51.6 MPa) respectively.

The deck was surveyed at 29, 39 months and 51 months. The crack density at 29 months was 0.160 m/m^2 (Figure 1.16). At 39 months, the crack density was 0.198 m/m^2 (Figure 1.17). At 51 months, the crack density increased significantly to 0.273 m/m^2 (Figure 1.18). The highest crack density is in span 3 followed by span 2. Most of the cracks are oriented in the transverse direction in the first two surveys; most cracks were in the vicinity of the piers, but midspan cracking increased markedly in the third survey.

The crack density at 42 months (0.219 m/m^2) is much lower than both the average density of 0.565 m/m^2 for the control decks with silica fume and the average density of 0.420 m/m^2 for control decks without silica fume overlays supported by steel girders. The crack density is close to average for LC-HPC decks on steel girders at 42 months.

The crack density of bridge US-59 6 is significantly less than that of bridge US-59 5, which contains fibers in the overlay. On survey 3, the crack density for US-59 5 was 70 percent more than bridge US-59 6. US-59 5 had a subdeck average slump and compressive strength that were slightly higher than that for US-59 6. Similarly, average slump on overlay of the US-59 5 was higher than that on US-59 6, both of which could have contributed to the higher crack density.

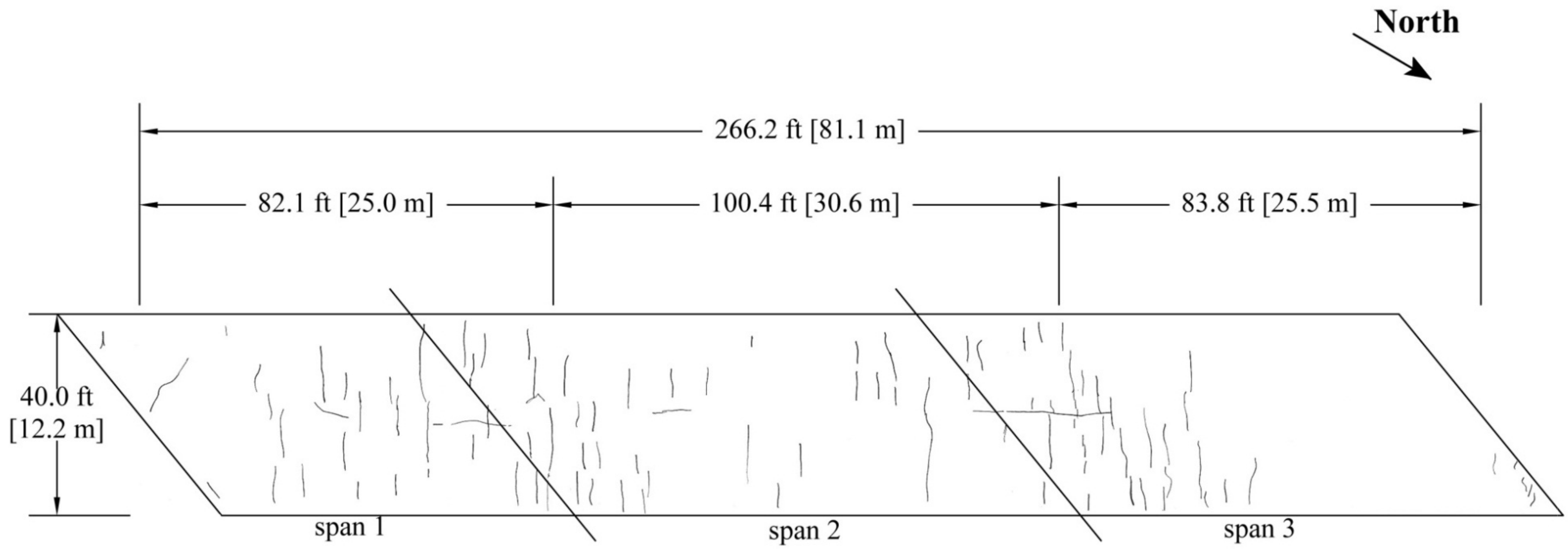


Bridge Number: 59-30-21.85
Bridge Location: NB US-59 over Midland R.R.
Construction Date: 4-30-2008
Crack Survey Date: 9-21-2010

Bridge Length: 266.22 ft (81.1 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 39.17°
Number of Spans: 3
Span 1: 82.11 ft (25.0m)
Span 2: 100.36 ft (30.6 m)
Span 3: 83.75 ft (25.5 m)
Number of Placements: 1

Bridge Age: 29 months
Crack Density: 0.160 m/m^2
Span 1: 0.240 m/m^2
Span 2: 0.170 m/m^2
Span 3: 0.060 m/m^2

FIGURE 1.16
US-59 6 (Survey 1)

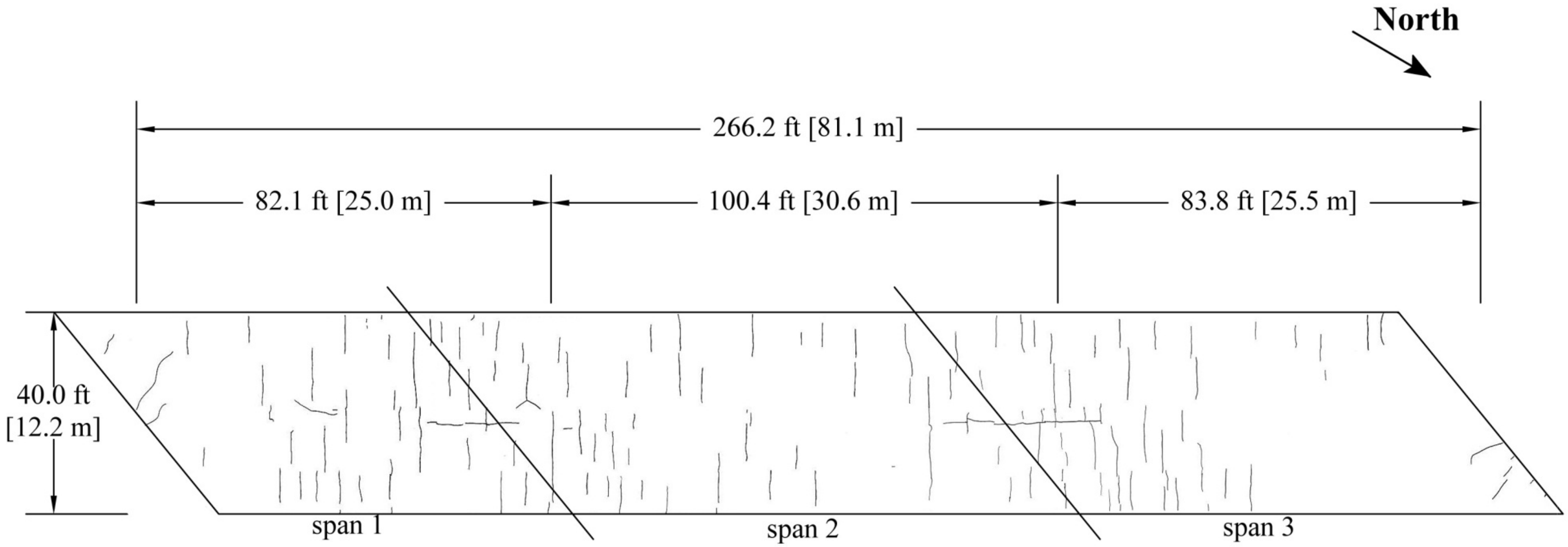


Bridge Number: 59-30-21.85
Bridge Location: NB US-59 over Midland R.R.
Construction Date: 4-30-2008
Crack Survey Date: 7-15-2011

Bridge Length: 266.22 ft (81.1 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 39.17°
Number of Spans: 3
Span 1: 82.11 ft (25.0m)
Span 2: 100.36 ft (30.6 m)
Span 3: 83.75 ft (25.5 m)
Number of Placements: 1

Bridge Age: 39 months
Crack Density: 0.198 m/m^2
Span 1: 0.255 m/m^2
Span 2: 0.182 m/m^2
Span 3: 0.163 m/m^2

FIGURE 1.17
US-59 6 (Survey 2)



Bridge Number: 59-30-21.85
Bridge Location: NB US-59 over Midland R.R.
Construction Date: 4-30-2008
Crack Survey Date: 8-9-2012

Bridge Length: 266.22 ft (81.1 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 39.17°
Number of Spans: 3
Span 1: 82.11 ft (25.0m)
Span 2: 100.36 ft (30.6 m)
Span 3: 83.75 ft (25.5 m)
Number of Placements: 1

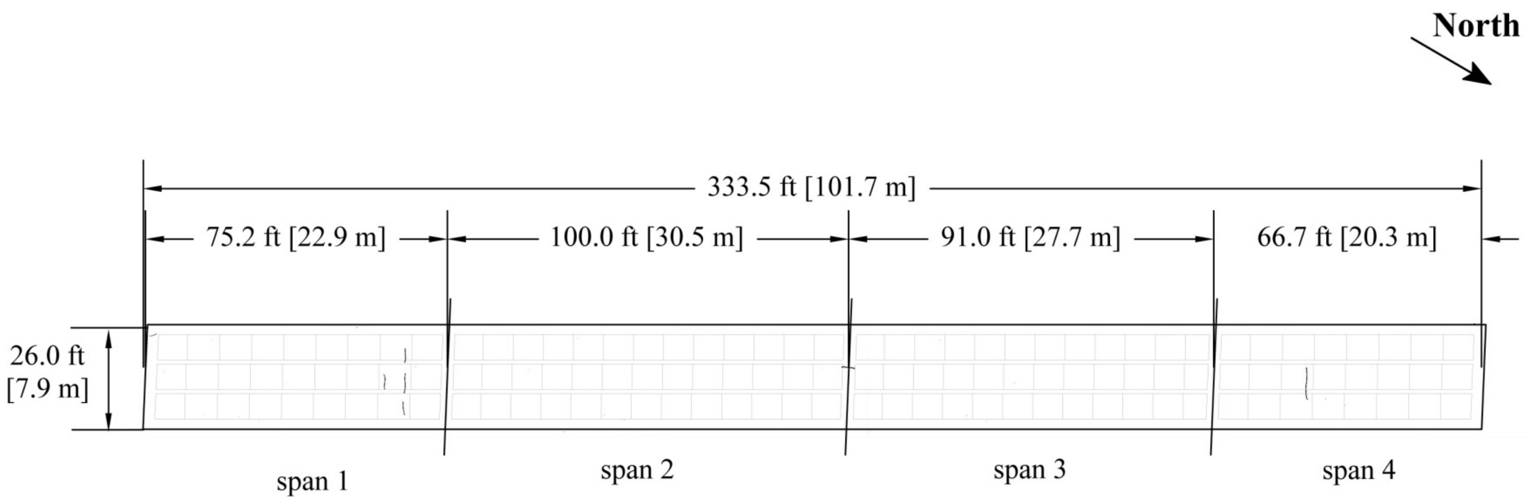
Bridge Age: 51 months
Crack Density: 0.273 m/m^2
Span 1: 0.302 m/m^2
Span 2: 0.260 m/m^2
Span 3: 0.218 m/m^2

FIGURE 1.18
US-59 6 (Survey 3)

1.4.9 US-59 7

US-59 7 has deck panels supported by prestressed girders and was constructed by Ames. The concrete contains 535 lb/yd^3 (317 kg/m^3) of cementitious material, with 60% cement, 35% slag, 5% silica fume, and granite coarse aggregate. The concrete for this deck had an average slump of 3¼ inches (85 mm) percent, and an average air content of 6.25 percent. The w/c ratio and paste content was 0.42 and 23.99 percent respectively, and the average compressive strength was 4720 psi (32.5 MPa). The compressive strength falls within the range specified for LC-HPC. The air content was just slightly lower than specified minimum of 6.5 percent, and the w/c ratio is lower than the specified minimum of 0.44 for LC-HPC. The average concrete temperature was 10.3°F (6.2°C) higher than the average air temperature.

Two crack surveys were performed, at 31 and 45 months. At 31 months, the crack density was 0.010 m/m^2 (Figure 1.19). The crack density at 45 month was 0.019 m/m^2 (Figure 1.20).This deck has the lowest crack density of all the US-59 bridges. It is also lower than for any of the LC-HPC decks with either prestressed or steel girders. The transverse cracks on the deck are aligned along the joints of the deck panels.

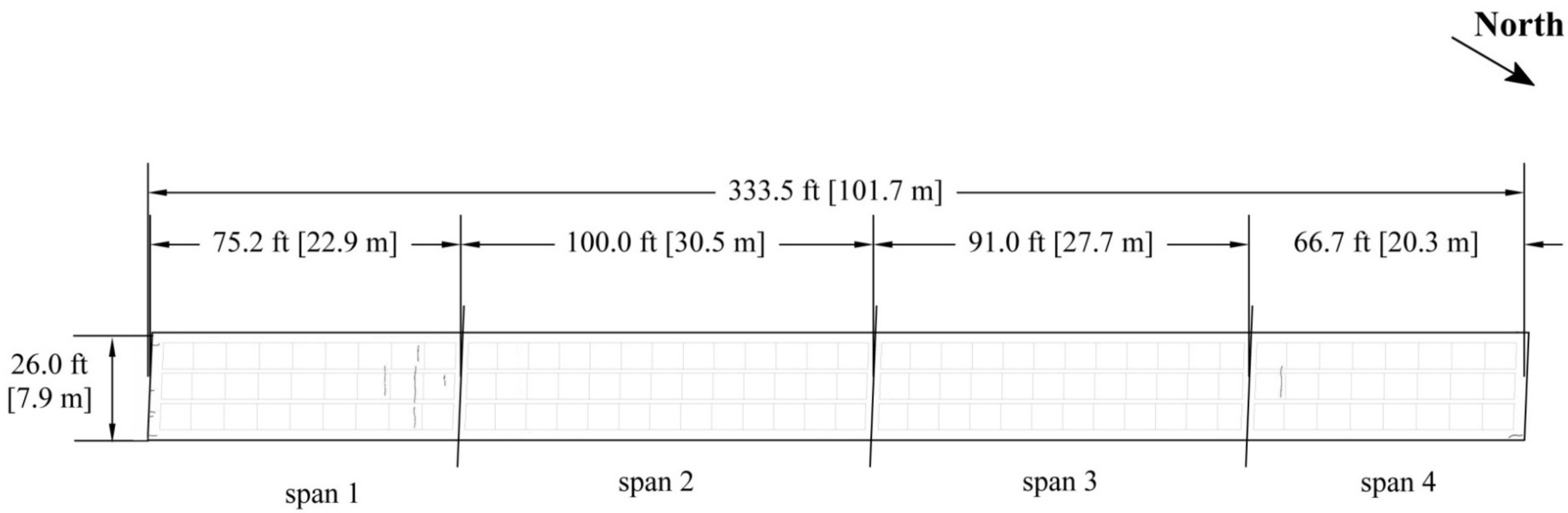


Bridge Number: 59-30-18.76
Bridge Location: SB US-59 Over I-35
Construction Date: 11-1-2008
Crack Survey Date: 7-18-2011

Bridge Length: 333.5 ft (101.7 m)
Bridge Width: 26.0 ft (7.9 m)
Skew: -2.3°
Number of Spans: 4
Span 1: 75.3 ft (23.0 m)
Span 2: 100.0 ft (30.5 m)
Span 3: 91.0 ft (27.7 m)
Span 4: 67.3 ft (20.5 m)

Bridge Age: 31 months
Crack Density: 0.010 m/m²
Span 1: 0.028 m/m²
Span 2: 0.000 m/m²
Span 3: 0.003 m/m²
Span 4: 0.014 m/m²

FIGURE 1.19
US-59 7 (Survey 1)



Bridge Number: 59-30-18.76
Bridge Location: SB US-59 Over I-35
Construction Date: 11-1-2008
Crack Survey Date: 7-25-2012

Bridge Length: 333.5 ft (101.7 m)
Bridge Width: 26.0 ft (7.9 m)
Skew: -2.3°
Number of Spans: 4
- **Span 1:** 75.3 ft (23.0 m)
- **Span 2:** 100.0 ft (30.5 m)
- **Span 3:** 91.0 ft (27.7 m)
- **Span 4:** 67.3 ft (20.5 m)

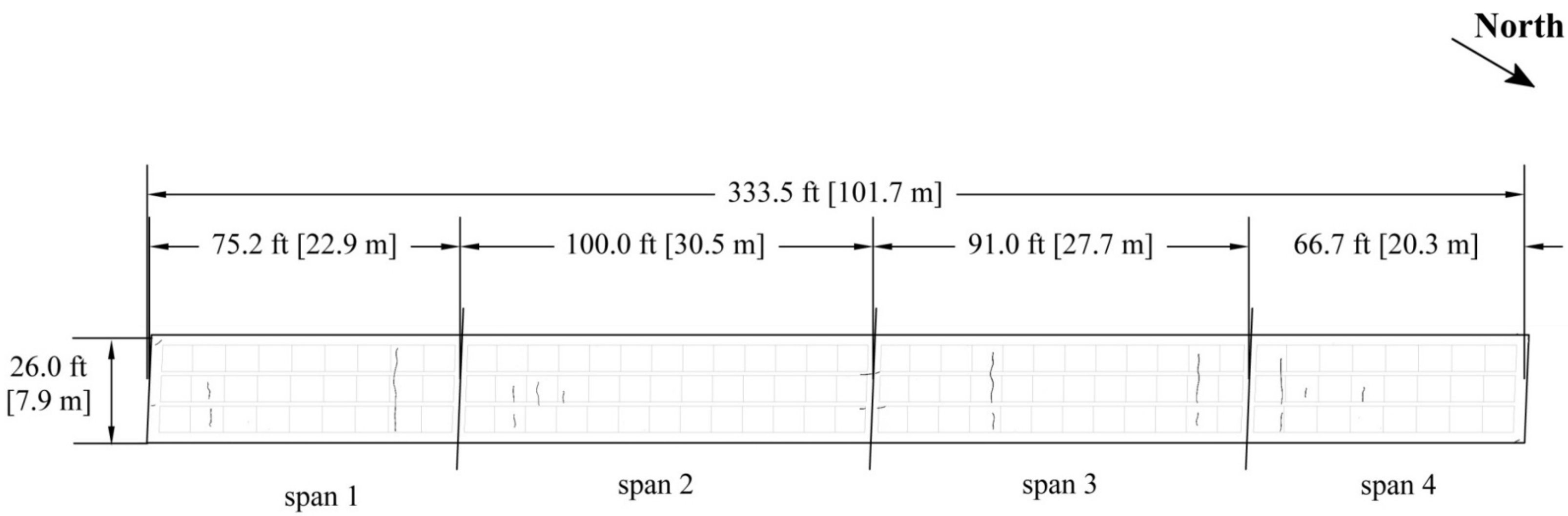
Bridge Age: 45 months
Crack Density: 0.019 m/m^2
- **Span 1:** 0.050 m/m^2
- **Span 2:** 0.000 m/m^2
- **Span 3:** 0.000 m/m^2
- **Span 4:** 0.017 m/m^2

FIGURE 1.20
US-59 7 (Survey 2)

1.4.10 US-59 8

US-59 8 is the twin to US-59 7, has deck panels supported by prestressed girders, and contains the same concrete mixture as US-59 7. It was constructed by Ames. The plastic concrete had an average slump of 2½ inches (65 mm) and an average air content of 6.25 percent. The w/c ratio and paste content was 0.42 and 23.99 percent respectively, and the compressive strength was 4580 psi (31.6 MPa). The average concrete temperature was 17.2°F (10°C) higher than the average air temperature.

Two crack surveys were performed at 33 and 45 months. At 33 months, the crack density was 0.039 m/m^2 (Figure 1.21), and at the 45 months, 0.049 m/m^2 (Figure 1.22). US-59 8 was second lowest cracking deck after its twin, US-59 7. Lower cracking in US-59 7 and 8 may be due to lower paste content (23.99%) than other decks on US-59. Only a small number of cracks were visible on this bridge deck, and most of those are aligned along joints of the deck panels. The w/c ratio and the air content are the same for the twin bridges, and the compressive strength and slumps are similar.

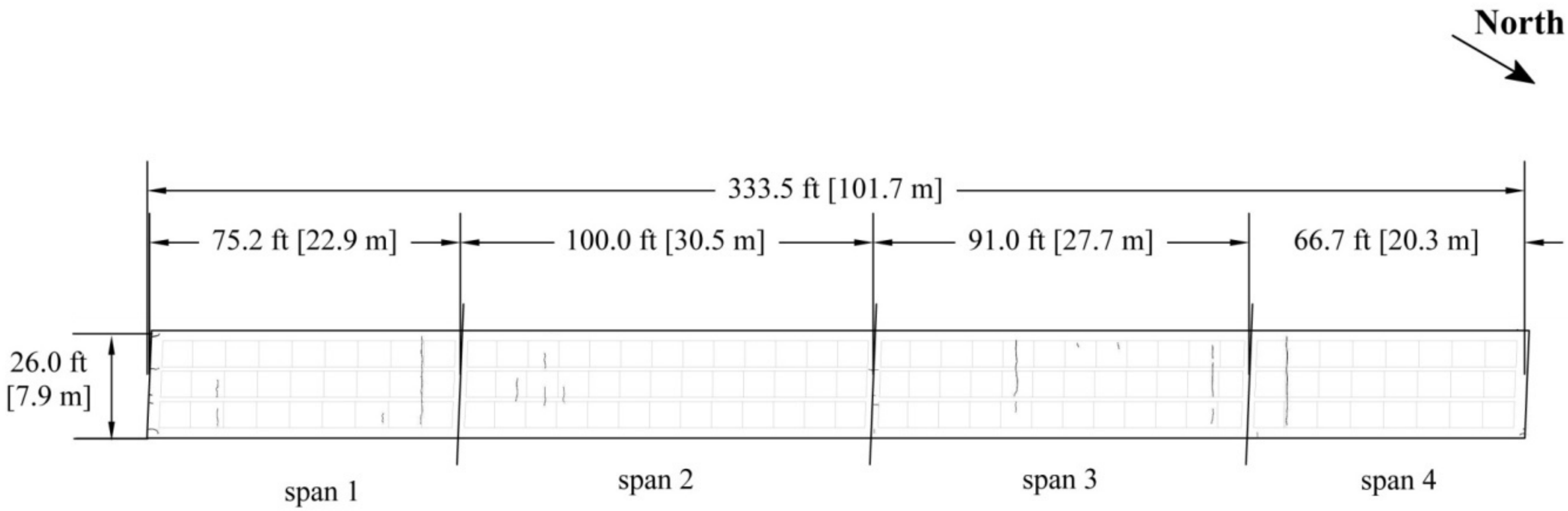


Bridge Number: 59-30-18.75
Bridge Location: NB US-59 Over I-35
Construction Date: 10-29-2008
Crack Survey Date: 7-18-2011

Bridge Length: 333.5 ft (101.7 m)
Bridge Width: 26.0 ft (7.9 m)
Skew: -2.3°
Number of Spans: 4
- **Span 1:** 75.3 ft (23.0 m)
- **Span 2:** 100.0 ft (30.5 m)
- **Span 3:** 91.0 ft (27.7 m
- **Span 4:** 67.3 ft (20.5 m)

Bridge Age: 33 months
Crack Density: 0.039 m/m^2
- **Span 1:** 0.050 m/m^2
- **Span 2:** 0.023 m/m^2
- **Span 3:** 0.046 m/m^2
- **Span 4:** 0.040 m/m^2

FIGURE 1.21
US-59 8 (Survey 1)



Bridge Number: 59-30-18.75
Bridge Location: NB US-59 Over I-35
Construction Date: 10-29-2008
Crack Survey Date: 7-25-2012

Bridge Length: 333.5 ft (101.7 m)
Bridge Width: 26.0 ft (7.9 m)
Skew: -2.3°
Number of Spans: 4
Span 1: 75.3 ft (23.0 m)
Span 2: 100.0 ft (30.5 m)
Span 3: 91.0 ft (27.7 m
Span 4: 67.3 ft (20.5 m)

Bridge Age: 45 months
Crack Density: 0.049 m/m²
Span 1: 0.062 m/m²
Span 2: 0.022 m/m²
Span 3: 0.048 m/m²
Span 4: 0.042 m/m²

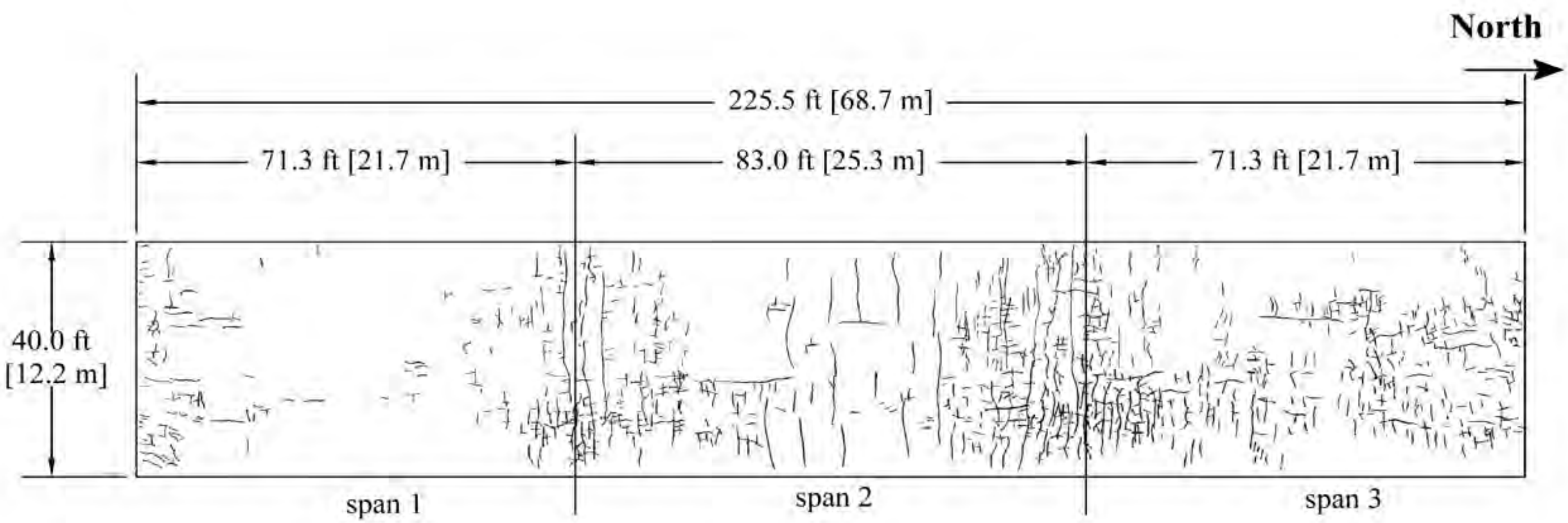
FIGURE 1.22
US-59 8 (Survey 2)

1.4.11 US-59 9

US-59 9 is supported by prestressed girders, has a silica fume overlay and was constructed by Beachner. The concrete in the subdeck contains 600 lb/yd^3 (356 kg/m^3) of cement and limestone coarse aggregate. The average slump for the subdeck was 3¾ inches (95 mm), and the average air content was 6.25 percent. The w/c ratio, paste content and compressive strength of the subdeck were, respectively, 0.44, 26.68 percent and 5110 psi (35.2 MPa). The w/c ratio and compressive strengths were within the limits set by LC-HPC specifications, but the slump was higher and the air content lower than the specified for LC-HPC bridge decks. The difference between the average concrete and air temperatures was 17.3°F (9.8°C). The average slump and compressive strength for the silica fume overlay was 4 inches and 9100 psi (62.7 MPa) respectively.

Two crack surveys were performed, at 33 and 45 months. The crack density at 33 months was 0.719 m/m^2 (Figure 1.23). At 45 months, the crack density increased to 0.853 m/m^2 (Figure 1.24). Many of the cracks were short and branch off each other in patterns that can be best described as map cracking. This type of cracking resembles plastic shrinkage cracking and may be attributed due to a delay in curing. Cracks are present throughout the length of the bridge, but more cracking is concentrated in the negative moment regions of the deck.

This deck has the highest crack density for any of the US-59 decks and the value is much higher than the crack density of the control bridge deck in the LC-HPC study that is supported by prestressed girders. Crack density is also much higher than average of 0.565 m/m^2 for the silica fume overlay control decks and the average of 0.420 m/m^2 for the control decks without silica fume overlay supported by steel girders in the LC-HPC study. This increase in crack density is likely due to the high strength (9100 psi) of the silica fume overlay and the high slump concrete used in the subdeck.

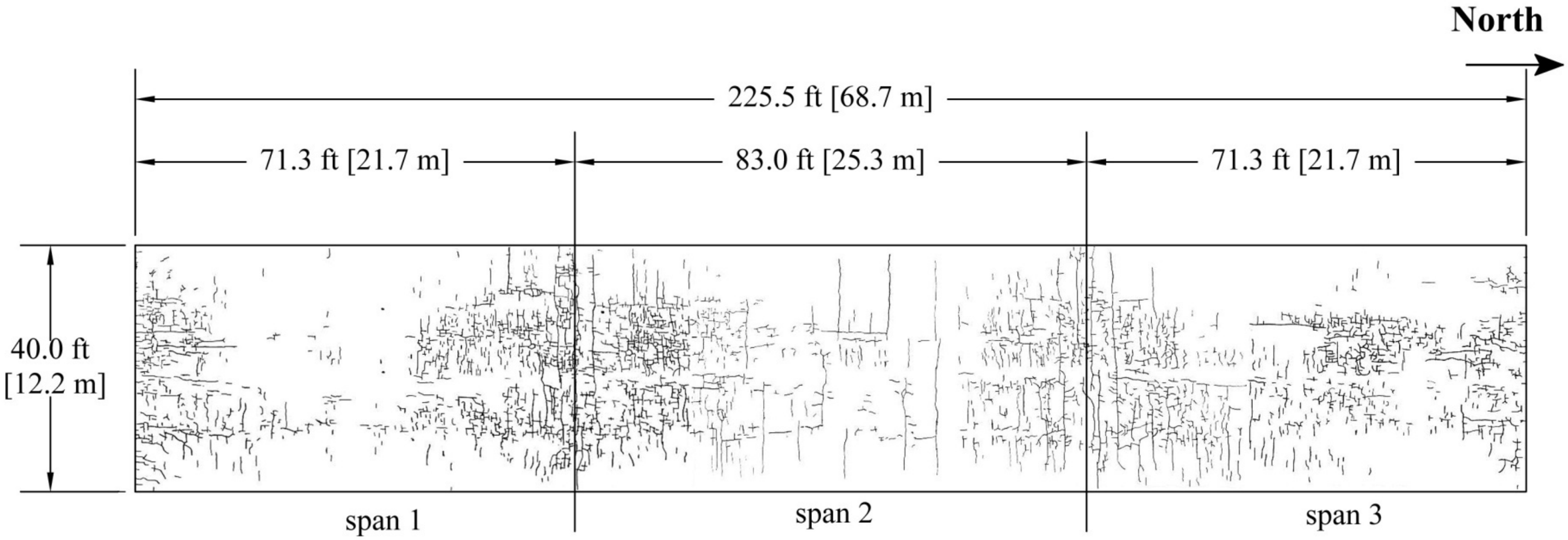


Bridge Number: 59-30-24.51
Bridge Location: SB US-59 over Stafford Rd.
Construction Date: 10-21-2008
Crack Survey Date: 7-11-2011

Bridge Length: 225.5 ft (68.7 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 0°
Number of Spans: 3
- **Span 1:** 71.3 ft (21.7 m)
- **Span 2:** 83.0 ft (25.3 m)
- **Span 3:** 71.3 (21.7 m)
- **Number of Placements:** 1

Bridge Age: 33 months
Crack Density: 0.719 m/m^2
- **Span 1:** 0.369 m/m^2
- **Span 2:** 0.895 m/m^2
- **Span 3:** 0.875 m/m^2

FIGURE 1.23
US-59 9 (Survey 1)



Bridge Number: 59-30-24.51
Bridge Location: SB US-59 over Stafford Rd.
Construction Date: 10-21-2008
Crack Survey Date: 7-16-2012

Bridge Length: 225.5 ft (68.7 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 0°
Number of Spans: 3
- **Span 1:** 71.3 ft (21.7 m)
- **Span 2:** 83.0 ft (25.3 m)
- **Span 3:** 71.3 (21.7 m)
- **Number of Placements:** 1

Bridge Age: 45 months
Crack Density: 0.853 m/m^2
- **Span 1:** 0.493 m/m^2
- **Span 2:** 1.026 m/m^2
- **Span 3:** 1.010 m/m^2

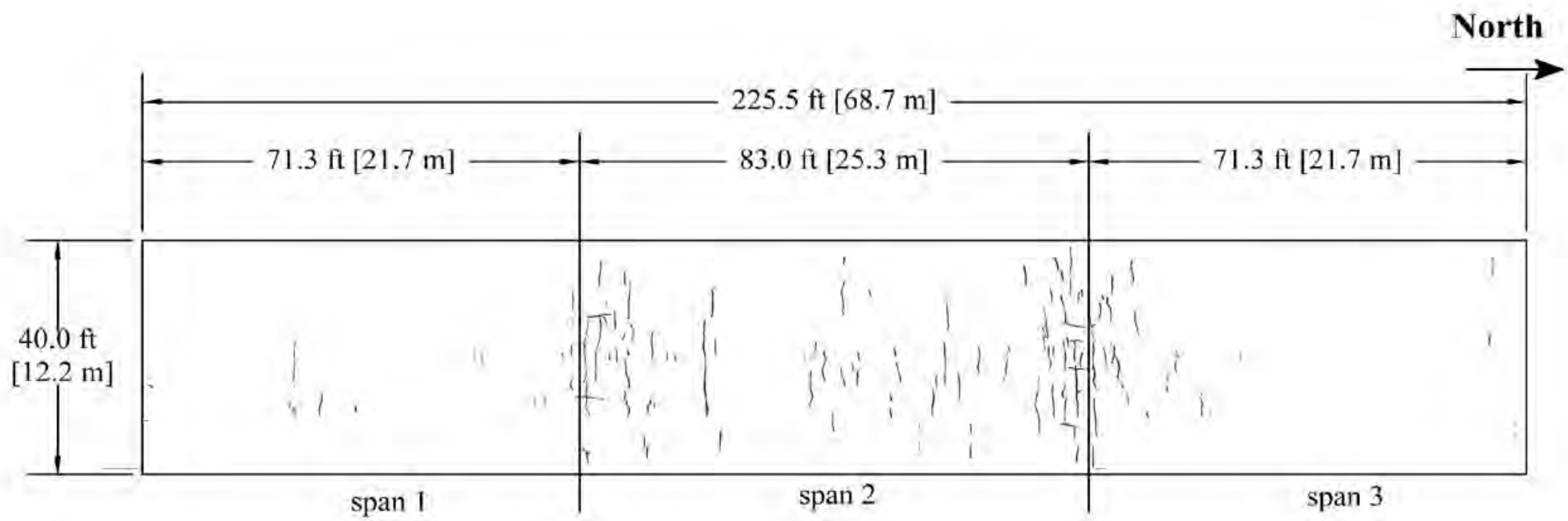
FIGURE 1.24
US-59 9 (Survey 2)

1.4.12 US-59 10

This is the twin to US-59 9. As with US-59 9, the deck is supported by prestressed girders and Beachner was the contractor. This deck, however, is monolithic, and the concrete contains limestone coarse aggregate, 1.55 inches long synthetic fibers (Grace 90/40 Strux), and a lower cement content, 560 lb/yd^3 (332 kg/m^3), than US-59 9. The average slump was 3 inches (75 mm) and the average air content was 7.0 percent for the subdeck. The w/c ratio, paste content and compressive strength for deck were, respectively, 0.42, 24.62 percent, and 5100 psi (35.2 MPa). The difference between the average concrete and air temperatures was 29.7°F (16.6°C) and the average air temperature was 34°F (1°C), which are higher and lower, respectively, than the LC- HPC requirements for cold weather placing concrete.

The bridge was surveyed twice, at 31 and 43 months. At 31 months, the crack density was 0.150 m/m^2 (Figure 1.25), and at 43 months, the crack density was 0.217 m/m^2 (Figure 1.26). This deck has more cracking than any other US-59 deck without an overlay supported by prestressed girders in this study. This is likely due to significant difference between the average concrete and air temperatures (29.7°F, 16.6°C) and low average air temperature during placement (34°F, 1°C), which may have contributed to thermal cracking. The crack density for this deck at 42 months (0.211 m/m^2) is higher than the control bridge deck in the LC-HPC study that is supported by prestressed girders at 42 months (0.205 m/m^2). Most of the cracks on US-59 10 are short and located in the middle span and over the piers of the bridge.

In spite of its relatively high crack density, the crack density of US-59 10 is only a fourth of the crack density of its twin, US-59 9. Since this deck does not have an overlay, has a lower cement content, a lower average slump, a lower paste content and a higher average air content, the crack density would be expected to be lower than that observed for US-59 9. With all of the differences between the two decks and with the high difference between the concrete and air temperatures, it is hard to conclude if the fibers helped decrease cracking for US-59 10.

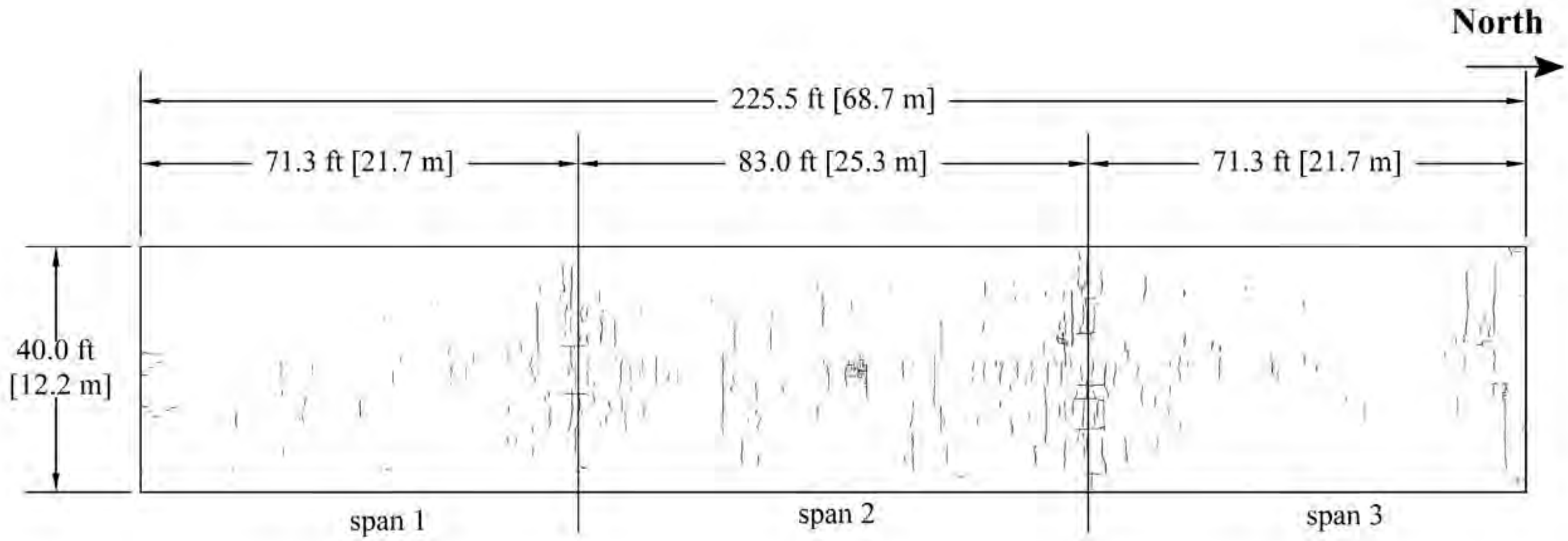


Bridge Number: 59-30-24.50
Bridge Location: NB US-59 over Stafford Rd.
Construction Date: 12-6-2008
Crack Survey Date: 7-8-2011

Bridge Length: 225.5 ft (68.7 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 0°
Number of Spans: 3
- **Span 1:** 71.3 ft (21.7 m)
- **Span 2:** 83.0 ft (25.3 m)
- **Span 3:** 71.3 (21.7 m)
- **Number of Placements:** 1

Bridge Age: 31 months
Crack Density: 0.150 m/m^2
- **Span 1:** 0.039 m/m^2
- **Span 2:** 0.234 m/m^2
- **Span 3:** 0.189 m/m^2

FIGURE 1.25
US-59 10 (Survey 1)



Bridge Number: 59-30-24.50
Bridge Location: NB US-59 over Stafford Rd.
Construction Date: 10-21-2008
Crack Survey Date: 7-16-2012

Bridge Length: 225.5 ft (68.7 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 0°
Number of Spans: 3
  Span 1: 71.3 ft (21.7 m)
  Span 2: 83.0 ft (25.3 m)
  Span 3: 71.3 (21.7 m)
  Number of Placements: 1

Bridge Age: 43 months
Crack Density: 0.217 m/m^2
  Span 1: 0.129 m/m^2
  Span 2: 0.313 m/m^2
  Span 3: 0.186 m/m^2

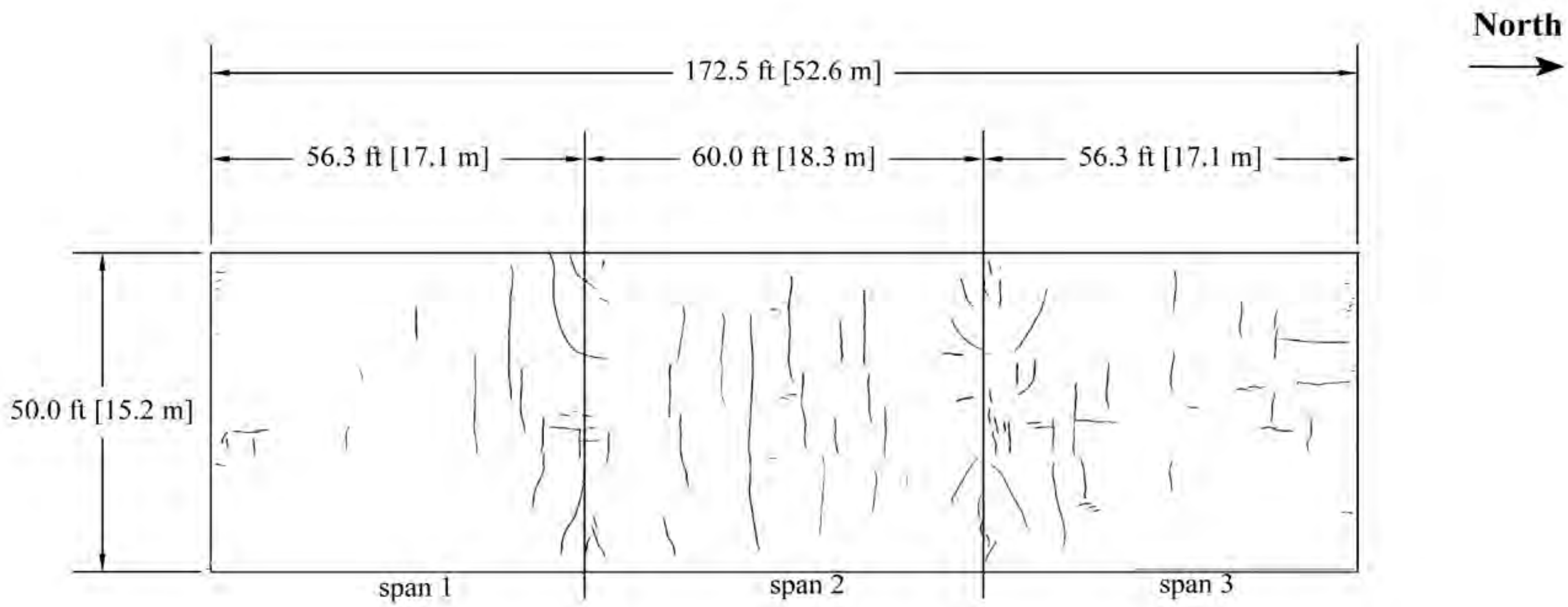
FIGURE 1.26
US-59 10 (Survey 2)

1.4.13 US-59 11

US-59 11 is supported by prestressed girders and has a silica fume overlay. Reece was the contractor. The concrete contains 620 lb/yd^3 (368 kg/m^3) of cement and limestone coarse aggregate. The subdeck concrete had an average slump of 4¾ inches (120 mm) and an average air content of 7.75 percent. The w/c ratio was 0.44, paste content was 27.95 percent, and the compressive strength of the subdeck was 4480 psi (30.9 MPa). The air content, w/c ratio, and compressive strength all fell within the specified ranges for LC-HPC, but the average slump was higher than the maximum specified slump of 3½ inches (90 mm), and the paste content was well above the range for LC-HPC decks. The difference between the average concrete and air temperatures was 16.3°F (8.6°C). The average slump and compressive strength of silica fume overlay was 3½ inches and 5470 psi (37.7 MPa).

The US-59 11 bridge was surveyed two times, at 33 and 46 months. The crack density at 33 months was 0.213 m/m^2 (Figure 1.27). At 46 months, the crack density increased slightly to 0.225 m/m^2 (Figure 1.28). The deck has long transverse cracks, more located on the middle of the mid span, long diagonal cracks over the piers, and short longitudinal cracks at both abutments.

The crack density for this bridge at 42 months, 0.221 m/m^2, which is higher than crack density of 0.205 m/m^2 for the control bridge deck with prestressed girders in the LC- HPC study at same age, and significantly lower than crack density 0.820 m/m^2 of US-59 9, which is also supported by prestressed girder, but has a silica fume overlay. US-59 11 was expected to crack more than US-59 9, due to its higher average slump and paste content. The crack density is much higher than decks on US-59 without overlays that are supported by prestressed girders.



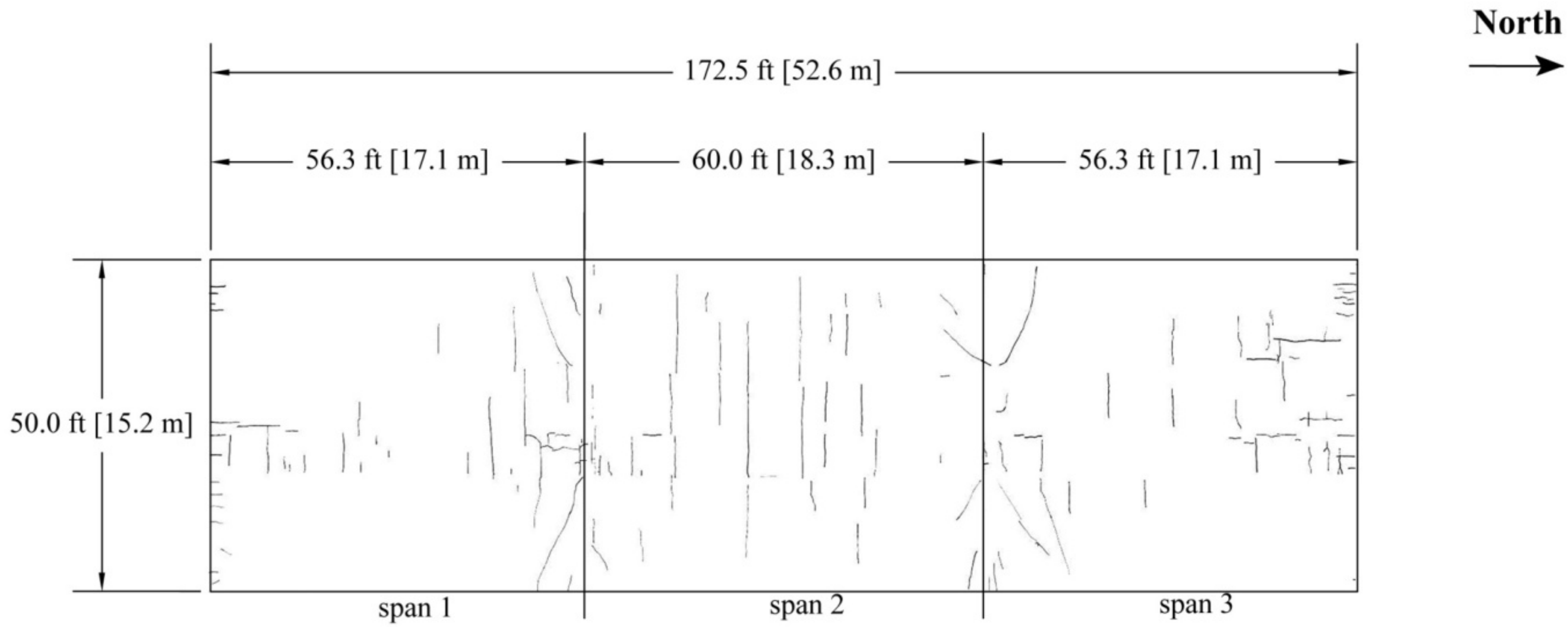
Bridge Number: 59-30-24.82
Bridge Location: SB US-59 over West Fork Tauy Creek
Construction Date: 10-3-08
Crack Survey Date: 7-11-11

Bridge Length: 172.5 ft (52.6 m)
Bridge Width: 50.0 ft (15.2 m)
Skew: 0°
Number of Spans: 3
- **Span 1:** 56.3 ft (17.1 m)
- **Span 2:** 60.0 ft (18.3 m)
- **Span 3:** 56.3 ft (17.1 m)

Number of Placements: 1

Bridge Age: 33 months
Crack Density: 0.213 m/m^2
- **Span 1:** 0.160 m/m^2
- **Span 2:** 0.258 m/m^2
- **Span 3:** 0.223 m/m^2

FIGURE 1.27
US-59 11 (Survey 1)



Bridge Number: 59-30-24.82
Bridge Location: SB US-59 over West Fork Tauy Creek
Construction Date: 10-3-08
Crack Survey Date: 7-30-12

Bridge Length: 172.5 ft (52.6 m)
Bridge Width: 50.0 ft (15.2 m)
Skew: 0°
Number of Spans: 3
- **Span 1:** 56.3 ft (17.1 m)
- **Span 2:** 60.0 ft (18.3 m)
- **Span 3:** 56.3 ft (17.1 m)

Number of Placements: 1

Bridge Age: 46 months
Crack Density: 0.225 m/m^2
- **Span 1:** 0.201 m/m^2
- **Span 2:** 0.244 m/m^2
- **Span 3:** 0.225 m/m^2

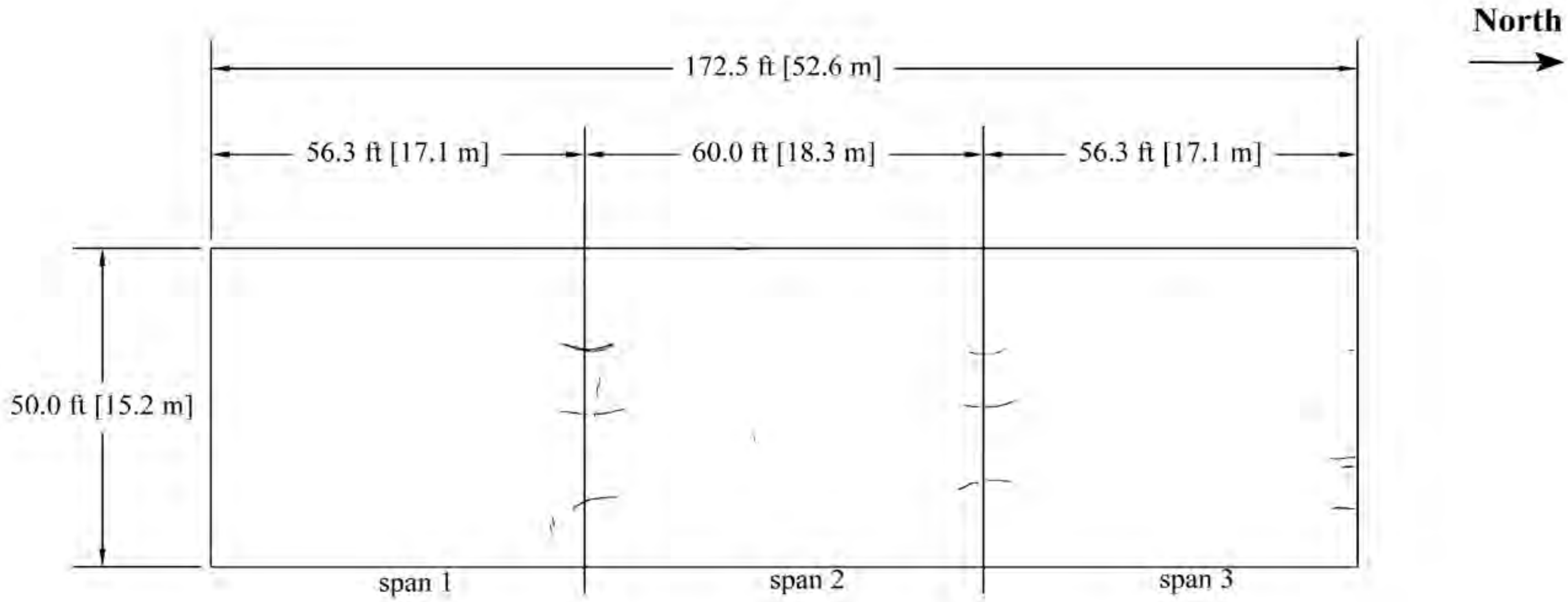
FIGURE 1.28
US-59 11 (Survey 2)

1.4.14 US-59 12

US-59 12 is the twin bridge to US-59 11. As with US-59 11, it is supported by prestressed girders. It was also constructed by Reece. The concrete also contains limestone coarse aggregate, but unlike US-59 11, does not have an overlay, has ¾ inch long synthetic fibers (Grace fibers) in the bridge deck, and has a lower cement content of 560 lb/yd^3 (332 kg/m^3). The average slump was 4 inches (100 mm), and the average air content was 7 percent. The w/c ratio, paste content and the compressive strength were, respectively, 0.42, 24.62 percent and 5740 psi (39.6 MPa). The air content was within the range specified for LC-HPC, but the slump was higher than the specified maximum, the w/c ratio was lower than the specified minimum, and the compressive strength was higher than the specified maximum. The average concrete temperature was 17.5 °F (9.4°C) higher than the average air temperature on the day of placement.

Two crack surveys were performed, at 30 and 43 months. The crack density at the 30 months was 0.022 m/m^2 (Figure 1.29). At 43 months, it increased to 0.075 m/m^2 (Figure 1.30). In survey 1 most of cracks were over the piers and abutments, but in survey 2, significant cracking was observed in the middle of the span. Overall, crack density is low, which is consistent with the other US-59 and LC-HPC bridge decks supported by the prestressed girders with low cement contents and no overlays at a similar age. The crack density at 42 month, 0.072 m/m^2, is lower than the average crack density of 0.214 m/m^2 for the two LC-HPC decks supported by prestressed girders. It is also much lower than the averages for both the LC-HPC decks supported by steel girders and the older monolithic decks, all of which were supported by steel girders.

The crack density of US-59 12 is significantly lower than the crack density of its twin, which could be attributed to the fact that US-59 12 has no overlay. Fibers in the US-59 12 deck may have also attributed to its lower crack density, but a direct comparison to a matching structure without fibers is not available in this study. The very low crack density, however, suggests that follow-up work should be considered.

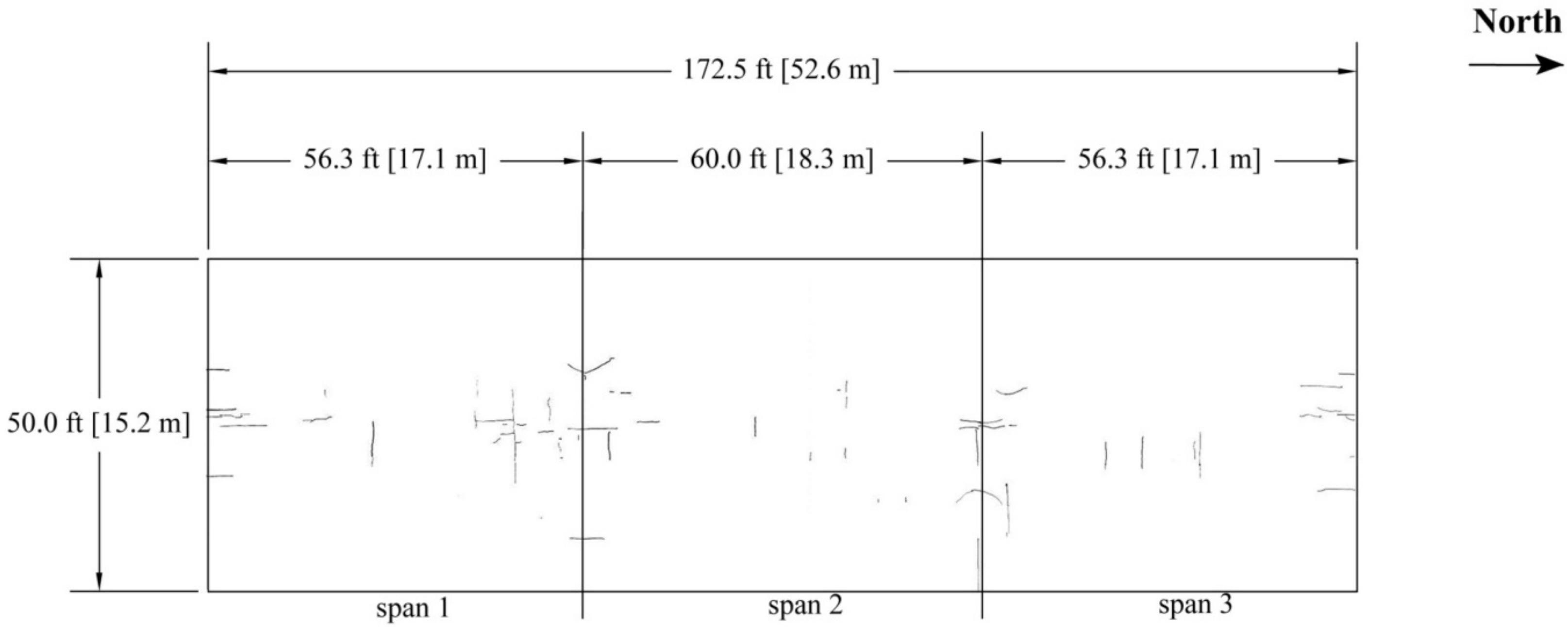


Bridge Number: 59-30-24.83
Bridge Location: NB US-59 over West Fork Tauy Creek
Construction Date: 1-9-2009
Crack Survey Date: 7-8-2011

Bridge Length: 172.5 ft (52.6 m)
Bridge Width: 50.0 ft (15.2 m)
Skew: 0°
Number of Spans: 3
Span 1: 56.3 ft (17.1 m)
Span 2: 60.0 ft (18.3 m)
Span 3: 56.3 ft (17.1 m)
Number of Placements: 1

Bridge Age: 30 months
Crack Density: 0.022 m/m^2
Span 1: 0.013 m/m^2
Span 2: 0.030 m/m^2
Span 3: 0.021 m/m^2

FIGURE 1.29
US-59 12 (Survey 1)



Bridge Number: 59-30-24.83
Bridge Location: NB US-59 over West Fork Tauy Creek
Construction Date: 1-9-2009
Crack Survey Date: 7-30-2012

Bridge Length: 172.5 ft (52.6 m)
Bridge Width: 50.0 ft (15.2 m)
Skew: 0°
Number of Spans: 3
- **Span 1:** 56.3 ft (17.1 m)
- **Span 2:** 60.0 ft (18.3 m)
- **Span 3:** 56.3 ft (17.1 m)

Number of Placements: 1

Bridge Age: 43 months
Crack Density: 0.075 m/m²
- **Span 1:** 0.088 m/m²
- **Span 2:** 0.062 m/m²
- **Span 3:** 0.074 m/m²

FIGURE 1.30
US-59 12 (Survey 2)

Chapter 2: Summary of Results and Comparisons with LC-HPC Bridge Decks

Of the twelve bridges surveyed, eight have prestressed concrete girders and four have steel girders. For the decks with prestressed girders, four have partial-depth precast deck panels, two are monolithic with synthetic fibers, and two have overlays. Of the four decks with steel girders, two have overlays, and two are monolithic. One of the two decks on steel girders with an overlay has fibers in the overlay. In this section, the crack survey results for the US-59 bridge decks are summarized and compared with the crack densities obtained from the LC-HPC bridge deck study. The values, including the crack densities interpolated to 42 months, are presented in Table 2.1.

TABLE 2.1
Summary of Crack Densities for Bridge Decks on US-59

Bridge ID	Date of Placement	2010 Survey		2011 Survey		2012 Survey		42-month Crack Density (m/m^2)
		Age at Survey (months)	Crack Density (m/m^2)	Age at Survey (months)	Crack Density (m/m^2)	Age at Survey (months)	Crack Density (m/m^2)	
US 59-1	11/13/2008	22	0.280	31	0.385	45	0.403	0.399
US 59-2	11/25/2008	22	0.140	32	0.217	46	0.306	0.281
US 59-3	9/30/2008	23	0.035	32	0.051	46	0.070	0.065
US 59-4	9/19/2008	23	0.067	33	0.056	46	0.082	0.074
US 59-5	5/14/2008	28	0.270	38	0.320	46	0.465	0.393
US 59-6	4/30/2008	29	0.160	39	0.198	51	0.273	0.219
US 59-7	11/1/2008	–	–	31	0.010	45	0.019	0.017
US 59-8	10/29/2008	–	–	33	0.039	45	0.049	0.047
US 59-9	10/21/2008	–	–	33	0.719	45	0.853	0.820
US 59-10	12/6/2008	–	–	31	0.150	43	0.217	0.211
US 59-11	10/3/2008	–	–	33	0.213	46	0.225	0.221
US 59-12	1/9/2009	–	–	30	0.022	43	0.075	0.072

– No Survey

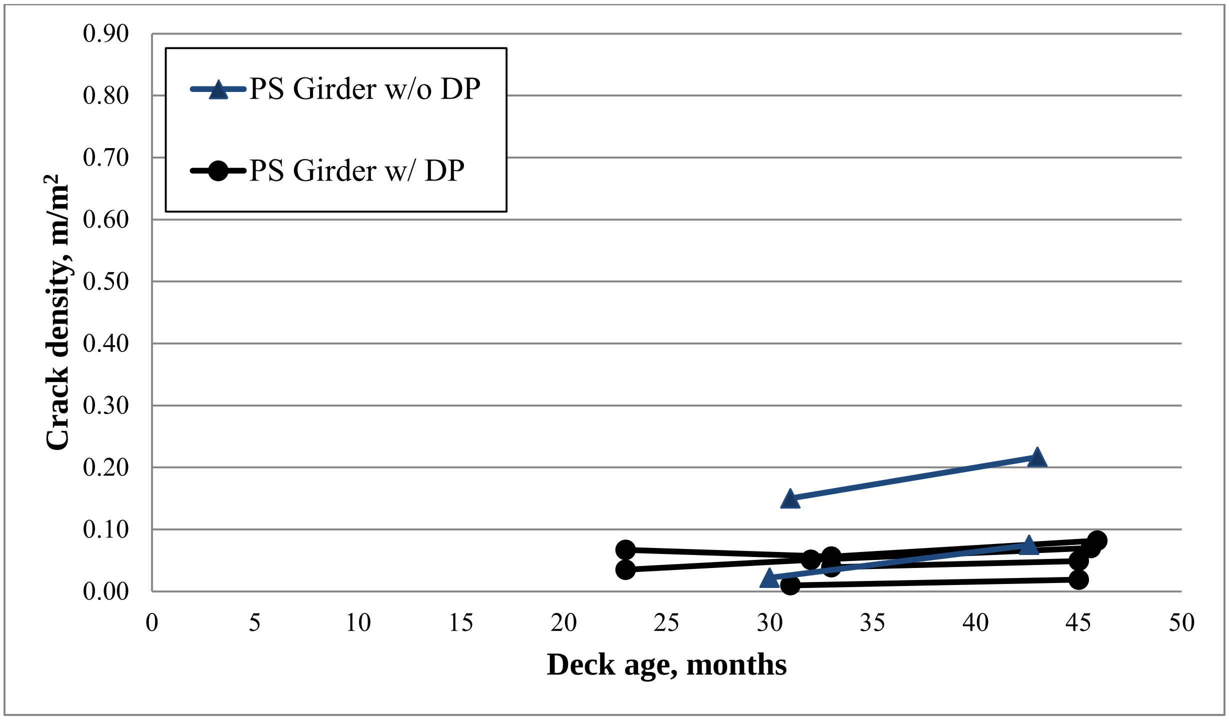


FIGURE 2.1
Crack Densitiy versus Age for US-59 Decks Supported by Prestressed Girders with and without Deck Panels (DP)

2.1 Deck Panels

Crack density is plotted versus bridge deck age for the two prestressed girder bridges with monolithic decks and the four with deck panels in Figure 2.1. For the latter, most transverse cracks appear to have formed above the joints of deck panels. All six bridges have low crack densities. The crack densities for the two decks without deck panels at the age of 42 months are 0.211 and 0.072 m/m^2 (for US-59 10 and 12, respectively), with an average of 0.142 m/m^2. The range for the four decks with deck panels at 42 months is 0.017 to 0.074 m/m^2, with an average of 0.051 m/m^2. With the exception of US-59 10, the crack densities are low. Because of the relatively narrow range in crack densities, the six decks shown in Figure 2.1 will be referred to as decks supported by prestressed girders, without a designation for deck panels in the remainder of the report.

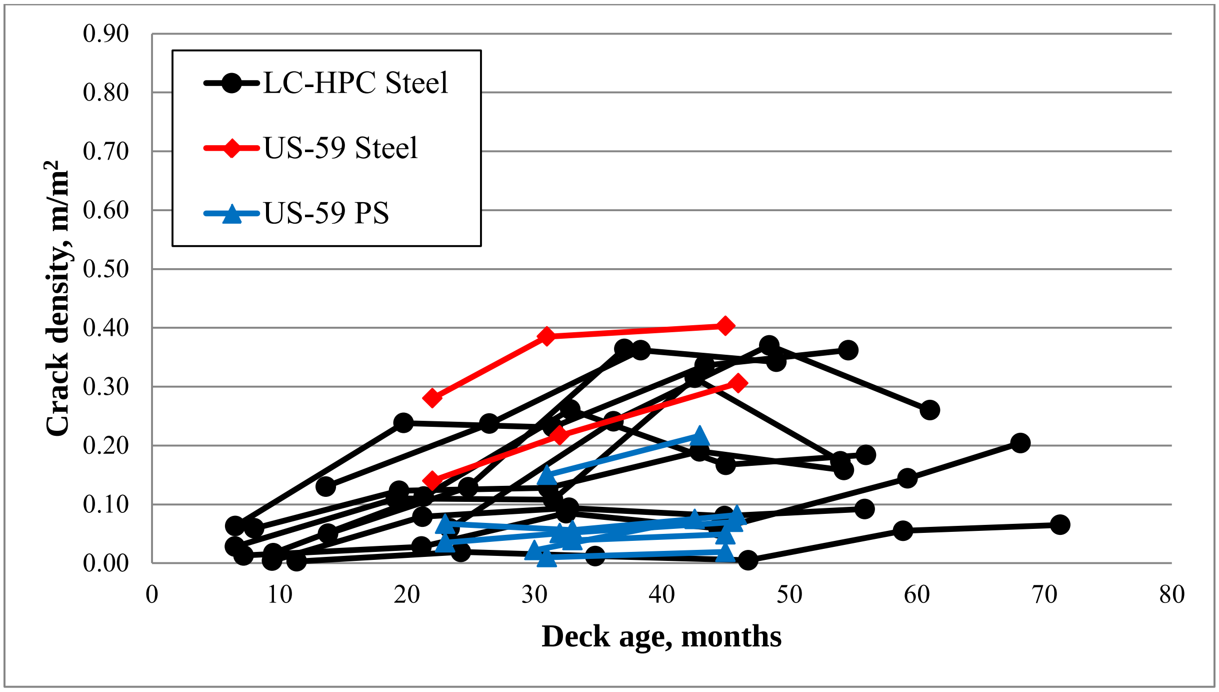


FIGURE 2.2
Crack Density versus Age for LC-HPC and US-59 Monolithic Decks Supported by Steel Girders Compared to US-59 Monolithic Decks Supported by Prestressed (PS) Girders

2.2 Steel Girders versus Prestressed Girders

Past research has shown that decks supported by prestressed girders exhibit less cracking than decks supported by steel girders (PCA 1970). Figure 2.2 compares crack densities for the LC-HPC and US-59 monolithic decks supported by steel girders with those on US-59 supported by prestressed girders as a function of age. These results support the earlier findings. Although the number of US-59 decks supported by prestressed girders is small, Figure 2.2 shows a clear trend of less cracking on the decks supported by prestressed girders compared to decks supported by steel girders, with five out of six of the decks on prestressed girders matching the best of the LC-HPC decks on steel girders and the majority of the decks on steel girders exhibiting more cracking than these five decks.

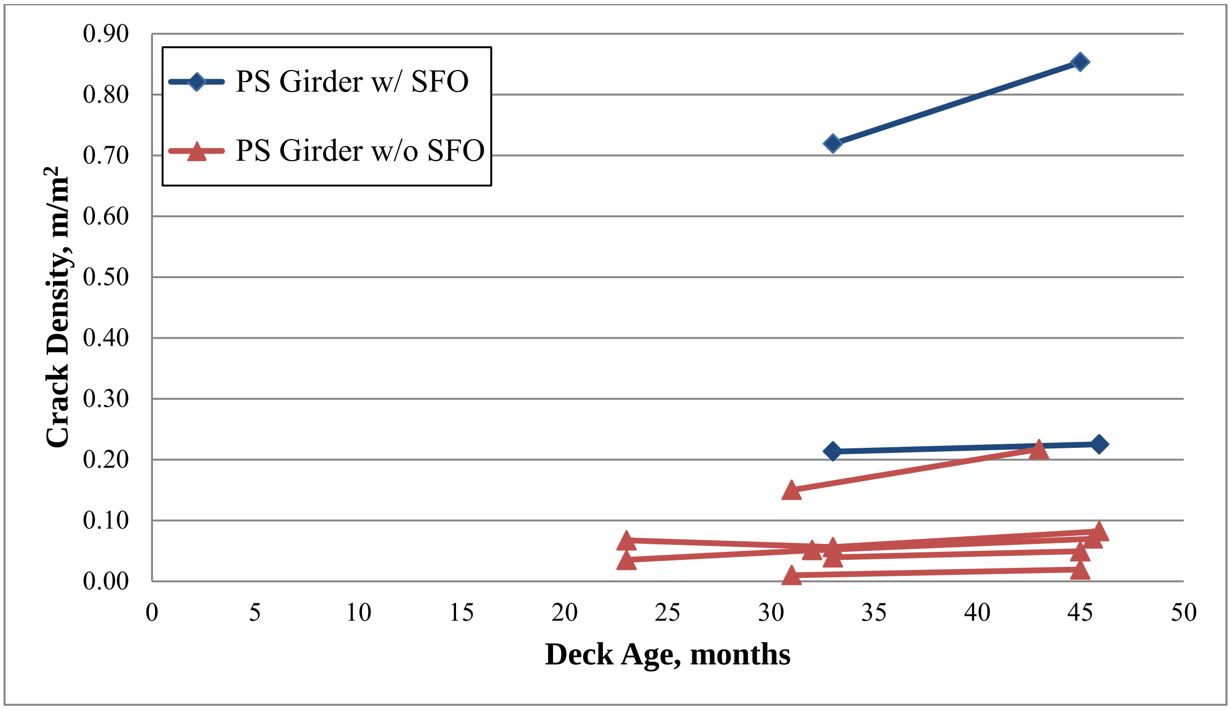


FIGURE 2.3
Crack Density versus Age for Decks with and without Silica Fume Overlays (SFO) Supported by Prestressed (PS) Girders

2.3 Overlays

Past research has also shown that the use of both conventional high-density and silica fume overlays (SFO) increases cracking (Lindquist et al. 2005). Knowing this, it would be expected that the decks with SFOs on US-59 would exhibit higher crack densities than the monolithic decks. Figure 2.3 compares crack density versus deck age for the decks on US-59 supported by prestressed concrete girders with and without SFOs. Both of the SFO decks in Figure 2.3 support earlier findings and exhibit higher crack densities than any of the decks without SFOs.

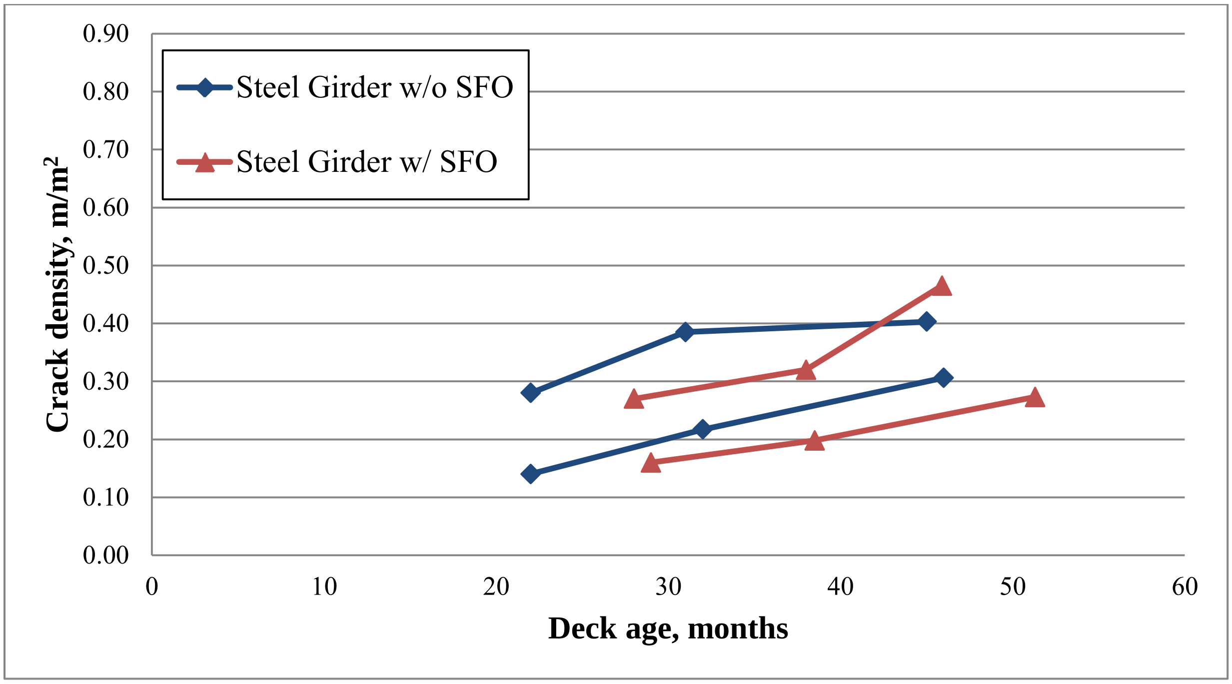


FIGURE 2.4
Crack Density versus Age for Decks and Decks with and without Silica Fume Overlays (SFO) Supported by Steel Girders

Crack density is plotted versus age for the US-59 decks supported by steel girders with and without SFOs in Figure 2.4. The average crack densities for the two decks with SFOs and two decks without SFOs as of the date of the last survey are, respectively, 0.306 and 0.340 m/m^2. The averages for the two deck types are similar. These results do not match the findings of past research that show benefits of monolithic decks. This could be attributed to the small number of decks in this sample, varying plastic and hardened concrete properties, and the practices employed by the different contractors.

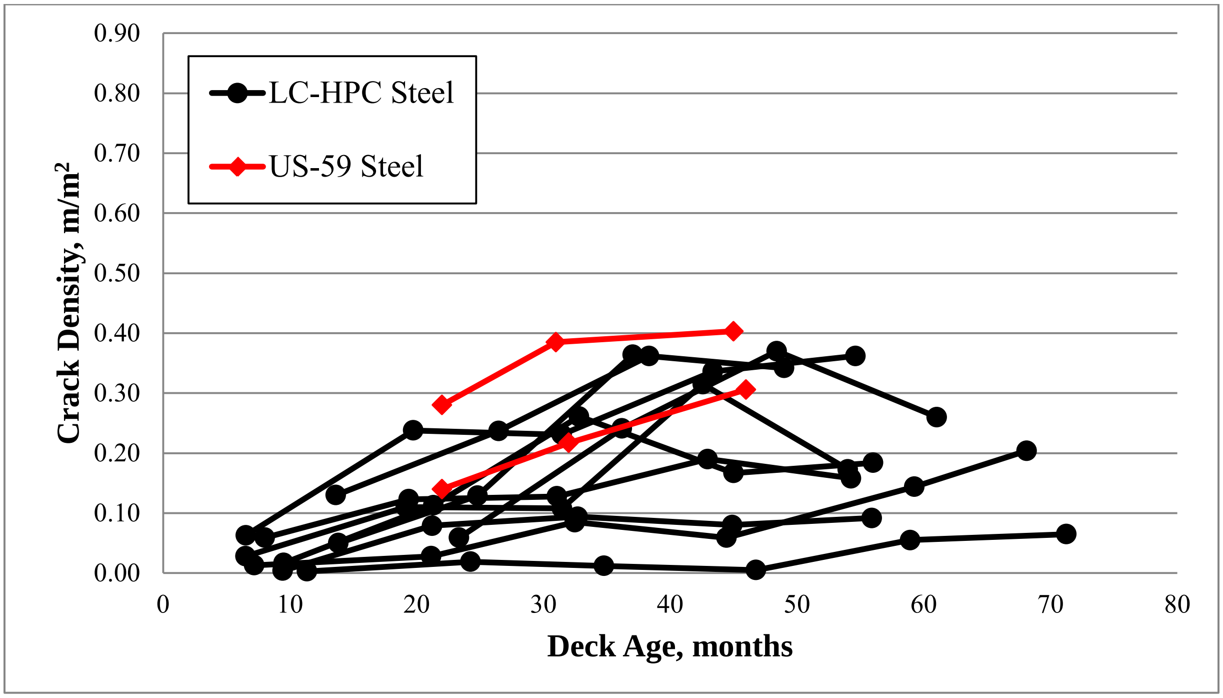


FIGURE 2.5
Crack Density for Age for LC-HPC and US-59 Monolithic Decks Supported by Steel Girders

2.4 LC- HPC and US-59 Decks Supported by Steel Girders

The average crack density for the US-59 monolithic decks supported by steel girders is higher than the average for LC-HPC steel girder bridge decks, as shown in Figure 2.5. The two monolithic decks on US-59 were not constructed in accordance to LC-HPC specifications. Both decks have a w/cm ratio of 0.42 (below the values of 0.44 and 0.45 used for LC-HPC decks) and tining was used for finishing, which delays the start of curing and is prohibited in the LC-HPC specifications. One US-59 deck (US-59 1) had an average slump (4 inches) that was higher than the *maximum* slump (3½ inches) allowed in LC-HPC decks, while the other US-59 deck (US-59 2) had an average slump equal to the maximum allowed for LC-HPC decks. The 28-day compressive strength of 6390 psi for US-59 2 exceeded the maximum of 5500 psi permitted for LC-HPC decks. The temperature difference of 26.3°F (14.5°C) between the concrete and air during placement of US-59 2 exceeded the maximum temperature difference (25°F) permitted by the LC-HPC specification for placing concrete in cold weather. Higher slump leads to greater settlement, higher compressive strength correlates with increased cracking, and a greater temperature difference leads to an increased potential for thermal cracks. Because of these factors, it would be expected that the monolithic US-59 steel girder bridge decks would crack more than the LC-HPC bridges.

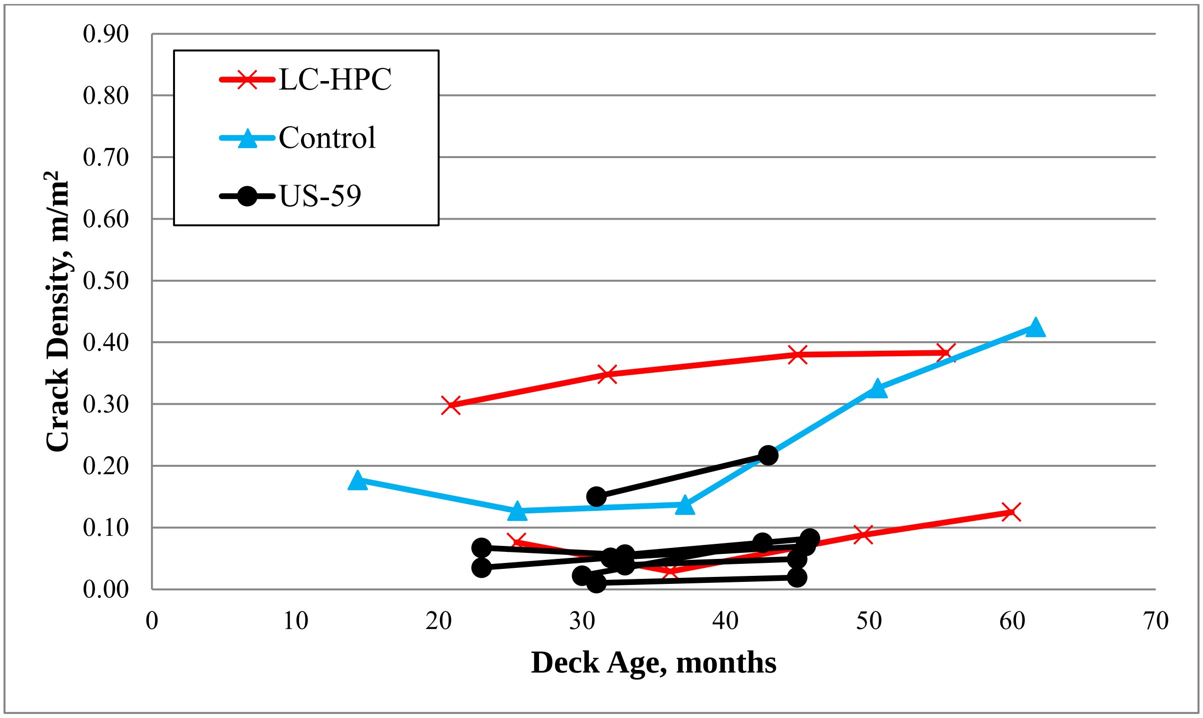


FIGURE 2.6
Crack Density versus Age for LC-HPC, Control, and US-59 Monolithic Decks Supported by Prestressed Girders

2.5 Monolithic LC-HPC, Control, and US-59 Decks Supported by Prestressed Girders

Crack density is plotted versus deck age for the monolithic LC-HPC, control (from the LC-HPC study), and US-59 decks supported by prestressed girders in Figure 2.6. One of the two LC-HPC decks (LC-HPC 8) has a high crack density that may be the result of excessive camber, as suggested by the crack pattern (Kaul et at. 2012). The monolithic US-59 decks and the other LC-HPC deck on prestressed girders exhibit similar crack densities. All of the US-59 monolithic bridge decks supported by prestressed girders, except US-59 10, have lower crack densities than the control deck from the LC-HCP study.

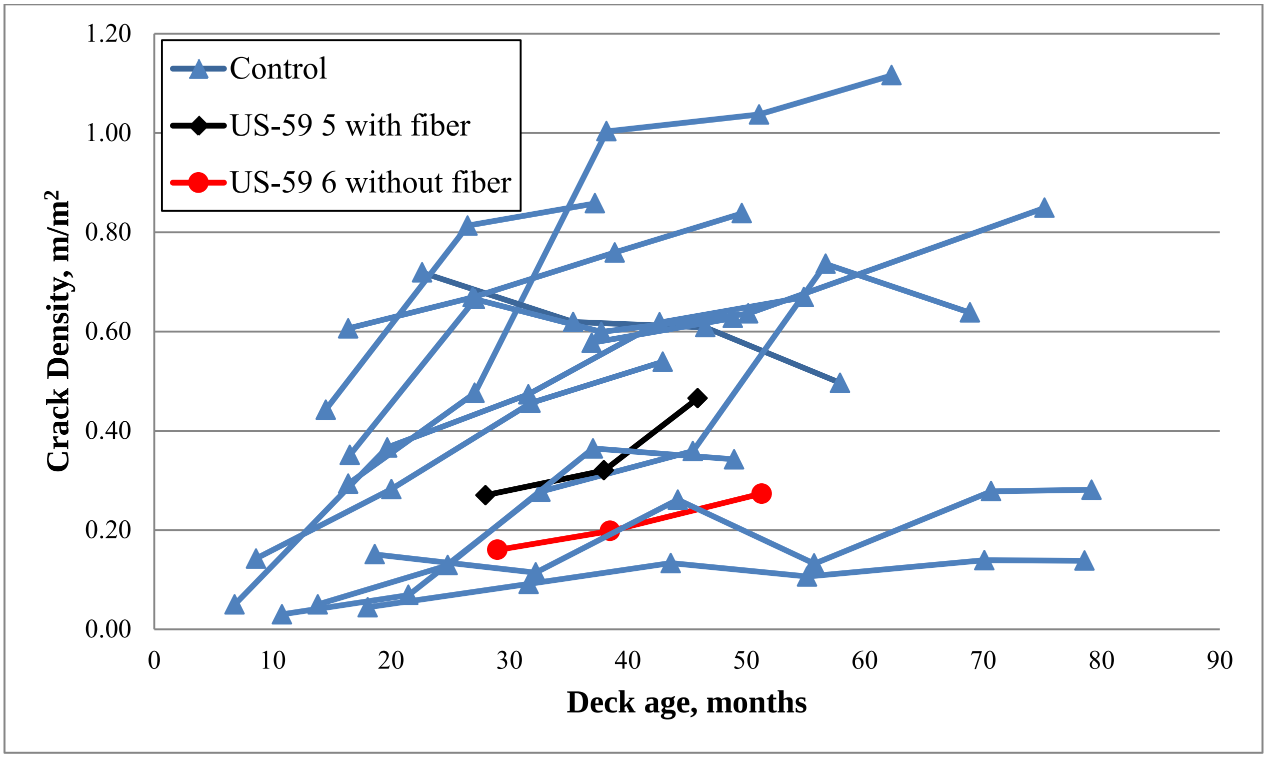


FIGURE 2.7
Crack Density versus Age for Control (from LC-HPC Study) and US-59 Decks with Silica Fume Overlays Supported by Steel Girders

2.6 Fibers

Average crack densities are plotted versus age for the control decks in the LC-HPC study (all of which had a silica fume overlay) and the US-59 decks with silica fume overlays supported by steel girders in Figure 2.7. The control decks in the LC-HPC study have higher crack densities than US-59 decks with or without fibers in the overlay. The higher crack densities may be due in part to higher average slumps in the subdecks of those control decks (range from 2.75 to 9.25 inches) (Yuan et al. 2011) compared to US-59 decks (range from 4.5 to 5 inches).

The one deck that contains fibers in the overlay, US-59 5, is supported by steel girders and has a 42-month crack density of 0.393 m/m^2. US-59 6, the twin bridge of US-59 5, contains no fibers in the overlay and has a lower crack density, 0.219 m/m^2, as shown in Figure 2.7.

US-59 10 and US-59 12 are monolithic decks constructed by Beachner and Reece, respectively. Both bridge decks contain fibers and are supported by prestressed concrete girders. These bridge decks have lower crack densities than their twin bridges, US-59 9 and US-59 11, respectively, which have silica fume overlays. The crack density for US-59 10 at 42 months is 0.211 m/m^2, which is higher than the crack density for US-59 12 (0.072 m/m^2) at the same age. The higher crack density for US-59 10 is likely due to a greater difference in average concrete and air temperature (29.7°F [16.6°C] versus 17.5°F [9.4°C] for US-59 12) and lower average air temperature (34°F [1°C] versus 44°F [7°C] for US-59 12) during concrete placement, which are not permitted by the LC-HPC specification for cold weather placing. No conclusions can be made on the benefits of fibers due to the dissimilar deck types in the study.

2.7 Slump

The LC-HPC specification requires concrete slump to be between 1½ and 3 inches to limit settlement cracking. The *average* slumps for the concrete placed in the US-59 decks ranged from 2½ to 5 inches. To evaluate the effect of slump on cracking for the US-59 decks, comparisons are limited to the eight decks constructed by Ames because the two decks placed by Beachner and by Reece each include one monolithic deck and one overlay deck. The crack densities at 42 months for the US-59 decks supported by prestressed and steel girders constructed by Ames are plotted as a function of average slump in Figure 2.8. At 42 months, an increase in crack density (0.281 to 0.399 m/m^2) is observed for the monolithic decks supported by steel girders without SFO as the average slump increases from 3.5 to 4 inches. Similarly, at 42 months, an increase in crack density is observed for decks supported by steel girders with a SFO as the average slump in the subdeck increases (0.219 m/m^2 for 4.5 inches slump versus 0.393 m/m^2 with a 5 inches slump). Average slumps for the decks supported by prestressed girders with deck panels range from 2.5 inches to 4 inches. An increase in crack density is also observed for decks supported by prestressed girders as the average slump increases, although the total crack densities and the average increase in crack densities are much lower than measured on the decks supported by steel girders. These results support earlier findings on the relationship between slump and cracking (Yuan et al. 2011).

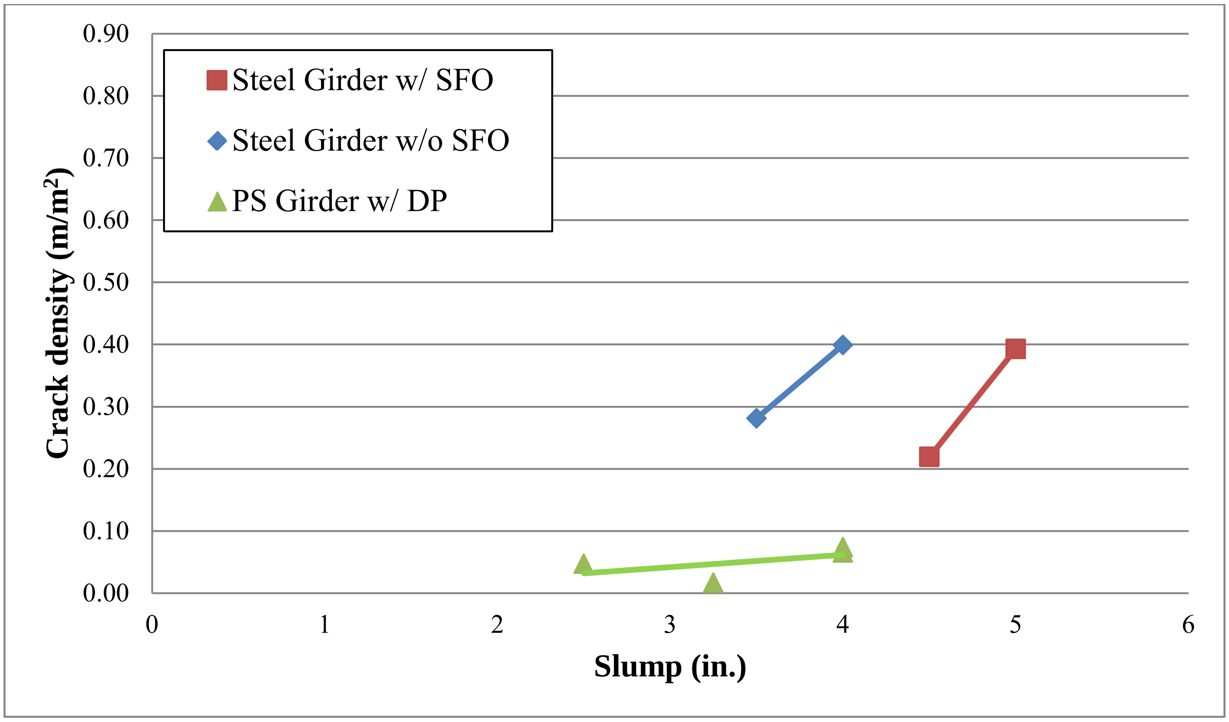


FIGURE 2.8
Crack Density at 42 Months versus Slump for US-59 Decks Supported by Steel and Prestressed Girders Constructed by Ames

2.8 Strength

Lower compressive strengths result in increased creep, which can alleviate a portion of the tensile stresses that develop in a deck (Lindquist et al. 2008). The LC-HPC bridge deck specification requires the average concrete strength at 28 day to be between 3500 to 5500 psi. The average strengths for the US-59 decks range from 4260 to 6390 psi.

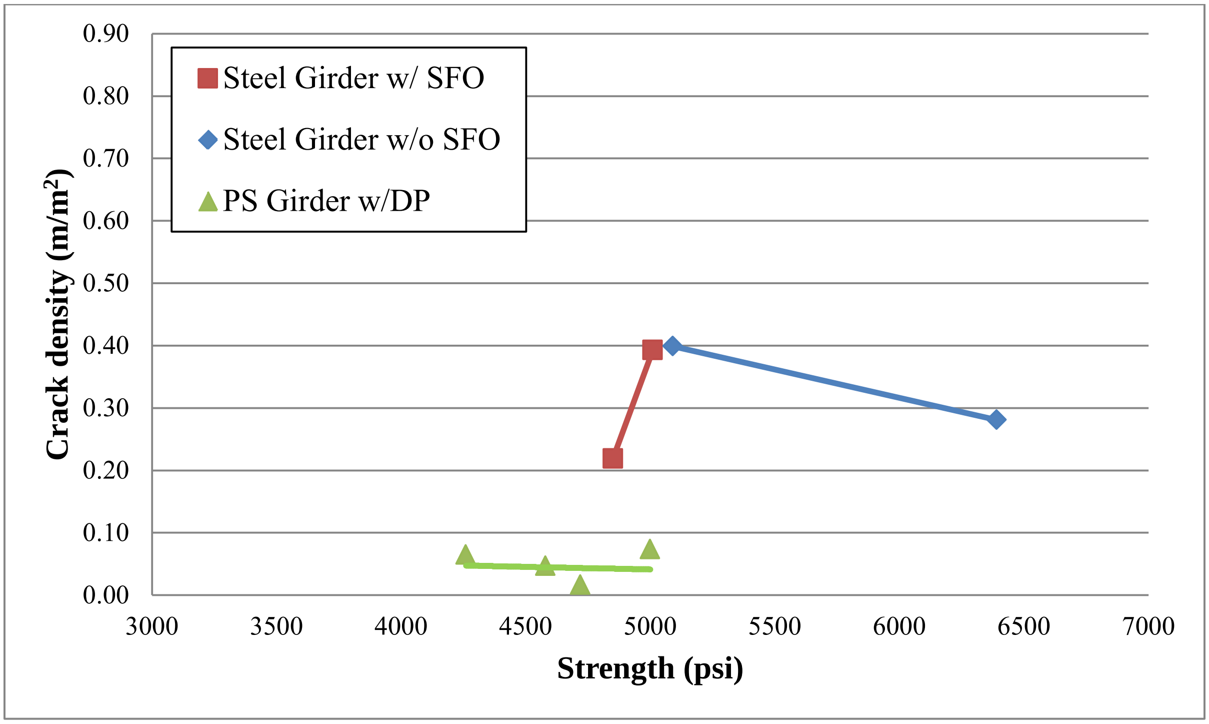


FIGURE 2.9
Crack Density versus Strength for US-59 Decks Supported by Steel and Prestressed Girders Constructed by Ames

The crack densities of US-59 decks supported by prestressed and steel girders constructed by Ames are presented as a function of strength in Figure 2.9. The decks supported by steel girders with SFO, US-59 5 and 6, exhibit an increase in crack density at 42 months (0.219 to 0.393 m/m^2) as the average strength of the subdeck increases from 4850 to 5010 psi. As discussed in the previous section, US-59 6 also had a higher slump than US-59 5. The twin decks US-59 1 and 2 supported by steel girders without a SFO exhibited a decrease in crack density (0.399 to 0.281 m/m^2) as the compressive strength increases from 5090 to 6390 psi. This observation may be due to a higher average slump 4 inches versus 3.5 inches or to other factors. For decks supported by prestressed girders with deck panels, a slight reduction in crack density is observed as the average strength increases. This observation may also be due to a higher average slump for the higher cracking decks, but overall the eight decks cast by Ames do not show a clear trend in the relationship between compressive strength and cracking.

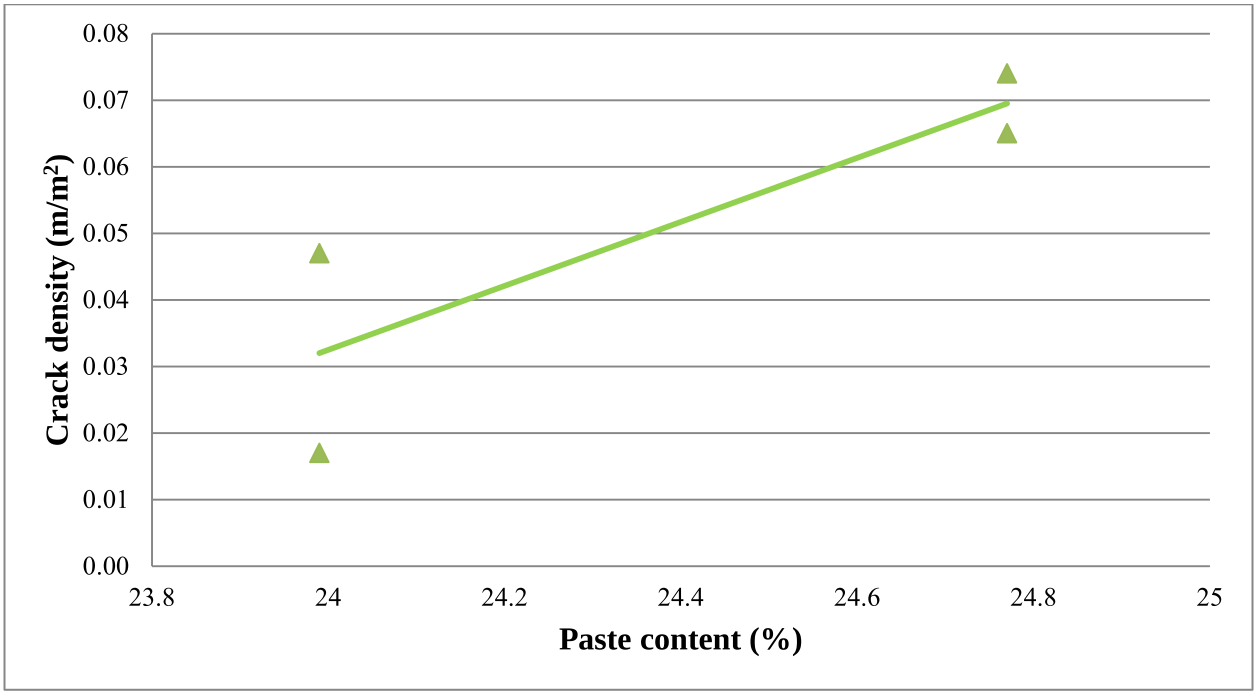


FIGURE 2.10
Crack Density at 42 Days versus Paste Content for US-59 Decks Prestressed Girders with Deck Panels

2.9 Paste Content

Shrinkage of concrete is largely influenced by its paste content (volume fraction of water and cement) in the concrete mixture because it is the constituent that undergoes the majority of the shrinkage. Past research has shown that crack density increases in bridge decks with increased paste content (Yuan et al. 2011).

The paste contents for decks supported by prestressed girders with deck panels range from 23.99 percent to 24.77 percent, as shown in Figure 2.10. A small increase in crack density is observed for these decks as paste content increases. Similar observations are not available for the other decks in this study due to the small sample size.

2.10 Cementitious Material

Three types of cementitious material combinations were used in the US-59 bridge decks; cement only; a binary mixture with 65 percent cement and 35 percent slag cement; and a ternary mixture with 60 percent cement, 35 percent slag cement, and 5 percent silica fume.

Two of the four decks supported by steel girders (US-59 1 and 2) contain a ternary mixture. The other two decks supported by steel girders (US-59 5 and 6) have silica fume overlay and a binary mixture in the subdeck. Thus, a comparison based on type of cementitious material is not possible for these decks.

Two of the four decks supported by prestressed girders with deck panels (US-59 3 and 4) contain the binary mixture and the other two decks (US-59 7 and 8) contain the ternary mixture. The US-59 decks supported by prestressed girders without deck panels (US-59 9, 10, 11 and 12) contain cement only in the mixture.

A lower crack density was observed for the US-59 decks supported by prestressed girders with deck panels containing the ternary mixture (US-59 7 and 8) than for the decks containing the binary mixture (US-59 3 and 4), as shown in the Figure 2.11. Average crack densities for decks containing the ternary and binary mixture at 42 months were 0.032 m/m^2 and 0.070 m/m^2 respectively. Both values are low. This lower crack density in decks containing the ternary mixture may be due to the lower paste content (23.99 percent versus 24.77) and average slump (3 inches versus 4 inches) compared to the decks with the binary mixture. Similar observations are not available for the other decks in this study.

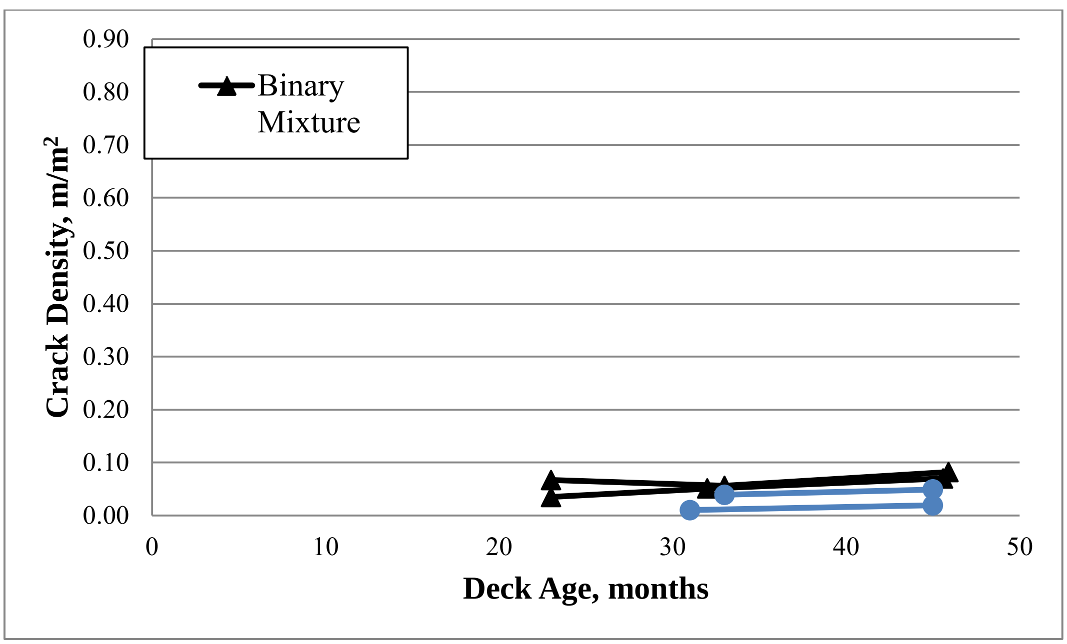


FIGURE 2.11
Crack Density versus Age for US-59 Decks with Deck Panels Supported Prestressed Girders with Different Cementitious Material

2.11 Aggregate

Two aggregate blends were used in the concrete in the US-59 bridge decks. Six of the twelve decks contain 50 percent river sand and 50 percent ½ inch crushed limestone. The other six decks contain 39.8 percent river sand, 45 percent ¾ inch crushed granite, and 15.2 percent ½ inch crushed granite.

Past research has shown that the modulus of elasticity and absorption of the aggregate affects the shrinkage of concrete (Lindquist et al. 2008). Concrete containing aggregate with a high modulus of elasticity, as indicated by low absorption and porosity (e.g., granite), exhibits less long-term shrinkage than concrete containing aggregate with a low modulus of elasticity (e.g., limestone). However, lower early age shrinkage has been observed for concrete containing coarse aggregate with higher-absorption (e.g., limestone), which may be a result of internal curing provided by the porous aggregate.

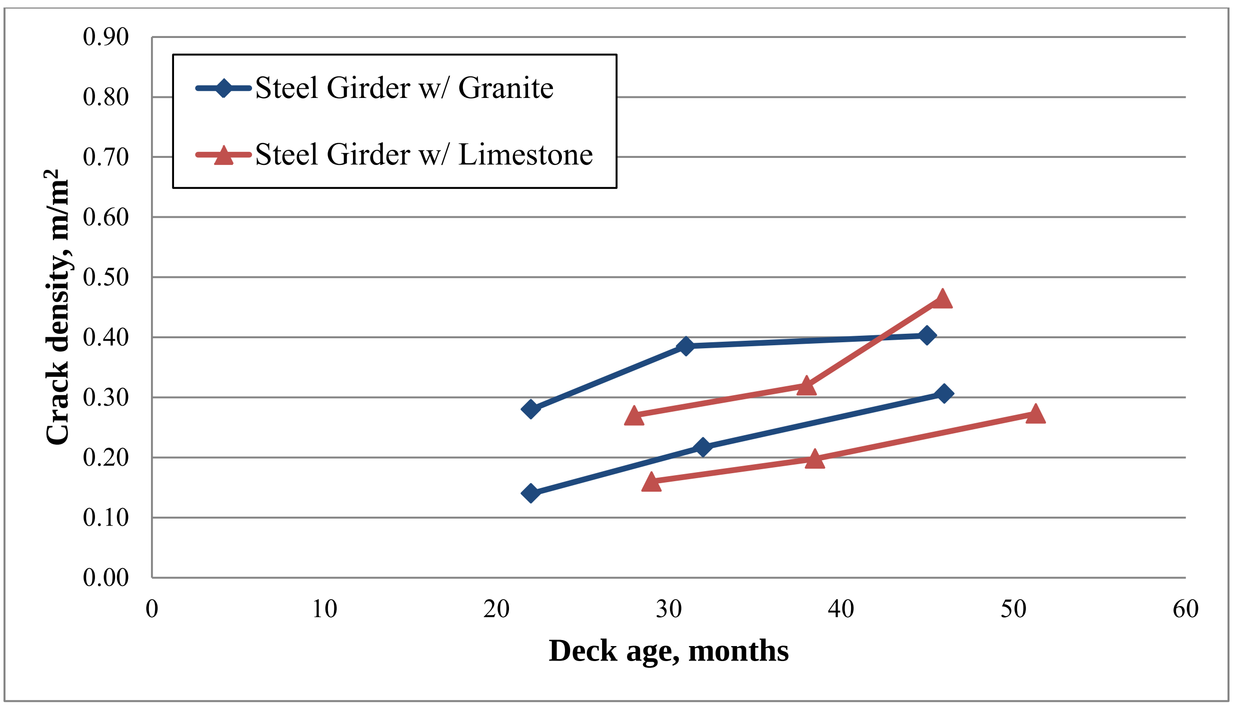


FIGURE 2.12
Crack Density versus Age of Deck for US-59 Decks Supported on Steel Girder

Of the four decks supported by steel girders, two contain the granite mixture (US-59 1 and 2) and two contain the limestone mixture (US-59 5 and 6). As shown in Figure 2.12, the crack densities of the decks with the two aggregate blends overlap, with the decks containing limestone exhibiting a lower average crack density in the initial surveys – performance that may be tied to internal curing. A reduction in the rate of crack growth, however, occurred between the second and third surveys for the two decks containing granite, while the two decks with limestone experienced an increase in the rate of crack growth. Part of the difference at later ages may be due to the higher modulus of elasticity of the granite compared to the limestone.

Similar observations are not available for the decks supported by prestressed girders with or without deck panels in this study because each deck type contains a single type of aggregate (i.e. decks with deck panels contain granite, and decks without deck panels contain limestone).

Chapter 3: Summary and Conclusions

Bridge deck crack surveys were performed on twelve bridges on US-59 to determine the effects of mixture proportions, deck type, and girder type on the crack density of reinforced concrete bridge decks. Of the twelve decks surveyed, eight are supported by prestressed concrete girders and four are supported by steel girders. Four of the decks with prestressed girders were cast on partial-depth precast deck panels, two are monolithic, and two have overlays. Of the four decks with steel girders, two have overlays and two are monolithic. One contractor, Ames, placed eight of the decks, and two other contactors, Beachner and Reece, placed two decks each. Following the surveys, crack maps were plotted and analyzed and cracking trends were observed. The results for the US-59 bridge decks are compared with crack densities obtained in a study of low-cracking high-performance concrete (LC-HPC) bridge decks.

The following conclusions are based on the results of this study.

1. Monolithic concrete bridge decks supported by prestressed concrete girders crack less than decks supported by steel girders in the first 42 months after construction, an observation that is consistent with earlier studies.
2. At an age of approximately 42 months, the US-59 decks on prestressed girders and deck panels are not displaying significant cracking. The cracks are small but transverse cracks do appear to be aligned with the joints of the deck panels.
3. The US-59 monolithic decks supported by prestressed girders with and without deck panels exhibit similar cracking performance.
4. The US-59 decks supported by prestressed girders that have overlays exhibit significantly more cracking than the decks on prestressed girders without overlays.
5. The US-59 monolithic decks supported by steel girders exhibit higher crack densities than the LC-HPC decks on s teel girders. This can be attributed to the higher compressive strengths, higher differences in concrete and air temperatures, and differences in finishing and curing methods used for the US-59 decks compared to those used for the LC-HPC decks.

6. The US-59 bridge decks with silica fume overlays supported by steel girders have lower crack densities than similar control decks from the LC-HPC study with silica fume, which may be due to the higher average concrete slump used on the control decks.
7. One of two decks supported by steel girders with an overlay has fibers in the overlay. The deck supported by steel girders containing an overlay with fibers (US-59 5) has more cracking than the deck supported by steel girders containing an overlay without fibers (US-59 6). Two monolithic decks supported by prestressed girders have fibers and both exhibit lower cracking than their twin decks with overlays. No correlation could be established between the use of fibers and cracking performance due to limited number of decks with fibers.
8. Crack density increased with an increase in concrete slump for the US-59 decks constructed by Ames supported by both steel and prestressed girders. An increase in cracking with an increase in slump is an observation that is consistent with previous deck surveys in Kansas. A similar conclusion cannot be established for decks constructed by Beachner and Reece due to limited number decks.
9. For the US-59 decks constructed by Ames, a decrease in crack density was observed with an increase in the average compressive strength of the concrete for decks with deck panels supported by prestressed girders and decks without a SFO supported by steel girders. In contrast, an increase in crack density was observed with increases in average strength of concrete for decks supported by steel girders with a SFO. The small sample size precludes any general conclusion for the 12 decks in this survey.
10. An increase in crack density for the US-59 decks supported by prestressed girders with deck panels was observed with an increase in paste content, which is consistent with earlier studies. Correlations are not possible for the other decks in this survey due to the small sample size and the dissimilarity between decks.
11. In US-59 decks supported by prestressed girders with deck panels, decks with the ternary mixture have lower crack densities compared to decks with the binary

mixture. Higher cracking in decks with the binary mixture may be due to higher slump and paste contents.

12. For decks supported by steel girders, lower initial cracking followed by an increase in the rate of crack growth was observed for decks containing limestone compared to decks containing granite in the first 42 months. The lower initial cracking may be due to the ability of the porous limestone to provide internal curing. The increase in the rate of crack growth may be due to the higher long-term shrinkage resulting from the lower modulus of elasticity of the limestone compared to the granite.

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Appendix A: Bridge Deck Survey Specification*

*From Lindquist et al. (2005)

A1: Bridge Deck Survey Specification

1.0 DESCRIPTION.

This specification covers the procedures and requirements to perform bridge deck surveys of reinforced concrete bridge decks.

2.0 SURVEY REQUIREMENTS.

a. Pre-Survey Preparation.

(1) Prior to performing the crack survey, related construction documents need to be gathered to produce a scaled drawing of the bridge deck. The scale must be exactly 1 in. = 10 ft (for use with the scanning software), and the drawing only needs to include the boundaries of the deck surface.

NOTE 1 – In the event that it is not possible to produce a scaled drawing prior to arriving at the bridge deck, a hand-drawn crack map (1 in.= 10 ft) created on engineering paper using measurements taken in the field is acceptable.

(2) The scaled drawing should also include compass and traffic directions in addition to deck stationing. A scaled 5 ft by 5 ft grid is also required to aid in transferring the cracks observed on the bridge deck to the scaled drawing. The grid shall be drawn separately and attached to the underside of the crack map such that the grid can easily be seen through the crack map.

NOTE 2 – Maps created in the field on engineering paper need not include an additional grid.

(3) For curved bridges, the scaled drawing need not be curved, i.e., the curve may be approximated using straight lines.

(4) Coordinate with traffic control so that at least one side (or one lane) of the bridge can be closed during the time that the crack survey is being performed.

b. Preparation of Surface.

(1) After traffic has been closed, station the bridge in the longitudinal direction at ten feet intervals. The stationing shall be done as close to the centerline as possible. For curved bridges, the stationing shall follow the curve.

(2) Prior to beginning the crack survey, mark a 5 ft by 5 ft grid using lumber crayons or chalk on the portion of the bridge closed to traffic corresponding to the grid on the scaled drawing. Measure and document any drains, repaired areas, unusual cracking, or any other items of interest.

(3) Starting with one end of the closed portion of the deck, using a lumber crayon or chalk, begin tracing cracks that can be seen while bending at the waist. After beginning to trace cracks, continue to the end of the crack, even if this includes portions of the crack that were not initially seen while bending at the waist. Areas covered by sand or other debris need not be surveyed. Trace the cracks using a different color crayon than was used to mark the grid and stationing.

(4) At least one person shall recheck the marked portion of the deck for any additional cracks. The goal is not to mark every crack on the deck, only those cracks that can initially be seen while bending at the waist.

NOTE 3 – An adequate supply of lumber crayons or chalk should be on hand for the survey. Crayon or chalk colors should be selected to be readily visible when used to mark the concrete.

c. Weather Limitations.

(1) Surveys are limited to days when the expected temperature during the survey will not be below 60°F.

(2) Surveys are further limited to days that are forecasted to be at least mostly sunny for a majority of the day.

(3) Regardless of the weather conditions, the bridge deck must be completely dry before the survey can begin.

3.0 BRIDGE SURVEY.

a. Crack Surveys.

Using the grid as a guide, transfer the cracks from the deck to the scaled drawing. Areas that are not surveyed should be marked on the scaled drawing. Spalls, regions of scaling, and other areas of special interest need not be included on the scale drawings but should be noted.

b. Delamination Survey.

At any time during or after the crack survey, bridge decks shall be checked for delamination. Any areas of delamination shall be noted and drawn on a separate drawing of the bridge. This second drawing need not be to scale.

c. Under Deck Survey.

Following the crack and delamination survey, the underside of the deck shall be examined and any unusual or excessive cracking noted.

Appendix B: LC-HPC Specifications*

*From Yuan et al. (2011)

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION TO THE

STANDARD SPECIFICATIONS, 2007 EDITION

Add a new SECTION to DIVISION 1100:

LOW-CRACKING HIGH-PERFORMANCE CONCRETE – AGGREGATES

1.0 DESCRIPTION

This specification is for coarse aggregates, fine aggregates, and mixed aggregates (both coarse and fine material) for use in bridge deck construction.

2.0 REQUIREMENTS

a. Coarse Aggregates for Concrete.

(1) Composition. Provide coarse aggregate that is crushed or uncrushed gravel, chat, or crushed stone. (Consider calcite cemented sandstone, rhyolite, basalt and granite as crushed stone

(2) Quality. The quality requirements for coarse aggregate for bridge decks are in **TABLE 1-1**:

TABLE 1-1: QUALITY REQUIREMENTS FOR COARSE AGGREGATES FOR BRIDGE DECK				
Concrete Classification	**Soundness (min.)**	**Wear (max.)**	**Absorption (max.)**	**Acid Insol. (min.)**
Grade 3.5 (AE) (LC-HPC) [1]	0.90	40	0.7	55

[1] Grade 3.5 (AE) (LC-HPC) – Bridge Deck concrete with select coarse aggregate for wear and acid insolubility.

(3) Product Control.

(a) Deleterious Substances. Maximum allowed deleterious substances by weight are:

- Material passing the No. 200 sieve (KT-2)......................... 2.5%
- Shale or Shale-like material (KT-8)................................... 0.5%
- Clay lumps and friable particles (KT-7) 1.0%
- Sticks (wet) (KT-35) 0.1%
- Coal (AASHTO T 113).. 0.5%

(b) Uniformity of Supply. Designate or determine the fineness modulus (grading factor) according to the procedure listed in the Construction Manual Part V, Section 17 be fore delivery, or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

(4) Do not combine siliceous fine aggregate with siliceous coarse aggregate if neither meet the requirements of **subsection 2.0c.(2)(a)**. Consider such fine material, regardless of proportioning, as a Basic Aggregate that must conform to **subsection 2.0c**.

(5) Handling Coarse Aggregates.

(a) Segregation. Before acceptance testing, remix all aggregate segregated by transportation or stockpiling operations.

(b) Stockpiling.

- Stockpile accepted aggregates in layers 3 to 5 feet thick. Berm each layer so that aggregates do not "cone" down into lower layers.
- Keep aggregates from different sources, with different gradings, or with a significantly different specific gravity separated.
- Transport aggregate in a manner that insures uniform gradation.
- Do not use aggregates that have become mixed with earth or foreign material.
- Stockpile or bin all washed aggregate produced or handled by hydraulic methods for 12 hour s (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
- Provide additional stockpiling or binning in cases of high or non-uniform moisture.

b. Fine Aggregates for Basic Aggregate in MA for Concrete.

(1) Composition.

(a) Type FA-A. Provide either singly or in combination natural occurring sand resulting from the disintegration of siliceous or calcareous rock, or manufactured sand produced by crushing predominately siliceous materials.

(b) Type FA-B. Provide fine granular particles resulting from the crushing of zinc and lead ores (Chat).

(2) Quality.

(a) Mortar strength and Organic Impurities. If the District Materials Engineer determines it is necessary, because of unknown characteristics of new sources or changes in existing sources, provide fine aggregates that comply with these requirements:

- Mortar Strength (Mortar Strength Test, KTMR-26). Compressive strength when combined with Type III (high early strength) cement:
  - At age 24 hours, minimum…………..100%*
  - At age 72 hours, minimum…………..100%*

  *Compared to strengths of specimens of the same proportions, consistency, cement and standard 20-30 Ottawa sand.
- Organic Impurities (Organic Impurities in Fine Aggregate for Concrete Test, AASHTO T 21). The color of the supernatant liquid is equal to or lighter than the reference standard solution.

(b) Hardening characteristics. Specimens made of a mixture of 3 parts FA-B and 1 part cement with sufficient water for molding will harden within 24 hours. There is no hardening requirement for FA-A.

(3) Product Control.

(a) Deleterious Substances.

- Type FA-A: Maximum allowed deleterious substances by weight are:
  - Material passing the No. 200 sieve (KT-2)………..……………. 2.0%
  - Shale or Shale-like material (KT-8) ……………………………. 0.5%
  - Clay lumps and friable particles (KT-7)………..………………. 1.0%
  - Sticks (wet) (KT-35)………………………...…………..…… 0.1%
- Type FA-B: Provide materials that are free of organic impurities, sulfates, carbonates, or alkali. Maximum allowed deleterious substances by weight are:
  - Material passing the No. 200 sieve (KT-2)…………….…....... 2.0%
  - Clay lumps & friable particles (KT-7)…………………………. 0.25%

(c) Uniformity of Supply. Designate or determine the fineness modulus (grading factor) according to the procedure listed in the Construction Manual Part V, Section 17 before delivery, or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

(4) Proportioning of Coarse and Fine Aggregate. Use a proven optimization method such as the Shilstone Method or the KU Mix Method.

Do not combine siliceous fine aggregate with siliceous coarse aggregate if neither meet the requirements of **subsection 2.0c.(2)(a)**. Consider such fine material, regardless of proportioning, as a Basic Aggregate and must conform to the requirements in **subsection 2.0c**.

(5) Handling and Stockpiling Fine Aggregates.

- Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.
- Transport aggregate in a manner that insures uniform grading.
- Do not use aggregates that have become mixed with earth or foreign material.
- Stockpile or bin all washed aggregate produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
- Provide additional stockpiling or binning in cases of high or non-uniform moisture.

c. Mixed Aggregates for Concrete.

(1) Composition.

(a) Total Mixed Aggregate (TMA). A natural occurring, predominately siliceous aggregate from a single source that meets the Wetting & Drying Test (KTMR-23) and grading requirements.

(b) Mixed Aggregate. A combination of basic and coarse aggregates that meet **TABLE 1-2**.

- Basic Aggregate (BA). Singly or in combination, a natural occurring, predominately siliceous aggregate that does not meet the grading requirements of Total Mixed Aggregate.

(c) Coarse Aggregate. Granite, crushed sandstone, chat, and gravel. Gravel that is not approved under **subsection 2.0c.(2)** may be used, but only with basic aggregate that meets the wetting and drying requirements of TMA.

(2) Quality.

(a) Total Mixed Aggregate.

- Soundness, minimum (KTMR-21) …….…………0.90
- Wear, maximum (KTMR-25) ……………….……50%
- Wetting and Drying Test (KTMR-23) for Total Mixed Aggregate
  Concrete Modulus of Rupture:
  - *At 60 days, minimum………………………….550 psi*
  - *At 365 days, minimum….…………….………550 psi*

  Expansion:
  - *At 180 days, maximum……………….………0.050%*
  - At 365 days, maximum……………………..……0.070%

  Aggregates produced from the following general areas are exempt from the Wetting and Drying Test:
  - **Blue River Drainage Area.**
  - **The Arkansas River from Sterling, west to the Colorado state line.**
  - **The Neosho River from Emporia to the Oklahoma state line.**

(b) Basic Aggregate.

- Retain 10% or more of the BA on the No. 8 sieve before adding the Coarse Aggregate. Aggregate with less than 10% retained on the No. 8 sieve is to be considered a Fine Aggregate described in **subsection 2.0b**. Provide material with less than 5% calcareous material retained on the ⅜" sieve.
- Soundness, minimum (KTMR-21)……………….0.90
- Wear, maximum (KTMR-25)…………………..…50%
- Mortar strength and Organic Impurities. If the District Materials Engineer determines it is necessary, because of unknown characteristics of new sources or changes in existing sources, provide mixed aggregates that comply with these requirements:
  - Mortar Strength (Mortar Strength Test, KTMR-26). Compressive strength when combined with Type III (high early strength) cement:
    - At age 24 hours, minimum…………..100%*
    - At age 72 hours, minimum…………..100%*
      *Compared to strengths of specimens of the same proportions, consistency, cement and standard 20-30 Ottawa sand.
  - Organic Impurities (Organic Impurities in Fine Aggregate for Concrete Test, AASHTO T 21). The color of the supernatant liquid is equal to or lighter than the reference standard solution.

(3) Product Control.

(a) Size Requirement. Provide mixed aggregates that comply with the grading requirements in **TABLE 1-2**.

TABLE 1-2: GRADING REQUIREMENTS FOR MIXED AGGREGATES FOR CONCRETE BRIDGE DECKS												
Type	**Usage**	**Percent Retained on Individual Sieves - Square Mesh Sieves**										
		1½"	**1"**	**3/4"**	**1/2"**	**3/8"**	**No. 4**	**No. 8**	**No. 16**	**No. 30**	**No. 50**	**No. 100**
MA-4	Optimized for LC-HPC Bridge Decks*	0	2-6	5-18	8-18	8-18	8-18	8-18	8-18	8-15	5-15	0-10

*Use a proven optimization method, such as the Shilstone Method or the KU Mix Method.
Note: Manufactured sands used to obtain optimum gradations have caused difficulties in pumping, placing or finishing. Natural coarse sands and pea gravels used to obtain optimum gradations have worked well in concretes that were pumped.

(b) Deleterious Substances. Maximum allowed deleterious substances by weight are:

- Material passing the No. 200 sieve (KT-2)……………..….. 2.5%
- Shale or Shale-like material (KT-8)…………………..……. 0.5%
- Clay lumps and friable particles (KT-7)…………………… 1.0%
- Sticks (wet) (KT-35)………………………..…………… 0.1%
- Coal (AASHTO T 113)…..……………………..………. 0.5%

(c) Uniformity of Supply. Designate or determine the fineness modulus (grading factor) according to the procedure listed in the Construction Manual Part V, Section 17 be fore delivery, or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

(4) Handling Mixed Aggregates.

(a) Segregation. Before acceptance testing, remix all aggregate segregated by transit or stockpiling.

(b) Stockpiling.

- Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.
- Transport aggregate in a manner that insures uniform grading.
- Do not use aggregates that have become mixed with earth or foreign material.
- Stockpile or bin all washed aggregate produced or handled by hydraulic methods for 12 hour s (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
- Provide additional stockpiling or binning in cases of high or non-uniform moisture.

3.0 TEST METHODS

Test aggregates according to the applicable provisions of **SECTION 1117**.

4.0 PREQUALIFICATION

Aggregates for concrete must be prequalified according to **subsection 1101.2**.

5.0 BASIS OF ACCEPTANCE

The Engineer will accept aggregates for concrete base on the prequalification required by this specification, and **subsection 1101.4**.

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION TO THE
STANDARD SPECIFICATIONS 2007 EDITION

Add a new SECTION to DIVISION 400:

LOW-CRACKING HIGH-PERFORMANCE CONCRETE

1.0 DESCRIPTION

Provide the grades of low-cracking high-performance concrete (LC-HPC) specified in the Contract Documents.

2.0 MATERIALS

Coarse, Fine & Mixed Aggregate ....**07-PS0165, latest version**
Admixtures...**DIVISION 1400**
Cement ....**DIVISION 2000**
Water ....**DIVISION 2400**

3.0 CONCRETE MIX DESIGN

a. General. Design the concrete mixes specified in the Contract Documents.

Provide aggregate gradations that comply with **07-PS0165, latest version** and Contract Documents.

If desired, contact the DME for available information to help determine approximate proportions to produce concrete having the required characteristics on the project.

Take full responsibility for the actual proportions of the concrete mix, even if the Engineer assists in the design of the concrete mix.

Submit all concrete mix designs to the Engineer for review and approval. Submit completed volumetric mix designs on KDOT Form No. 694 (or other forms approved by the DME).

Do not place any concrete on the project until the Engineer approves the concrete mix designs. Once the Engineer approves the concrete mix design, do not make changes without the Engineer's approval.

Design concrete mixes that comply with these requirements:

b. Air-Entrained Concrete for Bridge Decks. Design air-entrained concrete for structures according to **TABLE 1-1**.

TABLE 1-1: AIR ENTRAINED CONCRETE FOR BRIDGE DECKS				
Grade of Concrete Type of Aggregate (SECTION 1100)	**lb of Cementitious per cu yd of Concrete, min/max**	**lb of Water per lb of Cementitious***	**Designated Air Content Percent by Volume****	**Specified 28-day Compressive Strength Range, psi**
Grade 3.5 (AE) (LC-HPC)				
MA-4	500 / 540	0.44 – 0.45	8.0 ± 1.0	3500 – 5500

*Limits of lb. of water per lb. of cementitious. Includes free water in aggregates, but excludes water of absorption of the aggregates. With approval of the Engineer, may be decreased to 0.43 on-site.

**Concrete with an air content less than 6.5% or greater than 9.5% shall be rejected. The Engineer will sample concrete for tests at the discharge end of the conveyor, bucket or if pumped, the piping.

c. Portland Cement. Select the type of portland cement specified in the Contract Documents. Mineral admixtures are prohibited for Grade 3.5 (AE) (LC-HPC) concrete.

d. Design Air Content. Use the middle of the specified air content range for the design of air-entrained concrete.

e. Admixtures for Air-Entrainment and Water Reduction. Verify that the admixtures used are compatible and will work as intended without detrimental effects. Use the dosages recommended by the admixture manufacturers to determine the quantity of each admixture for the concrete mix design. Incorporate and mix the admixtures into the concrete mixtures according to the manufacturer's recommendations.

Set retarding or accelerating admixtures are prohibited for use in Grade 3.5 (AE) (LC-HPC) concrete. These include Type B, C, D, E, and G chemical admixtures as defined by ASTM C 494/C 494M – 08. Do not use admixtures containing chloride ion (CL) in excess of 0.1 percent by mass of the admixture in Grade 3.5 (AE) (LC-HPC) concrete.

(1) Air-Entraining Admixture. If specified, use an air-entraining admixture in the concrete mixture. If another admixture is added to an air-entrained concrete mixture, determine if it is necessary to adjust the air-entraining admixture dosage to maintain the specified air content. Use only a vinsol resin or tall oil based air-entraining admixture.

(2)Water-Reducing Admixture. Use a Type A water reducer or a dual rated Type A water reducer – Type F high-range water reducer, when necessary to obtain compliance with the specified fresh and hardened concrete properties.

Include a batching sequence in the concrete mix design. Consider the location of the concrete plant in relation to the job site, and identify the approximate quantity, when and at what location the water-reducing admixture is added to the concrete mixture.

The manufacturer may recommend mixing revolutions beyond the limits specified in **subsection 5.0**. If necessary and with the approval of the Engineer, address the additional mixing revolutions (the Engineer will allow up to 60 additional revolutions) in the concrete mix design.

Slump control may be accomplished in the field only by redosing with a water-reducing admixture. If time and temperature limits are not exceeded, and if at least 30 mixing revolutions remain, the Engineer will allow redosing with up to 50% of the original dose.

(3) Adjust the mix designs during the course of the work when necessary to achieve compliance with the specified fresh and hardened concrete properties. Only permit such modifications after trial batches to demonstrate that the adjusted mix design will result in concrete that complies with the specified concrete properties.

The Engineer will allow adjustments to the dose rate of air entraining and water-reducing chemical admixtures to compensate for environmental changes during placement without a new concrete mix design or qualification batch.

f. Designated Slump. Designate a slump for each concrete mix design within the limits in **TABLE 1-2**.

*TABLE 1-2: DESIGNATED SLUMP**

Type of Work	*Designated Slump* **(inches)**
Grade 3.5 (AE) (LC-HPC)	1 ½ - 3

*The Engineer will obtain sample concrete at the discharge end of the conveyor, bucket or if pumped, the piping.

If potential problems are apparent at the discharge of any truck, and the concrete is tested at the truck discharge (according to **subsection 6.0**), the Engineer will reject concrete with a slump greater than 3 ½ inches at the truck discharge, 3 inches if being placed by a bucket.

4.0 REQUIREMENTS FOR COMBINED MATERIALS

a. Measurements for Proportioning Materials.

(1) Cement. Measure cement as packed by the manufacturer. A sack of cement is considered as 0.04 cubic yards weighing 94 pounds net. Measure bulk cement by weight. In either case, the measurement must be accurate to within 0.5% throughout the range of use.

(2) Water. Measure the mixing water by weight or volume. In either case, the measurement must be accurate to within 1% throughout the range of use.

(3) Aggregates. Measure the aggregates by weight. The measurement must be accurate to within 0.5% throughout the range of use.

(4) Admixtures. Measure liquid admixtures by weight or volume. If liquid admixtures are used in small quantities in proportion to the cement as in the case of air-entraining agents, use readily adjustable mechanical dispensing equipment capable of being set to deliver the required

quantity and to cut off the flow automatically when this quantity is discharged. The measurement must be accurate to within 3% of the quantity required.

b. Testing of Aggregates. Testing Aggregates at the Batch Site. Provide the Engineer with reasonable facilities at the batch site for obtaining samples of the aggregates. Provide adequate and safe laboratory facilities at the batch site allowing the Engineer to test the aggregates for compliance with the specified requirements.

KDOT will sample and test aggregates from each source to determine their compliance with specifications. Do not batch the concrete mixture until the Engineer has determined that the aggregates comply with the specifications. KDOT will conduct sampling at the batching site, and test samples according to the Sampling and Testing Frequency Chart in Part V. For QC/QA Contracts, establish testing intervals within the specified minimum frequency.

After initial testing is complete and the Engineer has determined that the aggregate process control is satisfactory, use the aggregates concurrently with sampling and testing as long as tests indicate compliance with specifications. When batching, sample the aggregates as near the point of batching as feasible. Sample from the stream as the storage bins or weigh hoppers are loaded. If samples can not be taken from the stream, take them from approved stockpiles, or use a template and sample from the conveyor belt. If test results indicate an aggregate does not comply with specifications, cease concrete production using that aggregate. Unless a tested and approved stockpile for that aggregate is available at the batch plant, do not use any additional aggregate from that source and specified grading until subsequent sampling and testing of that aggregate indicate compliance with specifications. When tests are completed and the Engineer is satisfied that process control is again adequate, production of concrete using aggregates tested concurrently with production may resume.

c. Handling of Materials.

(1) Aggregate Stockpiles. Approved stockpiles are permitted only at the batch plant and only for small concrete placements or for the purpose of maintaining concrete production. Mark the approved stockpile with an “Approved Materials” sign. Provide a suitable stockpile area at the batch plant so that aggregates are stored without detrimental segregation or contamination. At the plant, limit stockpiles of tested and approved coarse aggregate and fine aggregate to 250 tons each, unless approved for more by the Engineer. If mixed aggregate is used, limit the approved stockpile to 500 tons, the size of each being proportional to the amount of each aggregate to be used in the mix.

Load aggregates into the mixer so no material foreign to the concrete or material capable of changing the desired proportions is included. When 2 or more sizes or types of coarse or fine aggregates are used on the same project, only 1 size or type of each aggregate may be used for any one continuous concrete placement.

(2) Segregation. Do not use segregated aggregates. Previously segregated materials may be thoroughly re-mixed and used when representative samples taken anywhere in the stockpile indicated a uniform gradation exists.

(3) Cement. Protect cement in storage or stockpiled on the site from any damage by climatic conditions which would change the characteristics or usability of the material.

(4) Moisture. Provide aggregate with a moisture content of ± 0.5% from the average of that day. If the moisture content in the aggregate varies by more than the above tolerance, take whatever corrective measures are necessary to bring the moisture to a constant and uniform consistency before placing concrete. This may be accomplished by handling or manipulating the

stockpiles to reduce the moisture content, or by adding moisture to the stockpiles in a manner producing uniform moisture content through all portions of the stockpile.

For plants equipped with an approved accurate moisture-determining device capable of determining the free moisture in the aggregates, and provisions made for batch to batch correction of the amount of water and the weight of aggregates added, the requirements relative to manipulating the stockpiles for moisture control will be waived. Any procedure used will not relieve the producer of the responsibility for delivery of concrete meeting the specified water-cement ratio and slump requirements.

Do not use aggregate in the form of frozen lumps in the manufacture of concrete.

(5) Separation of Materials in Tested and Approved Stockpiles. Only use KDOT Approved Materials. Provide separate means for storing materials approved by KDOT. If the producer elects to use KDOT Approved Materials for non-KDOT work, during the progress of a project requiring KDOT Approved Materials, inform the Engineer and agree to pay all costs for additional materials testing.

Clean all conveyors, bins and hoppers of unapproved materials before beginning the manufacture of concrete for KDOT work.

5.0 MIXING, DELIVERY, AND PLACEMENT LIMITATIONS

a. Concrete Batching, Mixing, and Delivery. Batch and mix the concrete in a central-mix plant, in a truck mixer, or in a drum mixer at the work site. Provide plant capacity and delivery capacity sufficient to maintain continuous delivery at the rate required. The delivery rate of concrete during concreting operations must provide for the proper handling, placing and finishing of the concrete.

Seek the Engineer's approval of the concrete plant/batch site before any concrete is produced for the project. The Engineer will inspect the equipment, the method of storing and handling of materials, the production procedures, and the transportation and rate of delivery of concrete from the plant to the point of use. The Engineer will grant approval of the concrete plant/batch site based on compliance with the specified requirements. The Engineer may, at any time, rescind permission to use concrete from a previously approved concrete plant/batch site upon failure to comply with the specified requirements.

Clean the mixing drum before it is charged with the concrete mixture. Charge the batch into the mixing drum so that a portion of the water is in the drum before the aggregates and cementitious. Uniformly flow materials into the drum throughout the batching operation. Add all mixing water in the drum by the end of the first 15 seconds of the mixing cycle. Keep the throat of the drum free of accumulations that restrict the flow of materials into the drum.

Do not exceed the rated capacity (cubic yards shown on the manufacturer's plate on the mixer) of the mixer when batching the concrete. The Engineer will allow an overload of up to 10% above the rated capacity for central-mix plants and drum mixers at the work site, provided the concrete test data for strength, segregation and uniform consistency are satisfactory, and no concrete is spilled during the mixing cycle.

Operate the mixing drum at the speed specified by the mixer's manufacturer (shown on the manufacturer's plate on the mixer).

Mixing time is measured from the time all materials, except water, are in the drum. If it is necessary to increase the mixing time to obtain the specified percent of air in air-entrained concrete, the Engineer will determine the mixing time.

If the concrete is mixed in a central-mix plant or a drum mixer at the work site, mix the batch between 1 to 5 minutes at mixing speed. Do not exceed the maximum total 60 mixing revolutions. Mixing time begins after all materials, except water, are in the drum, and ends when the discharge chute opens. Transfer time in multiple drum mixers is included in mixing time. Mix time may be reduced for plants utilizing high performance mixing drums provided thoroughly mixed and uniform concrete is being produced with the proposed mix time. Performance of the plant must comply with Table A1.1, of ASTM C 94, Standard Specification for Ready Mixed Concrete. Five of the six tests listed in Table A1.1 must be within the limits of the specification to indicate that uniform concrete is being produced.

If the concrete is mixed in a truck mixer, mix the batch between 70 and 100 revolutions of the drum or blades at mixing speed. After the mixing is completed, set the truck mixer drum at agitating speed. Unless the mixing unit is equipped with an accurate device indicating and controlling the number of revolutions at mixing speed, perform the mixing at the batch plant and operate the mixing unit at agitating speed while traveling from the plant to the work site. Do not exceed 350 total revolutions (mixing and agitating).

If a truck mixer or truck agitator is used to transport concrete that was completely mixed in a stationary central mixer, agitate the concrete while transporting at the agitating speed specified by the manufacturer of the equipment (shown on the manufacturer's plate on the equipment). Do not exceed 250 total revolutions (additional re-mixing and agitating).

Provide a batch slip including batch weights of every constituent of the concrete and time for each batch of concrete delivered at the work site, issued at the batching plant that bears the time of charging of the mixer drum with cementitious and aggregates. Include quantities, type, product name and manufacturer of all admixtures on the batch ticket.

If non-agitating equipment is used for transportation of concrete, provide approved covers for protection against the weather when required by the Engineer.

Place non-agitated concrete within 30 minutes of adding the cement to the water.

Do not use concrete that has developed its initial set. Regardless of the speed of delivery and placement, the Engineer will suspend the concreting operations until corrective measures are taken if there is evidence that the concrete can not be adequately consolidated.

Adding water to concrete after the initial mixing is prohibited. Add all water at the plant. If needed, adjust slump through the addition of a water reducer according to **subsection 3.0e.(2)**.

b. Placement Limitations.

(1) Concrete Temperature. Unless otherwise authorized by the Engineer, the temperature of the mixed concrete immediately before placement is a minimum of 55°F, and a maximum of 70°F. With approval by the Engineer, the temperature of the concrete may be adjusted 5°F above or below this range.

(2) Qualification Batch. For Grade 3.5 (AE) (LC-HPC) concrete, qualify a field batch (one truckload or at least 6 cubic yards) at least 35 days prior to commencement of placement of the bridge decks. Produce the qualification batch from the same plant that will supply the job concrete. Simulate haul time to the jobsite prior to discharge of the concrete for testing. Prior to placing concrete in the qualification slab and on the job, submit documentation to the Engineer verifying that the qualification batch concrete meets the requirements for air content, slump, temperature of plastic concrete, compressive strength, unit weight and other testing as required by the Engineer.

Before the concrete mixture with plasticizing admixture is used on the project, determine the air content of the qualification batch. Monitor the slump, air content, temperature and workability at initial batching and estimated time of concrete placement. If these properties are not adequate, repeat the qualification batch until it can be demonstrated that the mix is within acceptable limits as specified in this specification.

(3) Placing Concrete at Night. Do not mix, place or finish concrete without sufficient natural light, unless an adequate and artificial lighting system approved by the Engineer is provided.

(4) Placing Concrete in Cold Weather. Unless authorized otherwise by the Engineer, mixing and concreting operations shall not proceed once the descending ambient air temperature reaches 40°F, and may not be initiated until an ascending ambient air temperature reaches 40°F. The ascending ambient air temperature for initiating concreting operations shall increase to 45°F if the maximum ambient air temperature is expected to be between 55°F and 60°F during or within 24 hours of placement and to 50°F if the ambient air temperature is expected to equal or exceed 60°F during or within 24 hours of placement.

If the Engineer permits placing concrete during cold weather, aggregates may be heated by either steam or dry heat before placing them in the mixer. Use an apparatus that heats the weight uniformly and is so arranged as to preclude the possible occurrence of overheated areas which might injure the materials. Do not heat aggregates directly by gas or oil flame or on sheet metal over fire. Aggregates that are heated in bins, by steam-coil or water-coil heating, or by other methods not detrimental to the aggregates may be used. The use of live steam on or through binned aggregates is prohibited. Unless otherwise authorized, maintain the temperature of the mixed concrete between 55°F to 70°F at the time of placing it in the forms. With approval by the Engineer, the temperature of the concrete may be adjusted up to 5°F above or below this range. Do not place concrete when there is a probability of air temperatures being more than 25°F below the temperature of the concrete during the first 24 hours after placement unless insulation is provided for both the deck and the girders. Do not, under any circumstances, continue concrete operations if the ambient air temperature is less than 20°F.

If the ambient air temperature is 40°F or less at the time the concrete is placed, the Engineer may permit the water and the aggregates be heated to at least 70°F, but not more than 120°F.

Do not place concrete on frozen subgrade or use frozen aggregates in the concrete.

(5) Placing Concrete in Hot Weather. When the ambient temperature is above 90°F, cool the forms, reinforcing steel, steel beam flanges, and other surfaces which will come in contact with the mix to below 90°F by means of a water spray or other approved methods. For Grade 3.5 (AE) (LC-HPC) concrete, cool the concrete mixture to maintain the temperature immediately before placement between 55°F and 70°F. With approval by the Engineer, the temperature of the concrete may be up to 5°F below or above this range.

Maintain the temperature of the concrete at time of placement within the specified temperature range by any combination of the following:

- Shading the materials storage areas or the production equipment.
- Cooling the aggregates by sprinkling with potable water.
- Cooling the aggregates or water by refrigeration or replacing a portion or all of the mix water with ice that is flaked or crushed to the extent that the ice will completely melt during mixing of the concrete.
- Liquid nitrogen injection.

6.0 INSPECTION AND TESTING

The Engineer will test the first truckload of concrete by obtaining a sample of fresh concrete at truck discharge and by obtaining a sample of fresh concrete at the discharge end of the conveyor, bucket or if pumped, the piping. The Engineer will obtain subsequent sample concrete for tests at the discharge end of the conveyor, bucket or if pumped, the discharge end of the piping. If potential problems are apparent at the discharge of any truck, the Engineer will test the concrete at truck discharge prior to deposit on the bridge deck.

The Engineer will cast, store, and test strength test specimens in sets of 5. See **TABLE 1-3**.KDOT will conduct the sampling and test the samples according to **SECTION 2500** and **TABLE1-3**. The Contractor may be directed by the Engineer to assist KDOT in obtaining the fresh concrete samples during the placement operation.

A plan will be finalized prior to the construction date as to how out-of-specification concrete will be handled.

TABLE 1-3: SAMPLING AND TESTING FREQUENCY CHART

Tests Required (Record to)	Test Method	CMS	Verification Samples and Tests	Acceptance Samples and Tests
Slump (0.25 inch)	KT-21	a	Each of first 3 truckloads for any individual placement, then 1 o f every 3 truckloads	
Temperature (1°F)	KT-17	a	Every truckload, measured at the truck discharge, and from each sample made for slump determination.	
Mass (0.1 lb)	KT-20	a	One of every 6 truckloads	
Air Content (0.25%)	KT-18 or KT-19	a	Each of first 3 truckloads for any individual placement, then 1 o f every 6 truckloads	

TABLE 1-3: SAMPLING AND TESTING FREQUENCY CHART				
Tests Required (Record to)	**Test Method**	**CMS**	**Verification Samples and Tests**	**Acceptance Samples and Tests**
Cylinders (1 lbf; 0.1 in; 1 psi)	KT-22 and AASHTO T 22	VER	Make at least 2 groups of 5 cylinders per pour or major mix design change with concrete sampled from at least 2 different truckloads evenly spaced throughout the pour, with a minimum of 1 set for every 100 cu yd. Include in each group 3 test cylinders to be cured according to KT-22 and 2 t est cylinders to be field-cured. Store the field-cured cylinders on or adjacent to the bridge. Protect all surfaces of the cylinders from the elements in as near as possible the same way as the deck concrete. Test the field-cured cylinders at the same age as the standard-cured cylinders.	
Density of Fresh Concrete (0.1 lb/cu ft or 0.1% of optimum density)	KT-36	ACI		b,c: 1 pe r 100 cu yd for thin overlays and bridge deck surfacing.

Note a: "Type Insp" must = "ACC" when the assignment of a pay quantity is being made. "ACI" when recording test values for additional acceptance information.
Note b: Normal operation. Minimum frequency for exceptional conditions may be reduced by the DME on a project basis, written justification shall be made to the Chief of the Bureau of Materials and Research and placed in the project documents. (Multi-Level Frequency Chart (see page 17, Appendix A of Construction Manual, Part V).
Note c: Applicable only when specifications contain those requirements.

The Engineer will reject concrete that does not comply with specified requirements.

The Engineer will permit occasional deviations below the specified cementitious content, if it is due to the air content of the concrete exceeding the designated air content, but only up to the maximum tolerance in the air content. Continuous operation below the specified cement content for any reason is prohibited.

As the work progresses, the Engineer reserves the right to require the Contractor to change the proportions if conditions warrant such changes to produce a satisfactory mix. Any such changes may be made within the limits of the Specifications at no additional compensation to the Contractor.

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION TO THE
STANDARD SPECIFICATIONS, 2007 EDITION

Add a new SECTION to DIVISION 700:

LOW-CRACKING HIGH-PERFORMANCE CONCRETE – CONSTRUCTION

1.0 DESCRIPTION

Construct the low-cracking high-performance concrete (LC-HPC) structures according to the Contract Documents and this specification.

BID ITEMS	**UNITS**
Qualification Slab	Cubic Yard
Concrete (*) (AE) (LC-HPC)	Cubic Yard
*Grade of Concrete	

2.0 MATERIALS

Provide materials that comply with the applicable requirements.

LC-HPC ....**07-PS0166, latest version**

Concrete Curing Materials ....**DIVISION 1400**

3.0 CONSTRUCTION REQUIREMENTS

a. Qualification Batch and Slab. For each LC-HPC bridge deck, produce a qualification batch of LC-HPC that is to be placed in the deck and complies with **07-PS0166, latest version**, and construct a qualification slab that complies with this specification to demonstrate the ability to handle, place, finish and cure the LC-HPC bridge deck.

After the qualification batch of LC-HPC complies with **07-PS0166, latest version**, construct a qualification slab 15 to 45 days prior to placing LC-HPC in the bridge deck. Construct the qualification slab to comply with the Contract Documents, using the same LC-HPC that is to be placed in the deck and that was approved in the qualification batch. Submit the location of the qualification slab for approval by the Engineer. Place, finish and cure the qualification slab according to the Contract Documents, using the same personnel, methods and equipment (including the concrete pump, if used) that will be used on the bridge deck.

A minimum of 1 day after construction of the qualification slab, core 4 full-depth 4 inch diameter cores, one from each quadrant of the qualification slab, and forward them to the Engineer for visual inspection of degree of consolidation.

Do not commence placement of LC-HPC in the deck until approval is given by the Engineer. Approval to place concrete on the deck will be based on satisfactory placement, consolidation, finishing and curing of the qualification slab and cores, and will be given or denied within 24 hours of receiving the cores from the Contractor. If an additional qualification slab is deemed necessary by the Engineer, it will be paid for at the contract unit price for Qualification Slab.

b. Falsework and Forms. Construct falsework and forms according to **SECTION 708**.

c.Handling and Placing LC-HPC.

(1) Quality Control Plan (QCP). At a project progress meeting prior to placing LC-HPC, discuss with the Engineer the method and equipment used for deck placement. Submit an acceptable QCP according to the Contractor's Concrete Structures Quality Control Plan, Part V. Detail the equipment (for both determining and controlling the evaporation rate and LC-HPC temperature), procedures used to minimize the evaporation rate, plans for maintaining a continuous rate of finishing the deck without delaying the application of curing materials within the time specified in **subsection 3.0f.**, including maintaining a continuous supply of LC-HPC throughout the placement with an adequate quantity of LC-HPC to complete the deck and filling diaphragms and end walls in advance of deck placement, and plans for placing the curing materials within the time specified in **subsection 3.0f**. In the plan, also include input from the LC-HPC supplier as to how variations in the moisture content of the aggregate will be handled, should they occur during construction.

(2) Use a method and sequence of placing LC-HPC approved by the Engineer. Do not place LC-HPC until the forms and reinforcing steel have been checked and approved. Before placing LC-HPC, clean all forms of debris.

(3) Finishing Machine Setup. On bridges skewed greater than 10º, place LC-HPC on the deck forms across the deck on the same skew as the bridge, unless approved otherwise by State Bridge Office (SBO). Operate the bridge deck finishing machine on the same skew as the bridge, unless approved otherwise by the SBO. Before placing LP-HPC, position the finish machine throughout the proposed placement area to allow the Engineer to verify the reinforcing steel positioning.

(4) Environmental Conditions. Maintain environmental conditions on the entire bridge deck so the evaporation rate is less than 0.2 l b/sqft/hr. The temperature of the mixed LC-HPC immediately before placement must be a minimum of 55°F and a maximum of 70°F. With approval by the Engineer, the temperature of the LC-HPC may be adjusted 5°F above or below this range. This may require placing the deck at night, in the early morning or on another day. The evaporation rate (as determined in the American Concrete Institute Manual of Concrete Practice 305R, Chapter 2) is a function of air temperature, LC-HPC temperature, wind speed and relative humidity. The effects of any fogging required by the Engineer will not be considered in the estimation of the evaporation rate (**subsection 3.0c.(5)**).

Just prior to and at least once per hour during placement of the LC-HPC, the Engineer will measure and record the air temperature, LC-HPC temperature, wind speed, and relative humidity on the bridge deck. The Engineer will take the air temperature, wind, and relative humidity measurements approximately 12 inches above the surface of the deck. With this information, the Engineer will determine the evaporation rate using KDOT software or **FIGURE 710-1**.

When the evaporation rate is equal to or above 0.2 lb/ft^2/hr, take actions (such as cooling the LC-HPC, installing wind breaks, sun screens etc.) to create and maintain an evaporation rate less than 0.2 lb/ft^2/hr on the entire bridge deck.

(5) Fogging of Deck Placements. Fogging using hand-held equipment may be required by the Engineer during unanticipated delays in the placing, finishing or curing operations. If fogging is required by the Engineer, do not allow water to drip, flow or puddle on the concrete surface during fogging, placement of absorptive material, or at any time before the concrete has achieved final set.

(6) Placement and Equipment. Place LC-HPC by conveyor belt or concrete bucket. Pumping of LC-HPC will be allowed if the Contractor can show proficiency when placing the approved mix during construction of the qualification slab using the same pump as will be used on the job. Placement by pump will also be allowed with prior approval of the Engineer contingent upon successful placement by pump of the approved mix, using the same pump as will be used for the deck placement, at least 15 days prior to placing LC-HPC in the bridge deck. To limit the loss of air, the maximum drop from the end of a conveyor belt or from a concrete bucket is 5 feet and pumps must be fitted with an air cuff/bladder valve. Do not use chutes, troughs or pipes made of aluminum.

Place LC-HPC to avoid segregation of the materials and displacement of the reinforcement. Do not deposit LC-HPC in large quantities at any point in the forms, and then run or work the LC-HPC along the forms.

Fill each part of the form by depositing the LC-HPC as near to the final position as possible.

The Engineer will obtain sample LC-HPC for tests and cylinders at the discharge end of the conveyor, bucket, or if pumped, the piping.

(7) Consolidation.

- Accomplish consolidation of the LC-HPC on all span bridges that require finishing machines by means of a mechanical device on which internal (spud or tube type) concrete vibrators of the same type and size are mounted (**subsection154.2**).
- Observe special requirements for vibrators in contact with epoxy coated reinforcing steel as specified in **subsection 154.2**.
- Provide stand-by vibrators for emergency use to avoid delays in case of failure.
- Operate the mechanical device so vibrator insertions are made on a maximum spacing of 12 inch centers over the entire deck surface.
- Provide a uniform time per insertion of all vibrators of 3 to 15 seconds, unless otherwise designated by the Engineer.
- Provide positive control of vibrators using a timed light, buzzer, automatic control or other approved method.
- Extract the vibrators from the LC-HPC at a rate to avoid leaving any large voids or holes in the LC-HPC.
- Do not drag the vibrators horizontally through the LC-HPC.
- Use hand held vibrators (**subsection 154.2**) in inaccessible and confined areas such as along bridge rail or curb.
- When required, supplement vibrating by hand spading with suitable tools to provide required consolidation.
- Reconsolidate any voids left by workers.

Continuously place LC-HPC in any floor slab until complete, unless shown otherwise in the Contract Documents.

d. Construction Joints, Expansion Joints and End of Wearing Surface (EWS) Treatment. Locate the construction joints as shown in the Contract Documents. If construction joints are not shown in the Contract Documents, submit proposed locations for approval by the Engineer.

If the work of placing LC-HPC is delayed and the LC-HPC has taken its initial set, stop the placement, saw the nearest construction joint approved by the Engineer, and remove all LC-HPC beyond the construction joint.

Construct keyed joints by embedding water-soaked beveled timbers of a size shown on the Contract Documents, into the soft LC-HPC. Remove the timber when the LC-HPC has set. When resuming work, thoroughly clean the surface of the LC-HPC previously placed, and when required by the Engineer, roughen the key with a steel tool. Before placing LC-HPC against the keyed construction joint, thoroughly wash the surface of the keyed joint with clean water.

e. Finishing. Strike off bridge decks with a vibrating screed or single-drum roller screed, either self-propelled or manually operated by winches and approved by the Engineer. Use a self-oscillating screed on the finish machine, and operate or finish from a position either on the skew or transverse to the bridge roadway centerline. See **subsection 3.0c.(3)**. Do not mount tamping devices or fixtures to drum roller screeds; augers are allowed.

Irregular sections may be finished by other methods approved by the Engineer and detailed in the required QCP. See **subsection 3.0c.(1)**.

Finish the surface by a burlap drag, metal pan or both, mounted to the finishing equipment. Use a float or other approved device behind the burlap drag or metal pan, as necessary, to remove any local irregularities. Do not add water to the surface of LC-HPC. Do not use a finishing aid.

Tining of plastic LC-HPC is prohibited. All LC-HPC surfaces must be reasonably true and even, free from stone pockets, excessive depressions or projections beyond the surface.

Finish all top surfaces, such as the top of retaining walls, curbs, abutments and rails, with a wooden float by tamping and floating, flushing the mortar to the surface and provide a uniform surface, free from pits or porous places. Trowel the surface producing a smooth surface, and brush lightly with a damp brush to remove the glazed surface.

f. Curing and Protection.

(1) General. Cure all newly placed LC-HPC immediately after finishing, and continueuninterrupted for a minimum of 14 d ays. Cure all pedestrian walkway surfaces in the same manner as the bridge deck. Curing compounds are prohibited during the 14 da y curing period.

(2) Cover With Wet Burlap. Soak the burlap a minimum of 12 hours prior to placement on the deck. Rewet the burlap if it has dried more one hour before it is applied to the surface of bridge deck. Apply 1 layer of wet burlap within 10 minutes of LC-HPC strike-off from the screed, followed by a second layer of wet burlap within 5 minutes. Do not allow the surface to dry after the strike-off, or at any time during the cure period. In the required QCP, address the rate of LC-HPC placement and finishing methods that will affect the period between strike-off and burlap placement. See **subsection 3.0c.(1)**. During times of delay expected to exceed 10 minutes, cover all concrete that has been placed, but not finished, with wet burlap.

Maintain the wet burlap in a fully wet condition using misting hoses, self-propelled, machine-mounted fogging equipment with effective fogging area spanning the deck width moving continuously across the entire burlap-covered surface, or other approved devices until the LC-HPC has set sufficiently to allow foot traffic. At that time, place soaker hoses on the burlap, and supply running water continuously to maintain continuous saturation of all burlap material to the entire LC-HPC surface. For bridge decks with superelevation, place a minimum of 1 soaker hose along the high edge of the deck to keep the entire deck wet during the curing period.

(3) Waterproof Cover. Place white polyethylene film on top of the soaker hoses, covering the entire LC-HPC surface after soaker hoses have been placed, a maximum of 12 hours after the placement of the LC-HPC. Use as wide of sheets as practicable, and overlap 2 feet on all edges to form a complete waterproof cover of the entire LC-HPC surface. Secure the polyethylene film so that wind will not displace it. Should any portion of the sheets be broken or damaged before expiration of the curing period, immediately repair the broken or damaged portions. Replace sections that have lost their waterproof qualities.

If burlap and/or polyethylene film is temporarily removed for any reason during the curing period, use soaker hoses to keep the entire exposed area continuously wet. Replace saturated burlap and polyethylene film, resuming the specified curing conditions, as soon as possible.

Inspect the LC-HPC surface once every 6 hours for the entirety of the 14 day curing period, so that all areas remain wet for the entire curing period and all curing requirements are satisfied.

(4) Documentation. Provide the Engineer with a daily inspection set that includes:

- documentation that identifies any deficiencies found (including location of deficiency);
- documentation of corrective measures taken;
- a statement of certification that the entire bridge deck is wet and all curing material is in place;
- documentation showing the time and date of all inspections and the inspector's signature.
- documentation of any temporary removal of curing materials including location, date and time, length of time curing was removed, and means taken to keep the exposed area continuously wet.

(5) Cold Weather Curing. When LC-HPC is being placed in cold weather, also adhere to **07-PS0166, latest version**.

When LC-HPC is being placed and the ambient air temperature may be expected to drop below 40ºF during the curing period or when the ambient air temperature is expected to drop more than 25°F below the temperature of the LC-HPC during the first 24 hours after placement, provide suitable measures such as straw, additional burlap, or other suitable blanketing materials, and/or housing and artificial heat to maintain the LC-HPC and girder temperatures between 40ºF and 75ºF as measured on the upper and lower surfaces of the LC-HPC. Enclose the area underneath the deck and heat so that the temperature of the surrounding air is as close as possible to the temperature of LC-HPC and between 40ºF and 75ºF. When artificial heating is used to maintain the LC-HPC and girder temperatures, provide adequate ventilation to limit exposure to carbon dioxide if necessary. Maintain wet burlap and polyethylene cover during the entire 14 day curing period. Heating may be stopped after the first 72 hours if the time of curing is lengthened to account for periods when the ambient air temperature is below 40ºF. For every day the ambient air temperature is below 40ºF, an additional day of curing with a minimum ambient air temperature of 50ºF will be required. After completion of the required curing period, remove the curing and protection so that the temperature of the LC-HPC during the first 24 hours does not fall more than 25°F.

(6) Curing Membrane.At the end of the 14-day curing period remove the wet burlap and polyethylene and within 30 minutes, apply 2 coats of an opaque curing membrane to the LC-HPC. Apply the curing membrane when no free water remains on the surface but while the surface is still wet. Apply each coat of curing membrane according to the manufacturer's instructions with a minimum spreading rate per coat of 1 gallon per 80 square yards ofLC-HPC surface. If the LC-HPC is dry or becomes dry, thoroughly wet it with water applied as a fog

spray by means of approved equipment. Spray the second coat immediately after and at right angles to the first application.

Protect the curing membrane against marringfor a minimum of 7 days. Give any marred or disturbed membrane an additional coating. Should the curing membrane be subjected to continuous injury, the Engineer may limit work on the deck until the 7-day period is complete. Because the purpose of the curing membrane is to allow for slow drying of the bridge deck, extension of the initial curing period beyond 14 days, while permitted, shall not be used to reduce the 7-day period during which the curing membrane is applied and protected.

(7) Construction Loads. Adhere to **TABLE 710-2**.

If the Contractor needs to drive on the bridge before the approach slabs can be placed and cured, construct a temporary bridge from the approach over the EWS capable of supporting the anticipated loads. Do not bend the reinforcing steel which will tie the approach slab to the EWS or damage the LC-HPC at the EWS. The method of bridging must be approved by the Engineer.

TABLE 710-2: CONCRETE LOAD LIMITATIONS ON BRIDGE DECKS

Days after concrete is placed	Element	Allowable Loads
1*	Subdeck, one-course deck or concrete overlay	Foot traffic only.
3*	One-course deck or concrete overlay	Work to place reinforcing steel or forms for the bridge rail or barrier.
7*	Concrete overlays	Legal Loads; Heavy stationary loads with the Engineer's approval.***
10 (15)**	Subdeck, one-course deck or post-tensioned haunched slab bridges**	Light truck traffic (gross vehicle weight less than 5 tons).****
14 (21)**	Subdeck, one-course deck or post-tensioned haunched slab bridges**	Legal Loads; Heavy stationary loads with the Engineer's approval.***Overlays on new decks.
28	Bridge decks	Overloads, only with the State Bridge Engineer's approval.***

*Maintain a 7 day wet cure at all times (14-day wet cure for decks with LC-HPC).

** Conventional haunched slabs.

*** Submit the load information to the appropriate Engineer. Required information: the weight of the material and the footprint of the load, or the axle (or truck) spacing and the width, the size of each tire (or track length and width) and their weight.

****An overlay may be placed using pumps or conveyors until legal loads are allowed on the bridge.

g. Grinding and Grooving. Correct surface variations exceeding 1/8 inch in 10 feet by use of an approved profiling device, or other methods approved by the Engineer after the curing period. Perform grinding on hardened LC-HPC after the 7 day curing membrane period to achieve a plane surface and grooving of the final wearing surface as shown in the Contract Documents.

Use a self-propelled grinding machine with diamond blades mounted on a multi-blade arbor. Avoid using equipment that causes excessive ravels, aggregate fractures or spalls. Use vacuum equipment or other continuous methods to remove grinding slurry and residue.

After any required grinding is complete, give the surface a suitable texture by transverse grooving. Use diamond blades mounted on a self-propelled machine that is designed for texturing pavement. Transverse grooving of the finished surface may be done with equipment that is not self-propelled providing that the Contractor can show proficiency with the equipment. Use equipment that does not cause strain, excessive raveling, aggregate fracture, spalls, disturbance of the transverse or longitudinal joint, or damage to the existing LC-HPC surface. Make the grooving approximately 3/16 inch in width at 3/4 inch centers and the groove depth approximately 1/8 inch. For bridges with drains, terminate the transverse grooving approximately 2 feet in from the gutter line at the base of the curb. Continuously remove all slurry residues resulting from the texturing operation.

h. Post Construction Conference. At the completion of the deck placement, curing, grinding and grooving for a bridge using LC-HPC, a post-construction conference will be held with all parties that participated in the planning and construction present. The Engineer will record the discussion of all problems and successes for the project.

i. Removal of Forms and Falsework. Do not remove forms and falsework without the Engineer's approval. Remove deck forms approximately 2 weeks (a maximum of 4 weeks) after the end of the curing period (removal of burlap), unless approved by the Engineer.The purpose of 4 week maximum is to limit the moisture gradient between the bottom and the top of the deck.

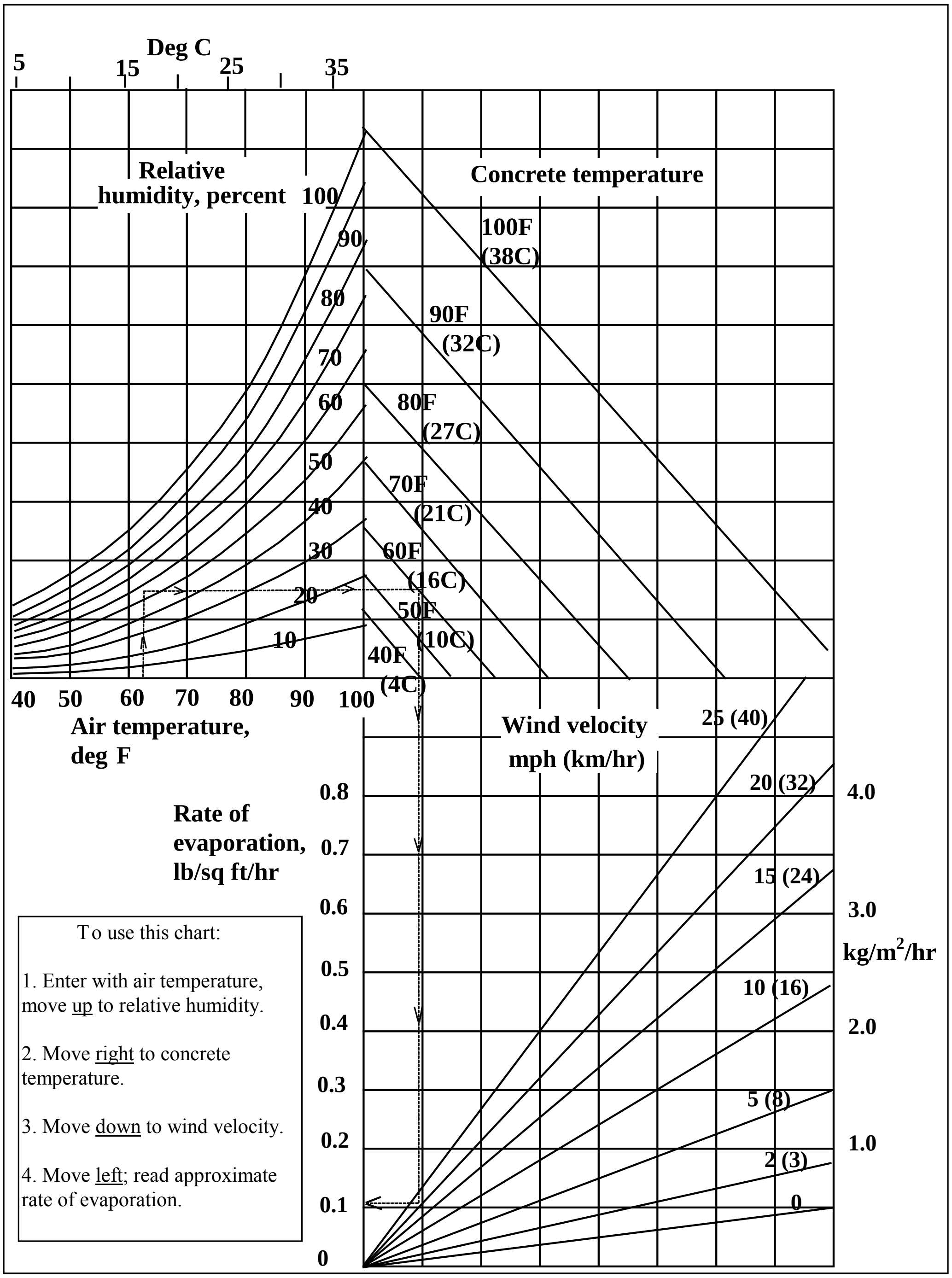
For additional requirements regarding forms and falsework, see **SECTION 708**.

4.0 MEASUREMENT AND PAYMENT

The Engineer will measure the qualification slab and the various grades of (AE) (LC-HPC) concrete placed in the structure by the cubic yard. No deductions are made for reinforcing steel and pile heads extending into the LP-HPC. The Engineer will not separately measure reinforcing steel in the qualification slab.

Payment for the "Qualification Slab" and the various grades of "(AE) (LC-HPC) Concrete" at the contract unit prices is full compensation for the specified work.

FIGURE 710-1: STANDARD PRACTICE FOR CURING CONCRETE



Effect of concrete and air temperatures, relative humidity, and wind velocity on the rate of evaporation of surface moisture from concrete. This chart provides a graphic method of estimating the loss of surface moisture for various weather conditions. To use the chart, follow the four steps outlined above. When the evaporation rate exceeds 0.2 lb/ft^2/hr (1.0 kg/ m^2/hr), measures shall be taken to prevent excessive moisture loss from the surface of unhardened concrete; when the rate is less than 0.2 lb/ft^2/hr (1.0 kg/m^2/hr) such measures may be needed. When excessive moisture loss is not prevented, plastic cracking is likely to occur.

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