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16. Abstract

This study evaluated concretes containing silica fume for use in overlays as a suitable alternative to the widely used latex-modified concrete (LMC). A two-lane, four-span bridge deck was overlaid with concrete containing silica fume at 7 percent or 10 percent by mass of the portland cement as an additional cementitious material.

The results of laboratory tests on field concretes containing silica fume were satisfactory. Silica fume concrete (SFC) bonds well with the base concrete and has very low permeability, high strength, and satisfactory freeze-thaw resistance. In the field, over the 5-year evaluation period, cracking and increases in half-cell potentials and chloride content indicated a tendency to corrosion. However, such increases are also evidenced with LMC overlays. Thus, the results indicate that SFC can be effectively used in thin overlays as an alternative to LMC. SFC, as LMC, is prone to plastic shrinkage; therefore, immediate and proper curing must be provided to avoid the shrinkage cracking that can occur under adverse curing conditions.

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FINAL REPORT

A FIELD INVESTIGATION OF A CONCRETE OVERLAY CONTAINING SILICA FUME ON ROUTE 50 OVER OPEQUON CREEK

Celik Ozyildirim, Ph.D.
Senior Research Scientist

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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ABSTRACT

This study evaluated concretes containing silica fume for use in overlays as a suitable alternative to the widely used latex-modified concrete (LMC). A two-lane, four-span bridge deck was overlaid with concrete containing silica fume at 7 percent or 10 percent by mass of the portland cement as an additional cementitious material.

The results of laboratory tests on field concretes containing silica fume were satisfactory. Silica fume concrete (SFC) bonds well with the base concrete and has very low permeability, high strength, and satisfactory freeze-thaw resistance. In the field, over the 5-year evaluation period, cracking and increases in half-cell potentials and chloride content indicated a tendency to corrosion. However, such increases are also evidenced with LMC overlays. Thus, the results indicate that SFC can be effectively used in thin overlays as an alternative to LMC. SFC, as LMC, is prone to plastic shrinkage; therefore, immediate and proper curing must be provided to avoid the shrinkage cracking that can occur under adverse curing conditions.

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INTRODUCTION

Laboratory investigations at the Virginia Transportation Research Council and studies by others have indicated that concretes containing silica fume (SF) have enhanced resistance to the penetration of chloride or other corrosive solutions.[1-3] Such liquids can cause corrosion of the reinforcing bars and consequent deterioration of the concrete in bridge decks. Currently, one of the protective systems against corrosion widely used in the repair of bridge decks is latex-modified concrete (LMC). The use of this concrete, generally in thin overlays with a minimum thickness of 32 mm (1 1/4 in), is effective, but costly. A possible economical alternative to LMC is concrete containing SF.[1] Savings result from a reduction in actual material costs and from the use of widely available truck mixers instead of the mobile mixers required for LMC.

OBJECTIVE AND SCOPE

The objective of the study was to determine whether concretes containing SF can be successfully used in thin overlays with a minimum thickness of 32 mm (1 1/4 in) as a suitable alternative to the widely used LMC system. The study involved a field installation of a bridge deck overlay with silica fume concrete (SFC) batched at a ready-mix concrete plant and transported to the job site.

The bridge was built in 1941 and is located on Route 50 over Opequon Creek near Winchester, Virginia. The deck carries westbound traffic and has two lanes and four spans. Each span is 13.0 m (42.5 ft) long and 7.3 m (24 ft) wide. The bridge is on a 5-degree grade and is lower at the west end.

METHODOLOGY

Overview

The minimum thickness of the overlay containing SF was specified as 32 mm (1 1/4 in). SF was added at 7 percent or 10 percent by mass of portland cement. Tests were made on four batches of SFC to determine the characteristics of the freshly mixed concretes. The placement procedure was observed, and the conditions of the overlay were monitored over a 5-year period to determine its performance.

Materials and Mixture Proportions

A Type II cement and a commercially available SF slurry were used. The SF had a specific gravity of 2.25. The cement factor was 390 kg/m^3 (658 lb/yd^3). The maximum specified water-to–cementitious materials (cement plus SF) ratio (w/c) was 0.40. The fine aggregate was a siliceous sand with a fineness modulus of 2.90 and a specific gravity of 2.61. The coarse aggregate was gravel with a specific gravity of 2.66 and a unit weight of 1,666 kg/m^3 (104.0 lb/ft^3); the nominal maximum size was 13 mm (1/2 in). The concrete mixture proportions established as a result of laboratory investigations[1] and recommendations by the marketers are summarized in Table 1.

All of the batches contained a commercially available neutralized vinsol resin for air entrainment and a sulfonated naphthalene formaldehyde condensate as a high-range water-reducing admixture (HRWR), both of which were added at the plant. However, to achieve the desired workability while maintaining the w/c, more HRWR was added at the job site in two of the seven batches.

Placement Procedure

The previous asphalt overlay was removed by scarifying and milling. On the day before placement of the overlay, the concrete surface was sandblasted, wetted, and covered with a plastic sheet. The placement of the overlay started from the higher east end. The plastic cover on the deck was removed, and the concrete surface was wetted if it was dry. Freestanding water was removed. To ensure a good bond between the base concrete and the overlay, a small portion of concrete was discharged and the mortar fraction was scrubbed on the surface with coarse-bristle brooms. All coarse aggregate left was brushed aside and discarded. As soon as possible after the scrubbing, the concrete from the trucks was placed on the deck surface and consolidated by a vibratory roller screed. Immersion-type vibrators were used to vibrate concrete along the joints and the edge of parapets. Behind the screed, the surface levelness was checked and hand floats were used to eliminate

Table 1
MIXTURE PROPORTIONS, kg/m^3

Ingredient	SF, 7%	SF, 10%
Portland cement	390	390
Silica fume	27.3	27.3
Fine aggregate	753	727
Coarse aggregate	899	899
Maximum w/c	0.40	0.40
Air content	7 ± 2%	7 ± 2%

1 kg/m^3 = 1.69 lb/yd^3.

defects. Along the edge of the parapet and at the longitudinal joint, the surface was screeded with hand floats. The deck surface was textured using metal tines and subsequently subjected to curing. Two curing procedures were used: (1) The concrete was covered with wet burlap and a polyethylene sheet. These were removed the next day, and the concrete was then sprayed with a curing compound. (2) A curing compound was applied after finishing and texturing. The use of wet burlap is very effective, but for savings and convenience, its substitution or limited use with a curing compound was evaluated.

At each span, concretes with different amounts of SF that had been cured by different methods were alternated to minimize differences in external factors, such as traffic, geometry, and weather (see Figure 1). When burlap was used, care was taken not to disturb the tined texture.

The bridge deck was overlaid with concrete containing 7 percent or 10 percent SF by mass of portland cement following the sequence shown in Figure 1. The concrete was placed on the traffic lane on May 13, 1987, and on the passing lane on June 5, 1987. A total of 22.6 m^3 (29.5 yd^3) of concrete was placed: 13.0 m^3 (17 yd^3) with 10 percent SF and 9.6 m^3 (12.5 yd^3) with 7 percent SF. The reported w/c was 0.39. In order to minimize the time between batching and placement, the concrete was furnished in four trucks, each carrying about 3 m^3 (4 yd^3), for the traffic lane and in three trucks, each with 3.5 m^3 (4 1/2 yd^3), for the passing lane. The travel time for the truck mixers was about 20 minutes, and the average time from batching to discharge was about 35 minutes.

Tests and Observations

Samples were obtained in accordance with ASTM C 172 from the middle third of four of the seven truckloads of concrete for the following tests.

Freshly Mixed Concrete

Tests were conducted to determine the air content (ASTM C 231), slump (ASTM C 143), and temperature at the fresh state. The required air content was 7 ± 2 percent, and the required slump was 150 ± 50 mm (6 ± 2 in).

Hardened Concrete

As summarized in Table 2, specimens were prepared for tests for compressive, flexural, and bond strength; permeability to chloride ions; resistance to cycles of freezing and thawing; and the air-void parameters.

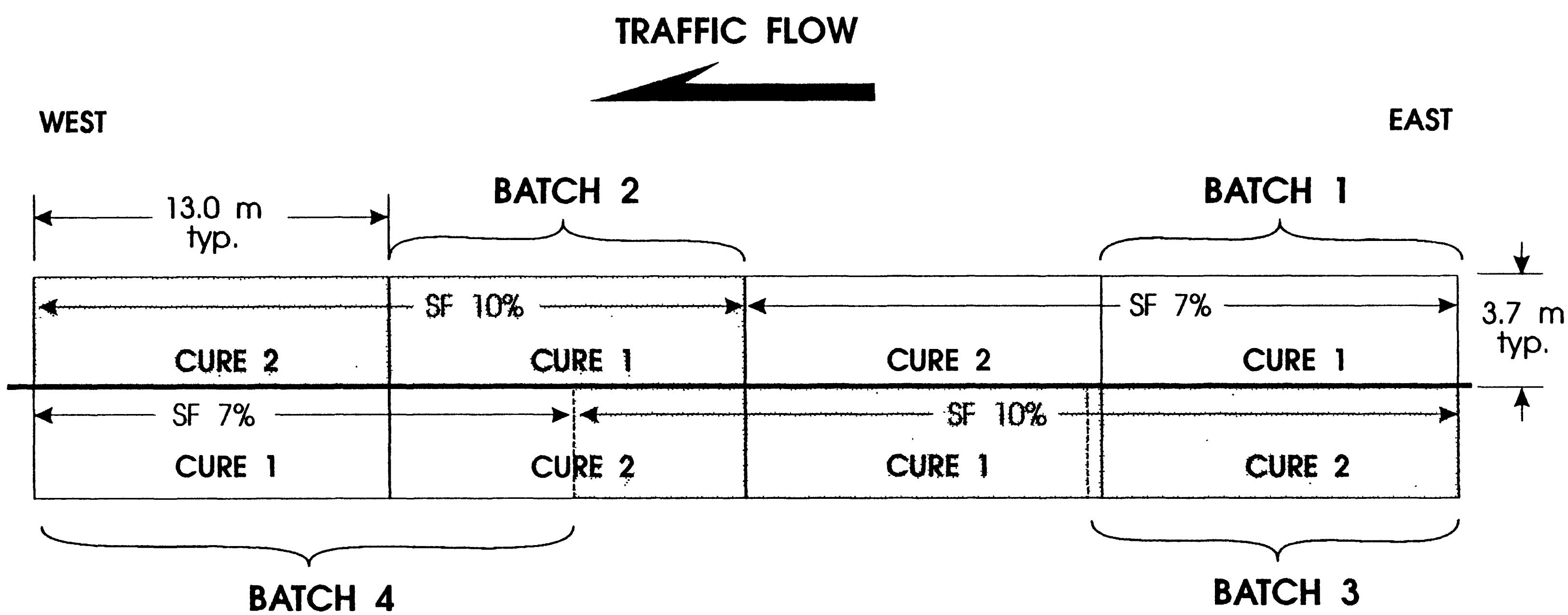

Figure 1. Westbound lanes. Cure 1: wet burlap and polyethylene sheet followed by a curing compound. Cure 2: curing compound.

Table 2
NUMBER OF SPECIMENS AND TESTS FOR EACH BATCH

Test	Specimens No.	Specimens Size, mm	Test Method	Age (days) Tested
Compressive strength	12	100 x 200	AASHTO T 22[a]	1, 7, 28
Flexural strength	3	75 x 75 x 285	ASTM C 78	28
Bond	3	100	[b]	28
Chloride permeability	4	100 x 50	AASHTO T 277[c]	28
Freeze-thaw	3	75 x 100 x 400	ASTM C 666[d]	21
Air-void system	1	100 x 200	ASTM C 457[e]	28

[a] Neoprene pads in steel end caps were used for capping.
[b] 50-mm-thick overlays on cylindrical specimens representing base concrete were sheared at the interface.
[c] 50-mm-thick slabs cut from the top of 100 x 200 mm cylinders.
[d] Cured 2 weeks moist, 1 week dry and tested in 2% NaCl.
[e] Linear traverse method.

Strength

The compressive strength was determined in accordance with AASHTO T 22 at 1, 7, and 28 days using 100 by 200 mm (4 by 8 in) cylinders. Neoprene pads in steel end caps were used for capping. The strength requirement was 31.0 MPa (4,500 psi) at 7 days.

The flexural strength was determined at 28 days in accordance with ASTM C 78 using a simple beam measuring 75 by 75 by 285 mm (3 by 3 by 11 1/4 in).

To determine bond strengths, specimens were prepared by overlaying slabs cut from cylinders 100 mm (4 in) in diameter and subjecting the interface to shear after 28 days of moist curing of the overlay. The slabs used as the base concrete were made from a typical bridge deck concrete prepared in the laboratory, where they were moist cured for at least 1 month. Prior to placement of the overlays, the surface of the base concrete was allowed to dry for at least 1 day. This condition differed from that on the bridge deck, where a mortar layer was scrubbed on the wetted base concrete.

Chloride Permeability

The degree of resistance of concretes to the penetration of chloride ions was determined by the rapid chloride permeability test (AASHTO T 277). The specimens were slabs 50 mm (2 in) thick cut from the top of 100 by 200 mm (4 by 8 in) cylinders. They were moist cured for 2 weeks, and then air dried until the time of the test. In this test, the specimens were subjected to a direct voltage; the charge passing through them in a 6-hr period was determined and expressed in coulombs.[4] The coulomb values obtained are related to the chloride permeability as shown in Table 3. LMC produced from locally available materials is expected to have low permeability (1,000 to 2,000 coulombs) at 28 days, and as hydration continues, the

Table 3
RAPID CHLORIDE PERMEABILITY TEST
(AASHTO T 277)

Charge Passed, coulombs	Chloride Permeability
>4000	High
2000 to 4000	Moderate
1000 to 2000	Low
100 to 1000	Very low
<100	Negligible

values are expected to drop to the very low range (100 to 1,000 coulombs) during the service life of the structure. Such concretes have performed satisfactorily over the years.[5] Concretes with SF would be considered satisfactory if permeability similar to that of LMC could be obtained. In earlier laboratory work, concretes with 5 percent SF replacement of the portland cement have yielded satisfactory results.[1] However, because of the variabilities expected in the field and the recommendations of the marketers of the SF products, greater amounts (7 percent and 10 percent) were used in this study.

Resistance to Freezing and Thawing

The resistance of the concretes to damage from cycles of freezing and thawing was determined using Procedure A of ASTM C 666 except that (1) the specimens were air dried for 1 week following the 2 weeks of moist curing, and (2) the test water contained 2 percent NaCl. The acceptance criteria are that the average of three specimens at 300 cycles must have a weight loss (WL) of 7 percent or less, a durability factor (DF) of 60 or more, and a surface rating (SR) of 3 or less. The surface rating was determined in accordance with ASTM C 672. The top and molded surface were rated separately, and the values averaged.

Air-Void Parameters

The air-void parameters of small, large, and total voids; specific surface; and spacing factor were determined in accordance with the linear traverse method of ASTM C 457. The specimens were moist cured for at least 1 month, cut vertically, and then lapped on one side for the linear traverse analysis. Voids were separated into two groups based on the diameter in the plane surface: small (≤1 mm) and large (>1 mm). Small voids are considered to result from air entrainment, and large ones from a lack of consolidation or from extra water in the mixture. The amount of large voids is generally 2 percent or less in properly prepared concretes. For adequate protection of critically saturated concrete from extreme exposures, a specific surface value of 24 mm^{-1} (600 in^{-1}) or more and a spacing factor of 0.20 mm (0.008 in) or less are generally recommended.[7]

Observations and Evaluations

During construction, the placement procedure was observed and recorded. Deck evaluations were made every year except the 4th for a 5-year period. They consisted of a visual survey, hammer and chain soundings, measurement of electrical half-cell potentials, and determination of the chloride content. The depth of the cover of hardened overlay was also measured using a device that generates an electrical field, which is affected by the steel. Only a visual survey was conducted in the 2nd year. In the 3rd year, five cores were obtained for additional testing.

RESULTS AND DISCUSSION

Four batches were tested individually for slump, air content, and temperature at the freshly mixed stage, and specimens were prepared from the same batches for tests at the hardened stage as shown in Table 2. The workability of the freshly mixed concrete was controlled by the use of HRWR. In the first batch tested, the slump was intentionally held low because of concerns about placement of high-slump concretes on a grade. At the job site, two additional dosages of HRWR were added to the concrete to improve workability. In the other three batches, the initial slump was higher than in the first batch and no additional HRWR was used at the job site, even though some slump loss occurred.

When the first two batches were placed in the traffic lane, the weather was cloudy with high temperatures in the low 20s (C) (70s [F]). When the next two batches were placed in the passing lane, it was a sunny day with high temperatures in the high 20s (C) (80s [F]). Test specimens were covered with wet burlap and plastic and left at the job site for a day before they were transferred to the laboratory.

Freshly Mixed Concrete

The results of the tests for slump and air content and the temperatures of the air and concrete are given in Table 4. Slump and air content were within the specifications when tested for acceptance from the beginning of the load as it arrived at the job site. However, as time passed, slump loss occurred and the slump of one batch was less than the lower limit of 100 mm (4 in) when sampled. Similarly, two of the four batches sampled from the middle third had a low air content that did not meet the specifications and the other two were in the lower portion of the acceptable range.

Table 4
CHARACTERISTICS OF FRESHLY MIXED CONCRETE

Batch	Lane	SF, %	Slump, mm	Air, %	Temperature,°C	
					Air	Concrete
1	Traffic	7	190	3.7	17	27
2	Traffic	10	65	4.5	17	26
3	Passing	10	190	5.2	20	27
4	Passing	7	180	6.0	26	28

1 mm = 0.04 in.
°C = (°F – 32)/1.8.

Hardened Concrete

Strength

The results of tests for compressive strength are given in Table 5. The 1-day compressive strength exceeded 24.1 MPa (3,500 psi) in concretes with either 7 percent or 10 percent SF. This strength is accepted as being sufficient for opening the lane to traffic. All concretes had a strength at 7 days that was significantly above the specified strength of 31.0 MPa (4,500 psi). The minimum value was 42.0 MPa (6,090 psi). The lowest 28-day strength was 53.8 MPa (7,800 psi).

The test values for flexural strength summarized in Table 5 ranged from 5.26 MPa (763 psi) to 6.60 MPa (957 psi), indicating that either addition rate of SF provides satisfactory values.

The results of tests for bond strength are given in Table 5. The minimum bond strength was 2.64 MPa (383 psi). This is well above the 1.38 MPa (200 psi) generally reported as being satisfactory.[6]

Chloride Permeability

The results of tests for rapid chloride permeability are given in Table 6. They indicated that all concretes had a value below 1,000 coulombs at 28 days, which is in the very low chloride permeability range. Concretes containing 10 percent SF had a lower coulomb value than those with 7 percent SF. For comparison, A-4 bridge deck concretes normally used by VDOT have a coulomb value above 4,000 (high range) at 28 days.

Resistance to Freezing and Thawing

The WL, DF, and SR values are given in Table 7. Concretes from all batches except batch 1 had an acceptable DF. Two of the batches had a high WL.

Table 5
STRENGTH DATA, MPa
(AVERAGE OF THREE SPECIMENS)

Batch	Lane	SF, %	Compressive Strength 1 day	Compressive Strength 7 days	Compressive Strength 28 days	Flexural Strength 28 days	Bond Strength 28 days
1	Traffic	7	29.9[a]	47.2	63.3	6.60	2.67
2	Traffic	10	25.7[a]	42.0	54.4	6.23	2.64
3	Passing	10	40.5[b]	46.7	53.8	5.93	4.81
4	Passing	7	37.0[b]	42.1	47.9	5.26	4.16

[a] 28 hours.
[b] 29 hours.
1 MPa = 145 psi.

Table 6
CHLORIDE PERMEABILITY AT 28 DAYS
(AVERAGE OF TWO SPECIMENS)

Batch	Lane	SF, %	Coulombs
1	Traffic	7	648
2	Traffic	10	354
3	Passing	10	437
4	Passing	7	716

Table 7
FREEZE-THAW DATA AT 300 CYCLES
(AVERAGE OF THREE SPECIMENS)

Batch	Lane	SF	Weight Loss, %	DF	SR
1	Traffic	7	5.8[a]	33	2.5
2	Traffic	10	7.8	60	1.9
3	Passing	10	0.5	80[b]	1.1
4	Passing	7	0.4	79	0.8

[a] Test terminated at 200 cycles when relative dynamic modulus values fell below 60%.
[b] One beam had a DF of 63.

Air-Void Parameters

Data on small, large, and total voids; specific surface; and spacing factor are summarized in Table 8. The total air-void content was in the lower half of the specified range. One of the specimens had a large amount of large voids. The slump for that batch was low, and the large voids could have been caused by difficulties in consolidation.

The specific surface values were low. This indicates a coarse air-void system, which is expected in concretes containing HRWR.[8] The spacing factor in three of the four concretes was above the maximum 0.20 mm (0.008 in) recommended for satisfactory performance. The concrete with the highest spacing factor and the lowest air content was the one that failed the freezing and thawing test.

Table 8
AIR-VOID SYSTEM OF HARDENED CONCRETE

			Void Content			Specific Surface, mm^{-1}	Spacing Factor, mm
Batch	Lane	SF, %	≤1 mm	>1 mm	Total		
1	Traffic	7	4.2	1.0	5.2	21	0.22
2	Traffic	10	3.5	3.0	6.5	19	0.22
3	Passing	10	4.5	1.3	5.8	23	0.20
4	Passing	7	5.1	2.1	7.2	18	0.21

1 mm = 0.04 in.

Concretes used on bridge decks are not normally critically saturated. Further, low-permeability concretes are difficult to saturate. Thus, SFC is expected to resist cycles of freezing and thawing in service better than they perform when tested in accordance with ASTM C 666, Procedure A. Similarly, specimens of LMC generally exhibit poor performance when tested in accordance with ASTM C 666,[5] but their field performance has been satisfactory. Thus, these borderline characteristics do not necessarily lead to poor field performance of the experimental overlays.

Deck Evaluations

An initial evaluation of the deck was made on June 9, 1987, just after the placement. Then, evaluations were made on June 23, 1988; May 23, 1989; April 19, 1990; and April 9, 1992.

Initial Evaluation

At the initial evaluation, the passing lane was still closed and the traffic lane had been opened to traffic for 2 weeks. The visual survey indicated the presence of only one crack about 0.3 m (1 ft) long, perpendicular to the transverse joint at the west end of the passing lane. This crack extended through the thickness of the overlay slab. Chain drag soundings revealed delaminated areas along the longitudinal joint in the first three spans, mainly in the passing lane. The delaminations were attributed to poor consolidation, which occurred in areas that could not be consolidated by the roller screed and were supposed to have been consolidated by an immersion-type vibrator. The contractor made repairs to correct the delaminated areas to the extent possible.

The average depth of cover above the reinforcement for each lane at each span is given in Table 9. The values indicated the presence of adequate cover, exceeding 75 mm (3 in) in all spans even though some of the cover was due to the old base concrete.

Table 9
AVERAGE DEPTH OF COVER

Span	Lane	SF, %	Average Cover Depth, mm	Standard Deviation, mm
1	Traffic	7	97	15
	Passing	10	107	11
2	Traffic	7	101	12
	Passing	10	105	15
3	Traffic	10	75	10
	Passing	7	86	10
4	Traffic	10	89	27
	Passing	7	114	14

1 mm = 0.04 in.

Electrical half-cell potentials were measured in accordance with ASTM C 876 using 1.2-m (4-ft) grids. The percentage of the readings in each of the three specified ranges given in ASTM is summarized for each lane at each span in Table 10. The first range is when readings are more positive than –0.20 V, indicating that there is a 90 percent probability that no corrosion of the reinforcing steel is occurring. In the second range, the readings are between –0.20 V and –0.35 V, indicating that it is uncertain whether corrosion is occurring. The third range is when the readings are more negative than –0.35 V, indicating that there is more than a 90 percent probability that corrosion is occurring. The results indicated that there were small areas of corrosion in all spans except the first. These were mainly at the transverse joints and at the curb along the passing lane.

First-Year Evaluation

In the evaluations at 1 year, the visual survey revealed several narrow cracks perpendicular to all the transverse joints, mostly about 0.3 m (1 ft) to 0.5 m (1 1/2 ft) in length. In general, scaling on the deck was absent or very light. However, there were some areas where the coarse aggregate was exposed and scaling was light to moderate. The scaling was limited to the top surface, and no visible loss of macrotexture was evident.

The soundings showed that there were delaminated areas along the longitudinal joint in all the spans, even at locations that had been repaired by the contractor. In addition, two small areas measuring 150 by 150 mm (6 by 6 in) in the passing lane of span 4 were delaminated. The electrical half-cell potentials given in Table 10 indicated the presence of small areas of corrosion in spans 1 and 4. These locations were mainly at the transverse joints.

Chloride content was determined at each span at two average depths. The first average depth was 13 mm (1/2 in), where the samples were obtained from a section 6 mm (1/4 in) to 19 mm (3/4 in) from the surface. The second average depth was 25 mm (1 in), obtained from a section 19 mm (3/4 in) to 32 mm (1 1/4 in) from the surface. The samples were taken randomly at each span at the wheel path to represent the two SF additions and the two curing conditions. The results indicated that the chloride content was negligible at the lower average depth of 25 mm (1 in). At the 13-mm (1/2-in) depth, one of the values was 0.83 kg/m^3 (1.41 lb/yd^3), which is slightly above the threshold level of 0.8 kg/m^3 (1.3 lb/yd^3).[9]

Second-Year Evaluation

After the second winter, the condition of the deck was observed from the curbside without closing the lanes. The cracks perpendicular to the joints were visible. In addition, very narrow pattern-type cracking was seen in the traffic lane of span 1 and the passing lane of span 3.

Table 10
PERCENTAGE DISTRIBUTION OF HALF-CELL POTENTIALS AT DIFFERENT AGES

			1987 (Initial)			1988 (1 yr)			1990 (3 yr)			1992 (5 yr)		
Span	Lane	SF, %	A[a]	B[b]	C[c]	A	B	C	A	B	C	A	B	C
1	Traffic	7	72	28	0	57	33	10	53	44	3	78	22	0
1	Passing	10	77	23	0	53	44	3	57	33	10	67	33	0
2	Traffic	7	86	14	0	90	10	0	36	64	0	0	92	8
2	Passing	10	40	56	4	97	3	0	44	48	8	2	83	15
3	Traffic	10	81	19	0	47	53	0	0	94	6	0	97	3
3	Passing	7 & 10	0	90	10	33	67	0	0	83	17	23	71	6
4	Traffic	10	83	17	0	2	81	17	58	39	3	8	86	6
4	Passing	7	6	77	17	0	92	8	27	60	13	21	71	8

[a] More positive than –0.20 V, indicative of no corrosion (90% probability that no corrosion is occurring).
[b] In the range of –0.20 to –0.35 V, presence of corrosion uncertain.
[c] More negative than –0.35 V, indicative of corrosion (90% probability that corrosion is occurring).

Table 11
CHLORIDE CONTENT, kg/m^3

				At 1 Year		At 3 Years		At 5 Years	
Span	Lane	Curing	SF, %	13 mm	25 mm	13 mm	25 mm	13 mm	25 mm
1	Traffic	Burlap	7	0.65	0.00	0.35	0.00	0.97	0.10
2	Passing	Burlap	10	0.55	0.03	1.27	0.23	1.03	0.22
3	Passing	CC	7	0.83	—	1.61	0.36	2.03	0.36
4	Traffic	CC	10	0.33	0.00	1.10	0.06	1.35	0.36

1 kg/m^3 = 1.69 lb/yd^3.

Third-Year Evaluation

The evaluations at the third year showed that the cracks at the transverse joints had increased in number, and some had lengthened. Very narrow pattern-type cracking was visible in the traffic lane of spans 1 and 3 and the passing lane of spans 2, 3, and 4. In general, the scaling was light and the areas with exposed aggregate retained macrotexture. The sounding revealed delaminations along the longitudinal joint and the two small areas in the passing lane of span 4. In addition, another section 0.5 by 1.2 m (1 1/2 by 4 ft) in the traffic lane of span 2 was delaminated adjacent to the transverse joint.

The electrical half-cell potentials shown in Table 10 indicated small areas of corrosion in all the spans, mainly at the transverse joint and some in spans 2 and 3 along the longitudinal joint. Samples for measuring the chloride content were taken near the earlier sampling areas except that the one in the first span was obtained near the longitudinal joint. The results given in Table 11 show that values were all below the threshold value at the 25 mm (1 in) average depth. Three of the four values at the 13 mm (1/2 in) depth ranged from 1.10 kg/m^3 (1.87 lb/yd^3) to 1.61 kg/m^3 (2.73 lb/yd^3); thus, they were above the threshold value.

Five cores were taken from the traffic lane to determine permeability. Two of the cores were obtained from different spans, had 7 percent SF, and had undergone different curing procedures with no visible cracks. Two other cores were obtained from the cracked sections: one had a longitudinal crack and was from the delaminated area near the transverse joint, and the other had pattern cracks. The remaining core was taken from the scaled area with exposed aggregate.

The cores tested for permeability had coulomb values of 471 and 472, indicative of very low permeability. Examination of the cores with cracks suggested that the cracks had originated in the plastic state. The cores also revealed that the overlay concrete had bonded well to the base concrete and the delamination found adjacent to the transverse joint by sounding was in the base concrete at the level of the steel and not at the interface between the overlay and base concrete. Based on three cores, an average overlay thickness of 44 mm (1 3/4 in) was determined. The core with exposed aggregates on the surface had borderline values for air content (4.8 percent) and the spacing factor (0.21 mm [0.0081 in]), which along with possible overworking of the surface (which causes loss of air at the surface) may have contributed to surface scaling. There were no cracks in the overlay concrete indicative of progressive freeze-thaw damage.

Fifth-Year Evaluation

At the fifth-year evaluation, the visual survey showed that cracking had increased. The cracks perpendicular to the joints had become wider, and some had exceeded 1 mm in width. The pattern-type cracking was visible in both lanes in all spans but was of different severity. In the traffic lane, the worst cracking was in span 1 and the least in span 2. In the passing lane, the worst cracking was in span 4 and the least in span 2. In general, there was less cracking in the traffic lane than in the passing lane. The pattern-type cracks on the deck between the joints were narrow. The sections with the worst pattern-type cracking contained 7 percent SF and were cured with wet burlap for the first day. Those with the least amount of cracking contained 10 percent SF and were cured with a curing compound without burlap. The large amount of cracking in sections with burlap was attributed to the delay in the application of the burlap because of concerns that early application might disturb the macrotexture.

Except at those areas with exposed aggregate along the wheel path or those initially scaled that were rated to be light to moderate, the scaling on the surface was mainly light.

Soundings of the deck again showed delaminations along the longitudinal joint, but the rest of the deck surface appeared to be sound and even the small sections found to be delaminated in earlier surveys were not detected in this evaluation.

Electrical half-cell potentials, given in Table 10, showed small areas in all spans, except span 1, that indicated corrosion. They suggested that corrosion was occurring mainly along the transverse joints, with some along the longitudinal joint in span 2. Over the 5-year period, the readings became slightly more negative, indicating a trend toward increased corrosion.

The chloride content of samples obtained along the wheel path and near the earlier sampling locations is summarized in Table 11. The values were less than one-half the threshold value at the average depth of 25 mm (1 in). At the average depth of 13 mm (1/2 in), all the values were in excess of the threshold value. Over the 5-year evaluation, the chloride content increased.

The increase in half-cell potentials and chloride content over time is also observed with LMC overlays[5]; however, the long-term performance of LMC has been satisfactory.

CONCLUSIONS

1. Concrete containing SF can be used as a cost-effective alternative to LMC in thin overlays on bridge decks.
2. Concretes containing either 7 percent or 10 percent SF have a satisfactory strength and a very low chloride permeability and are effective in minimizing the intrusion of chlorides.
3. The resistance of air-entrained SFCs to cycles of freezing and thawing is expected to be satisfactory.
4. SFC bonds well with the base concrete.
5. Prompt and effective curing is essential in order to prevent cracking.

6. Over the 5-year evaluation period, the half-cell potentials became more negative, indicating a trend toward increased corrosion. Similarly, the chloride content increased. Such increases are also evidenced in LMC. The performance of SFC appears to be comparable to that of LMC.

RECOMMENDATIONS

1. Concretes containing 7 percent SF should be used as an alternative to LMC.
2. SFCs should be moist cured promptly by fog misting until they are covered with wet burlap and plastic sheeting. Then, the curing should be continued by the use of a curing compound.
3. Evaluation of the curing procedure that uses only a curing compound should continue.

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