Repairs With Self-Consolidating Concrete and Galvanic Anodes to Extend Bridge Life

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16. Abstract:

Historically, repairs of substructure elements that contain vertical and overhead sections have used either shotcrete or a conventional Virginia Department of Transportation (VDOT) Class A3 (3,000 psi) or Class A4 (4,000 psi) concrete. This study investigated the use of self-consolidating concrete (SCC), which has high flow rates, bonds well, has low permeability, and provides smooth surfaces, as another option. SCC can be placed in narrow areas and fit the existing geometry of the element.

This study also explored the use of galvanic anodes in SCC repairs at various locations. VDOT has been evaluating galvanic anodes in substructure repairs as a means to prevent corrosion deterioration in the area around a repair, as corrosion is the main source of damage in reinforced-concrete structures. In VDOT's Lynchburg and Staunton districts, SCC repairs were made using galvanic anodes in select locations. Several locations in these structures were repaired with the same SCC mixture but did not include anodes so as to provide a means for determining the benefit of embedding galvanic anodes.

Based on the field results, repairs of substructure elements should be performed using SCC mixtures when possible. These repairs can be aesthetically pleasing if the formwork is carefully installed, as the finished SCC surface will strongly replicate the form surfaces and edges when the forms are removed. In addition, the use of SCC mixtures ensures that the repair concrete is of known quality and will flow through restrictive areas and fill the entire repair volume when the repair is properly performed. Although small vertical cracks were evident in the patched areas, they did not appear to reduce durability. This was based on the observation that after 7 years of service, the repairs and areas around the repairs did not exhibit corrosion activity.

In patch areas with anodes, the anodes did provide protection to the steel immediately adjacent to the repair area. However, chloride-contaminated concrete areas in contact with reinforcement steel not removed and located away from the repairs were not protected. Unfortunately, corrosion is progressing in these areas where the concrete was considered acceptable based on sounding of the concrete during repairs, but now these areas require repair. This is due to the fact that concrete sounding is not a reliable way of locating chloride-contaminated concrete since it requires the concrete to be delaminated. A more reliable method would be the use of half-cell potential measurements to identify the areas in which concrete should be repaired. Progression of corrosion demonstrates the necessity of removing all chloride-contaminated concrete adjacent to the reinforcement as anodes in the patch will provide protection only in a narrow area around the patch.

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FINAL REPORT

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ABSTRACT

Historically, repairs of substructure elements that contain vertical and overhead sections have used either shotcrete or a conventional Virginia Department of Transportation (VDOT) Class A3 (3,000 psi) or Class A4 (4,000 psi) concrete. This study investigated the use of self-consolidating concrete (SCC), which has high flow rates, bonds well, has low permeability, and provides smooth surfaces, as another option. SCC can be placed in narrow areas and fit the existing geometry of the element.

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Based on the field results, repairs of substructure elements should be performed using SCC mixtures when possible. These repairs can be aesthetically pleasing if the formwork is carefully installed, as the finished SCC surface will strongly replicate the form surfaces and edges when the forms are removed. In addition, the use of SCC mixtures ensures that the repair concrete is of known quality and will flow through restrictive areas and fill the entire repair volume when the repair is properly performed. Although small vertical cracks were evident in the patched areas, they did not appear to reduce durability. This was based on the observation that after 7 years of service, the repairs and areas around the repairs did not exhibit corrosion activity.

In patch areas with anodes, the anodes did provide protection to the steel immediately adjacent to the repair area. However, chloride-contaminated concrete areas in contact with reinforcement steel not removed and located away from the repairs were not protected. Unfortunately, corrosion is progressing in these areas where the concrete was considered acceptable based on sounding of the concrete during repairs, but now these areas require repair. This is due to the fact that concrete sounding is not a reliable way of locating chloride-contaminated concrete since it requires the concrete to be delaminated. A more reliable method would be the use of half-cell potential measurements to identify the areas in which concrete should be repaired. Progression of corrosion demonstrates the necessity of removing all chloride-contaminated concrete adjacent to the reinforcement as anodes in the patch will provide protection only in a narrow area around the patch.

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INTRODUCTION

In substructure elements, leaking joints, shy cover depth, poor quality concrete, and lack of consolidation have commonly contributed to corrosion-related spalling, and such adverse conditions have led to costly repairs. Historically, repairs consist of removing deteriorated concrete and then placing shotcrete or Virginia Department of Transportation (VDOT) Class A3 concrete, which has a minimum 28-day compressive strength requirement of 3,000 psi, or VDOT Class A4 concrete, which has a minimum 28-day compressive strength requirement of 4,000 psi. Shotcrete is projected under velocity to the surface, requiring special equipment and an experienced operator to ensure proper placement, thus making it a specialized and costly repair system (American Shotcrete Association, 2014). Moreover, in small jobs, mobilizing the shotcrete equipment and finding a qualified operator can be an issue, and the field experience of shotcrete repairs indicates variable performance for this type of repair. For VDOT Class A3 or Class A4 concrete, formwork is built that usually contains a larger volume than the area to be repaired, which is done solely to enable the placement and consolidation of concrete with conventional consistency. Therefore, the resulting repaired area often protrudes, does not match the geometry of the existing element, and is not aesthetically pleasing. In addition, the consolidation and the quality of these repairs are of concern. In both VDOT Class A3 and Class A3 concrete and shotcrete repair systems, bonding of the repair material to the existing concrete is also a concern.

Self-consolidating concrete (SCC) is a comparatively new technology that provides very high workability (American Concrete Institute, 2007). SCC has been used in Japan and Europe advantageously since the early 1990s (Okamura and Ouchi, 1999). VDOT has used SCC successfully in beams, drilled shafts, and pier caps (Ozyildirim, 2009; Ozyildirim and Moruza, 2016; Ozyildirim and Sharp, 2012). SCC easily fills the congested spaces between the reinforcement and the formwork under the influence of its own mass and without the addition of consolidation energy, such as vibration. SCC also improves the quality of concrete by enhancing strength and reducing permeability, prolonging the repair life by minimizing the intrusion of chlorides that initiate corrosion. Other advantages of SCC include smooth surfaces, which eliminate the need for additional labor to finish the surface of concrete elements; a faster rate of concrete placement since there is no need for consolidation; good bonding; and improved safety with no or limited vibration, eliminating the need for protruding patches. But perhaps most important, SCC provides the opportunity for greater consistency in the quality of repair concrete.

Shotcrete quality is highly variable, and although there are examples of long-lived shotcrete patches, poor-quality repairs are present in all parts of Virginia. The high flow rate of SCC enables repair of the deteriorated concrete in such a way that it will match the geometry of the existing concrete and provide a quality repair material.

In addition to the issues discussed, another problem can occur in a repair area: the original concrete can have a different chloride content and pH than the repair material (VDOT Class A3 or Class A4 concrete, shotcrete, or SCC). Differences in chloride content and pH between the original and repair materials can lead to ring corrosion, where the reinforcement adjacent to the repaired area corrodes and the concrete cracks and spalls. To mitigate such corrosion, galvanic anodes have been used that contain zinc (Brown and Sharp, 2005). The zinc in galvanic anodes corrodes in place of the steel, thus protecting the steel from damage. It is also important to note that the galvanic anode system is self-contained and does not require an outside power source; however, the anodes have a limited life span, as they lose effectiveness once the zinc has passivated or is depleted.

PROBLEM STATEMENT

In substructure repairs, shotcrete or VDOT Class A3 or Class A4 concrete with conventional consistency is commonly used. With VDOT Class A3 or A4 concrete, the concrete quality, appearance of the patch, and formwork issues are of concern. Shotcrete has an advantage of not requiring formwork. However, qualifications of the operator, cost, and the quality of the end product have been issues of concern with shotcrete. SCC has high workability, enabling convenient placement, and has a good appearance. In these systems, corrosion of the reinforcement in or adjacent to the repair has been a concern.

PURPOSE AND SCOPE

The purpose of this study was to evaluate the use of SCC in repairing bridge substructure elements and the influence of galvanic anodes in mitigating corrosion of the reinforcement around the repair area to extend bridge life. Repairs using SCC with galvanic anodes have the potential to provide aesthetically pleasing, high-quality concrete that would increase the repair life of the area around the repair until the galvanic anode reached the end of its functional life.

Bridge structures to be repaired were selected in VDOT’s Lynchburg District and Staunton District. In the Lynchburg District, two bridges near Altavista on Route 699 over Hills Creek (State Structure No. 6091) and Route 712 over the Route 29 Bypass (State Structure No. 6229) were selected. These bridges are near each other, so the same plant provided concrete for both structures. The Route 699 Bridge was the first bridge repaired with SCC in Virginia, in 2010.

In the Staunton District, one bridge south of Staunton on I-81 over Folly Mills Creek (State Structure No. 2036) and another bridge near Alma on Route 340 over the South Fork of the Shenandoah River (State Structure No.1025) were selected. The substructures in these bridges were repaired with SCC, with Structure Nos. 6229 and 1025 also receiving galvanic anodes.

The repair areas, where galvanic anodes were used to minimize corrosion, were selected based on visual observation and sounding. Two of the structures, the Route 712 and Route 340 bridges, were instrumented to determine the current to and voltage at the galvanic anodes and to verify the effectiveness in delaying the onset of corrosion activity. In the instrumented structure on Route 340, anodes from three different manufacturers were placed in different piers to determine the effectiveness of each. SCC was tested for material properties at the fresh and hardened states.

Assessment of the repairs was performed before, during, and after completion of repairs. The assessment involved an investigation of the bonding or lack of delamination by soundings with a hammer, and the corrosion activity was determined through half-cell potential measurements and the anode efficiency by current measurements. Corrosion-related data were collected for up to 7 years for the Route 712 Bridge and for 3.5 years for the Route 340 Bridge. Although the dates of the repairs varied, these repair sites were visited and visually reevaluated for several years, with the final evaluation done in spring 2017.

METHODS

Overview

The study included field installations with supporting laboratory testing. Repairs in the field were made with SCC containing pozzolans, which reduce the permeability of concrete and improve its durability. Sacrificial zinc anodes were placed in some of the repair areas to provide galvanic protection. These were compared to the control areas, which contained no anodes. Condition evaluations were performed routinely using visual examination, sounding, and current and half-cell potential measurements of the structures.

The concretes in structures constructed before 1990 typically contain chlorides because of exposure to chloride spray from traffic, marine environments, or leaking joints, and they rarely contain a pozzolanic material that would reduce their permeability. However, SCC used in repairs does not contain chlorides from outside sources but includes a pozzolanic material for durability. Thus, the existing concrete and the SCC have different properties, including different chloride contents and pH values that could lead to ring corrosion. However, SCC containing pozzolans has high resistivity that could reduce the galvanic activity; anode manufacturers specify concretes with low resistivity for galvanic activity, which are generally concretes with high permeability. To mitigate this problem in repairs of concretes with low permeability, some manufacturers provide embedment mortar that allows for low resistivity next to the anode. Embedment mortar was used in the Route 712 Bridge. This study also included a high-quality SCC to provide low resistivity by adding calcium nitrite to SCC in the Route 340 Bridge. Calcium nitrite is an ionic material that reduces the resistivity of concrete without compromising the low permeability characteristic of the concrete with pozzolans.

The SCC was sampled during placement, and the quality was assessed in the fresh and hardened states using the appropriate ASTM standards shown in Table 1. The corrosion activity of the SCC repair areas and surrounding concrete was monitored using half-cell potential measurements in accordance with ASTM C876. Some of the anodes also had external connections to enable current measurements.

Table 1. Tests of Fresh and Hardened Concretes

Test	Specification
Fresh Concrete	
Slump flow, T_{20}	ASTM C1611
Slump flow with J-ring	ASTM C1621
Air content	ASTM C173
Density	AASHTO C 138
Hardened Concrete	
Compressive strength	ASTM C39
Elastic modulus	ASTM C469
Splitting tensile strength	ASTM C496
Modulus of rupture	ASTM C78
Permeability	ASTM 1202
Drying shrinkage	ASTM C157

Lynchburg District Bridges

In the Lynchburg District, two bridges at Altavista were repaired using SCC, one on Route 699 and the other on Route 712. The Route 699 Bridge repair did not include the installation of anodes.

Route 699 Bridge

A trial mixture was prepared on March 16, 2010, for both the Route 699 and Route 712 bridges. The mixture proportions are given in Table 2. The trial batch mixture had a high cementitious material content composed of Type I/II cement and fly ash. The coarse aggregate was crushed limestone, arch marble dolomite. The nominal maximum size was 3/8 in. The fine aggregate was natural sand with a fineness modulus of 2.3 and a void content of 51.2. The water–cementitious material ratio (w/cm) was low, at 0.39. However, the water content was high because of the high cementitious material content. Commercially available synthetic air-entraining admixture, regular water-reducing admixture, and high-range water-reducing admixture (HRWRA) were added. This mixture was also used in the substructure repairs.

Table 2. Trial Mixture Proportions for Route 699 and Route 712 Bridges

Ingredient	Amount (lb/yd^3)
Portland cement Type I/II	660
Fly ash	200
Coarse aggregate	1,323
Fine aggregate	1,220
Water	335
w/cm	0.39

w/cm = water–cementitious material ratio.

On the Route 699 Bridge, one-half of the backwall was placed with SCC on April 29, 2010. There were also buttresses with sharp corners that supported the backwall that were repaired with the SCC mixture. Anodes were not used in this structure. The SCC mixture was tested for fresh and hardened concrete properties.

A bucket attached to a track hoe was used to place the concrete, as shown in Figure 1. Once placed, the backwall was finished and then covered with plastic for curing, as shown in Figure 2.

Figure 1. Bucket Being Used to Place Concrete Into Backwall for Route 699 Bridge

Figure 2. Concrete Finishing Before Curing (*left*) and Plastic Covering (*right*) for Route 699 Bridge

Route 712 Bridge

Anode Placement

It was decided that the cap and columns in Pier 1 at the north end with the most damaged area would be selected for testing and observation on the Route 712 Bridge. The deteriorated concrete was removed with jack hammers. The 49-ft-long cap had three columns. The limits of removal of the chloride-contaminated concrete were determined by sounding that indicated delaminations. Figure 3 provides an example of the conditions, showing the bottom side of the northern portion of the pier cap before and after concrete removal. After removal of concrete, anodes with and without embedment mortar were placed in selected patch areas, as shown in Figure 4.

(a)

(b) (c)

Figure 3. Cap for Route 712 Bridge: (a) after sounding for delamination but before concrete removal; (b) north side; (c) south side after concrete removal in preparation for anode placement and SCC repair

Figure 4. Exposed Steel and Anodes With Embedment Mortar for Route 712 Bridge

In the repair area, the north side did not contain any anodes, the middle section had anodes with embedment mortar, and the south side had anodes without the mortar. The embedment mortar had high alkalinity, enabling an electrical circuit. The repair was performed in two phases on two separate days. The corroded steel was exposed, and about 1.5 in was cleared behind the reinforcement; 1.25-in-thick anodes were placed behind or next to the reinforcement along the perimeter against the existing concrete. A minimum clearance of 1 in was planned around the anodes or the steel to ensure the flow of the SCC except when embedment mortar was used. Embedment mortar filled the area behind the anode. The placement of anodes required that a deeper section be removed to allow the insertion of the anode with a minimum 1-in concrete cover.

To monitor the current output from the anodes, four anodes were selected and wired in a manner that allowed for current measurements.

SCC Placement

For the repair sections, plywood forms were used on flat surfaces and tubular form material was used on the round surfaces. A funnel shown in Figure 5 held at bridge level was used to place the concrete, which was carried to the repair sites by gravity with the use of a flexible tube. At some intermediate points in the forms on the pier cap, buckets were used to place the SCC, as shown in Figure 5.

Figure 5. Funnel Being Used to Gain Pressure From Height Difference During Placement (*left*) and Manual Placement Method Using Container and Bucket (*right*) for Route 712 Bridge

Staunton District Bridges

In the Staunton District, two bridge structures were repaired using SCC: the I-81 Bridge and the Route 340 Bridge, which also contained anodes.

I-81 Bridge

A trial batch of concrete for the I-81 Bridge was prepared on July 22, 2011. The mixture proportions shown in Table 3 were selected for use in the substructure repair of the bridge. Type

I/II cement with Class F fly ash was used. The coarse aggregate was No. 8 pea gravel. The fine aggregate was natural sand. Air-entraining admixture, workability retaining admixture, and HRWRA were added.

There were two adjacent bridges, each with two piers. A pier cap and column in the bridge carrying the northbound traffic were repaired using SCC. Three other piers were repaired with shotcrete to serve as a comparison against the SCC. A pump was used to place the SCC.

The repaired pier during placement is shown in Figure 6. The SCC was placed using a small pump located on the bridge, also shown in Figure 6. Pumping allowed the SCC to flow fast, and the pump pressure enabled easy placement.

Because of the extended placement time, the column repairs, which were done after the pier cap repairs, required internal vibration to consolidate the concrete because it was losing workability. This process is shown in Figure 7.

Table 3. Trial Batch Mixture Proportions for I-81 Bridge

Material	Amount (lb/yd^3)
Portland cement Type I/II	525
Fly ash	225
Coarse aggregate	1,320
Fine aggregate	1,404
Water	267
w/cm	0.36

w/cm = water–cementitious material ratio.

Figure 6. Deteriorated Pier of I-81 Bridge During Placement (*left*) and Small Pump on Deck (*right*)

Figure 7. Column Being Consolidated With Internal Vibrator for I-81Bridge

Route 340 Bridge

Anode Placement

Anodes from three different manufacturers were installed in an attempt to determine the effectiveness of these anodes in preventing corrosion. Each manufacturer had six to eight anodes placed on two opposite sides of two of the piers, Piers 4 and 5. In order to create a basis of comparison, similar concrete with calcium nitrite was used in every location where anodes were placed; some locations were left with no anodes as a control group.

SCC Placement

SCC was used to repair three piers of the bridge, as shown in Figure 8. The bridge has six piers, three of which are in the river. The piers on land were repaired with SCC, with anodes placed in selected repair areas.

Trial mixtures were made that led to the selected mixture proportions shown in Table 4 for the Route 340 Bridge. In the trial mixtures, a w/cm of 0.40 was used, but during placement, the w/cm was lowered to 0.34 by the producer. This mixture also contained an air-entraining admixture, a retarding admixture, calcium nitrite, and HRWRA.

Figure 8. Route 340 Bridge

Table 4. Mixture Proportions for Route 340 Bridge

Material	Amount (lb/yd^3)
Portland cement Type I/II	546
Fly ash	182
Coarse aggregate	1,414
Fine aggregate	1,364
Water content (design)	292
Water content (actual)	244
Calcium nitrite (gal)	4
w/cm (design)	0.40
w/cm (actual)	0.34

w/cm = water–cementitious material ratio.

RESULTS

The literature survey indicated limited use of SCC in bridge substructure repairs nationwide, which led to this investigation. However, the survey indicated that SCC has the potential for use in substructure repairs because of its high workability and improved properties. Laboratory testing was conducted to ensure that SCC could be prepared with high slump flows (preferably a minimum of 25 in in accordance with ASTM C1611) to enable flow in narrow, congested areas. The minimum specified compressive strength in the substructure repairs was 3,000 psi. The desired flow and required strengths were achieved, and the concretes were stable (not segregating). The laboratory study confirmed that workable concretes having high strength and low permeability and not needing mechanical consolidation can be produced. Laboratory work also showed that SCC is sensitive to water and can lose stability (exhibit segregation) at high water contents. Subsequently, the field installations were performed.

Lynchburg District Bridges

The Route 699 Bridge and then the Route 712 Bridge were repaired in 2010. The fresh and hardened concrete properties were determined. The trial mixture data for both bridges shown in Tables 4 and 5 indicated good flow and strength.

Route 699 Bridge

The SCC for the Route 699 Bridge was tested at the jobsite and had a high slump flow (30 in) without any segregation. The 28-day strength was 7,610 psi. The repaired backwall showed a very smooth finish, as shown in Figure 9. The buttresses had sharp corners. Since this bridge repair did not include anodes, corrosion measurements were not made.


Figure 9. SCC Repaired Backwall (*left*) and Buttress (*right*) for Route 699 Bridge

Route 712 Bridge

SCC Placement

On May 27, 2010, concrete was delivered with a 29-in slump flow. It was segregating in the wheelbarrow. It was mixed in the wheelbarrow with a scoop to blend the ingredients, but when mixing stopped, the aggregates settled. This batch was rejected because of segregation. The next batch was sufficiently stable for use. It had a low slump flow of 23.2 in by the J-ring test. The goal was to exceed 25 in by the regular slump flow test without the J-ring. J-ring test values tend to be lower than the regular slump flow because of the presence of the ring. ASTM C1621 indicates that a difference between the regular slump flow and J-ring test values of less than 1 in indicates good passing ability and more than 2 in indicates poor passing ability. The fresh and hardened concrete properties are shown in Tables 5 and 6 as Batch 1.

Trial mixtures were made that led to the selected mixture proportions shown in Table 4 for the Route 340 Bridge. In the trial mixtures, a w/cm of 0.40 was used, but during placement, the w/cm was lowered to 0.34 by the producer. This mixture also contained an air-entraining admixture, a retarding admixture, calcium nitrite, and an HRWRA.

Table 5. Fresh Concrete Properties for Route 712 Bridge

Batch	Trial	Batch 1	Batch 2	Batch 3	Batch 4
Date	**03/16/10**	**05/27/10**	**06/09/10**	**07/21/10**	**08/18/10**
J-ring (in)	24.5	23.2	28.0	27.5	30.0
T_{20} (sec)[a]	2.5	3	2	2	1.5
Air content (%)	9.5	11.5	8.0	5.2	7.0
Density (lb/ft^3)	132.6	129.6	136.4	136.4	132.0
Concrete temp. (°F)	62	86	82	89	84

[a] Time to reach 20-in flow.

Table 6. Hardened Concrete Properties for Route 712 Bridge

Property	Age	Trial	Batch 1	Batch 3	Batch 4
Compressive strength (psi)	1 day	2,350	2,800	2,780	2,050
	7 days	5,250	4,240	4,170	3,350
	28 days	6,720	5,430	5,130	4,090
Elastic modulus (x10^6 psi)	1 day	----	----	2.74	1.87
	7 days	3.65	3.50	----	2.95
	28 days	4.10	3.85	3.87	----
Splitting tensile strength (psi)	7 days	525	445	----	420
	28 days	585	530	----	515
Flexural strength (psi)	28 days	790	955	925	635
Permeability (C)	28 days	1,131	844	1906	2055

Batch 1 had delays in placement and exhibited workability loss; it also had a low slump flow at the beginning. Therefore, on June 9, 2010, slump flow was increased and a workability retaining admixture was added at the plant at the recommended dosage of 4 oz/cwt. The bridge site was about 1 mile from the plant. As indicated in Table 5, the slump flow with J-ring was 28 in and the air content was 8% at the site. Some of the concrete was kept at the wheelbarrow for testing at the completion of the job to determine the slump retention achieved. The air temperature was 74 °F when tested at the beginning of placement. At the completion of placement, the air temperature was 88 °F. The concrete was in the wheelbarrow for 75 min and had a temperature of 84 °F. The concrete was mixed with a scoop and tested for consistency. The slump flow with J-ring was 24.5 in, and the time to the 20-in spread (T_{20}) was 3 sec, which indicated SCC still with satisfactory flow at the end of the placement. The air content was also satisfactory at 6.8%.

During the first portion of SCC placement, the concrete traveled about 20 ft and was deposited from the top with a high head, as shown in Figure 5. On one side of the cap, the repair area covered the entire face, whereas only the lower part of the cap face needed repair on the other side. At the ends, there were ports to enable expulsion of air and access to the repair area. Holes were drilled in multiple areas across the forms to expel air and to allow the flow of SCC to be monitored. During placement on the side that had a repair area extending the entire height, the form began to bulge. Therefore, the placement was stopped and the area was shored and strengthened. The time spent on strengthening the forms, about 15 min, caused stiffening of the SCC so that when placement resumed, the concrete did not flow easily and did not reach the ends. The contractor placed the stiffening concrete manually through the ports at the ends and used vibrators to ensure proper consolidation.

On July 21, 2010, two batches of concrete were used to repair the remaining one-half of the pier. The first batch was 2.5 yd^3 of concrete. It had a slump flow with J-ring of 24.2 in, with a T_{20} of 2 sec, indicating satisfactory flow. The concrete temperature was 89 °F, and the air temperature was 90 °F. The air content was 5.4%, and the unit weight was 138.0 lb/ft^3. When there was 2 yd^3 of concrete left in the truck, more HRWRA was added to increase the slump flow. The resulting slump flow with J-ring was 27.5 in, as shown as Batch 3 in Table 5, which indicated high flow. For placement, rather than use of the tube with a funnel, SCC was discharged into a large container, as shown in Figure 5. The container was raised to the level of the cap. Then, concrete was filled into buckets and poured through the ports in the formwork. This created problems since there was not enough head or pressure to help the SCC flow through the narrow openings and the congested reinforcement. In addition, slump loss made flow more difficult. Therefore, during placement, slump flow loss necessitated the use of vibrators to ensure proper consolidation.

On July 29, 2010, the concrete repair was inspected and large voids were observed on the top and bottom of the repair area placed on July 21, 2010. The ports to place the concrete were built too low, causing voids on the top, as shown in Figure 10. At the bottom, the reinforcement was close to the surface with shy cover between the bottom of the form and the steel, which would hinder the flow of SCC. In addition, pouring with small buckets did not provide enough pressure to assist with flow, causing a large void at the bottom, as shown in Figure 10.

Concrete was removed from the area with the large voids. After removal of the old SCC repair, new anodes were placed on August 18, 2010. This was done in a manner similar to that of the initial placement. The new formwork provided a continuous open chute at the high end of the repair area on both sides, and a larger clearance between the bottom form and the steel was created by lowering the formwork from the initial level, as shown in Figure 11.

Figure 10. Ports Lower Than High Points (*left*) and Void at Bottom With Shy Cover (*right*) for Route 712 Bridge

Figure 11. More Cover Depth in Repair Area of Route 712 Bridge Provided by Lowering of Formwork: Overview (*left*) and Measured Difference of Protrusion (*right*)

Although increasing the cover depth was necessary, extending the new surface to the column or to an edge would have provided a more aesthetically pleasing view. The first batch of 2 ft^3 of concrete delivered had a very high air content (15%) and was rejected. The second batch had a high flow rate (30 in) and an acceptable air content (7%) and thus was used. With the truck positioned under the bridge, SCC was placed in a container and lifted near the repair area. SCC was filled into the small buckets by dipping and then poured into the opening. The placement was completed in about 30 min. The slump flow at the completion of the project was 26.8 in. The forms were removed on Monday, August 23, 2010, and the inspection revealed sound concrete with no delaminated areas. As shown in Figure 12, except for the drop shown in Figure 11, the repaired pier cap and column looked satisfactory and matched the geometry well.

Figure 12. Repaired Cap and Column of Route 712 Bridge

The average compressive strengths of concretes ranged from 4,090 to 5,430 psi, as shown in Table 6. They were satisfactory but lower than for the Route 699 Bridge. This indicates the high variability in properties in SCC attributable mainly to its sensitivity to water content. The high water content in the samples from the Route 712 Bridge is also reflected in the high shrinkage values obtained for these concretes, as shown as Altavista 1 and 2 in Figure 13.

Even though the shrinkage values were high, there were no wide cracks in the repaired areas after the completion of the repairs because of the heavy reinforcement and limited size of the repair area. In a field survey on March 8, 2017, about 7 years after placement, cracks were observed, as shown in Figure 14. The cracks were tight and their spacing ranged from 1 to 4 ft.

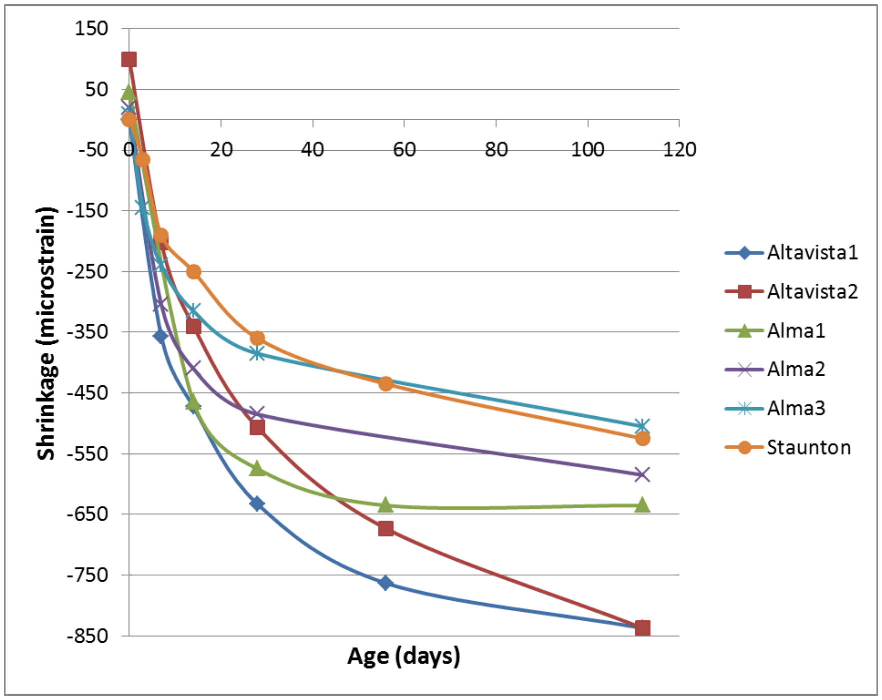


Figure 13. Shrinkage Data for Route 712 (Altavista), Route 340 (Alma), and I-81 (Staunton) Bridges

Figure 14. Tight Shrinkage Cracks in Repair Area of Route 712 Bridge in 2017

Condition Assessment

After 7 years, a condition survey was conducted to determine the benefit of galvanic point anodes. The repair areas themselves, both with and without anodes, did not exhibit corrosion activity and were well bonded to the existing concrete, although cracks in the repair concrete were evident in some locations. In the original concrete adjacent to the repair, some deterioration was noted, but where anodes were located, the anodes did provide protection to the steel adjacent to the repair area at a distance of about 6 in away. Therefore, the anodes do provide a benefit with an anodic region that is adjacent to the cathodic patch, the "halo effect." However, it was observed that in chloride-contaminated concrete areas away from the vicinity of the repair area, the contaminated concrete showed distress and required new repairs.

Measurements of current over time were gathered from four specially modified anodes. Initially, these anodes had differing current outputs, as shown in Figure 15. About one-third more charge was passed by the anodes that started at the higher current; however, after approximately 3 years, the current outputs of all anodes decreased to an equivalent level.

There are efforts by the manufacturers to increase the amount of zinc in the anodes to extend the time of protection with galvanic anodes. However, it is important to recognize that the galvanic cathodic protection (CP) systems will have a limited driving voltage and protection area as compared to an impressed current CP system. This is because the potential difference for galvanic CP systems is fixed by the voltage difference between the anode material selected for the galvanic systems and the steel that is being protected. Further, galvanic anodes can lose efficiency over time, so additional zinc may not provide for a much longer life. A properly designed impressed CP system, however, can be adjusted to provide sufficient protection of the steel. Therefore, the CP options for repair might be either to increase the number of galvanic anodes that are distributed in the old and new concrete or to select a CP system that has a greater driving voltage. Therefore, proper CP design is important, and the limited driving voltage of the galvanic system must be considered when the repair is made.

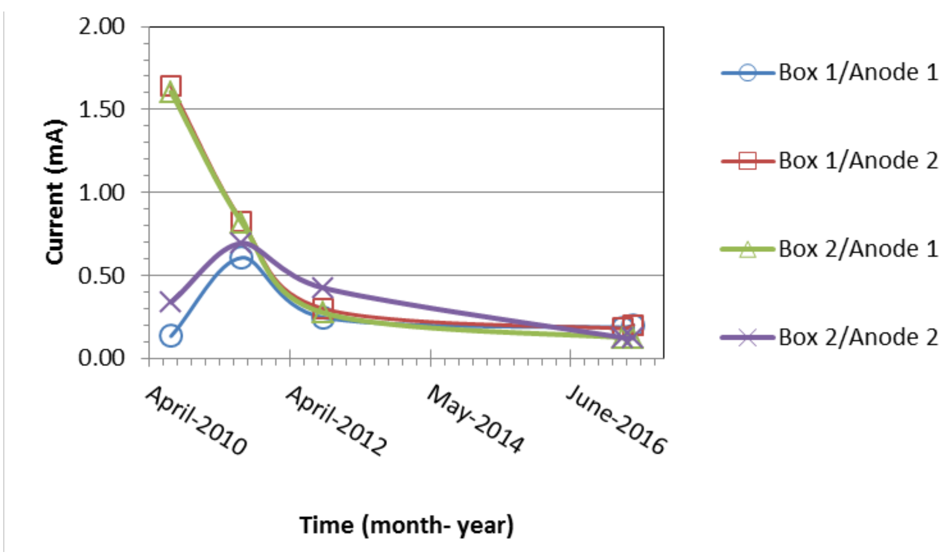


Figure 15. Current Measured at Four Anode Locations on Route 712 Bridge Over Time

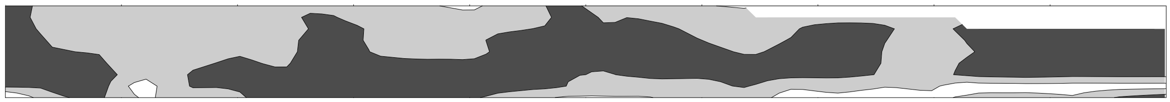
The limitations of the ability of galvanic anodes that were placed only in the patch repair to overcome insufficient removal of chloride-contaminated concrete were demonstrated at the Route 712 Bridge. Half-cell potential measurements gathered prior to the SCC repairs indicated an average half-cell value along the top of the cap of −384 mV vs. saturated copper sulfate electrode (CSE), with a standard deviation of 33 mV. Half-cell potential measurements made at the same time along the bottom of the cap showed average potential values of −406 mV vs. CSE, with a standard deviation of 69 mV. Although the chloride-contaminated concrete was removed from the bottom of the cap, concrete in the top region of the cap was left in place even though the half-cell values indicated a greater than a 90% probability of corrosion (based on values more negative than −350 mV vs. CSE). This was due to the fact that traditional sounding was used to determine where concrete should be removed rather than a direct assessment of the corrosion risk. The traditional sounding technique is limited to detecting delaminated concrete and will not detect a corroded area that has not yet delaminated.

To monitor the probability of corrosion in the original concrete, half-cell data were gathered during July and August over several years. These data were converted into potential maps and are shown in Figure 16. Although anodes were placed in selected regions along the bottom of the cap, the upper region, which exhibited corrosion risk based on half-cell measurements, did not receive any anodes. In Figure 16, it is clear that there is a high probability of corrosion in several regions where concrete was not removed. These regions are also outside the area of influence of the anodes that were placed along the bottom of the cap.

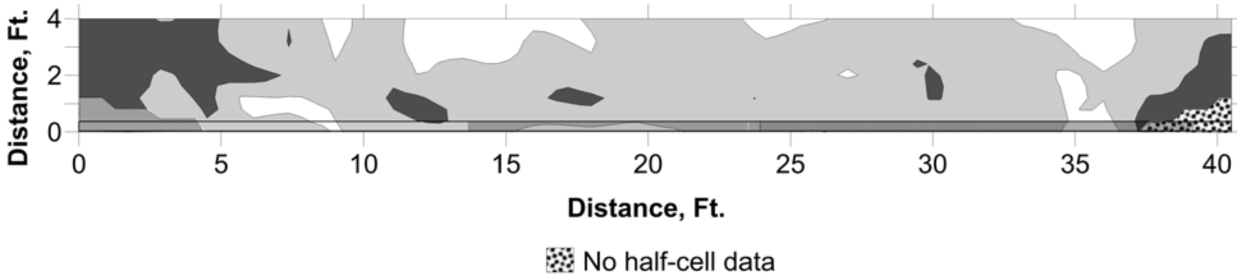
On May 5, 2017, an assessment was performed to focus on the region of repair and compare it to other regions exhibiting damage. The temperature on that day started slightly cooler for May, with an approximate high of 65 °F. Although this might have decreased the magnitude of the half-cell reading, as noticed in Figure 16 with increasing age, it also allowed for easier observation of cracks in the pier cap.

Cracks were observed in the upper region of the pier cap, which was especially noticeable below the bridge seat, as shown in Figure 17a. Cracking was also obvious along the end of the northern cap, as shown in Figure 17b. More negative half-cell potential readings were seen in regions with cracking, which is consistent with the higher probability of corrosion activity that is expected according to ASTM C876.

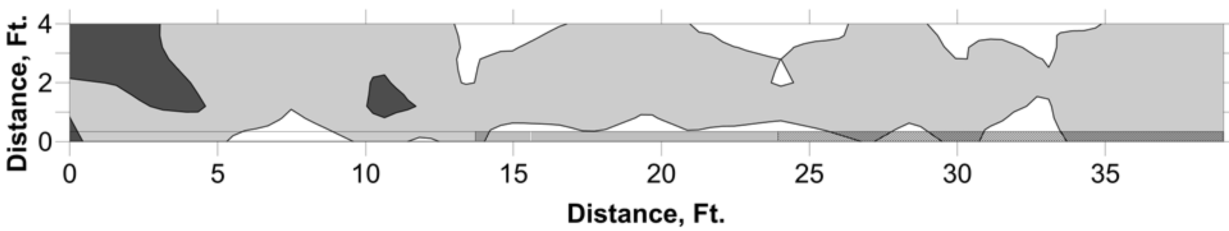
To visualize better the correlation between the half-cell potential readings and the crack locations, the half-cell potential map contour intervals based on data gathered on May 5, 2017, were adjusted and the crack locations in the original concrete were overlaid to create Figure 18. Although the lower portion of the structure where the repair was made several years ago is doing well and showing only smaller periodic cracks with no indication of corrosion, the older chloride-contaminated concrete that was not removed has continued to corrode and is now exhibiting damage.



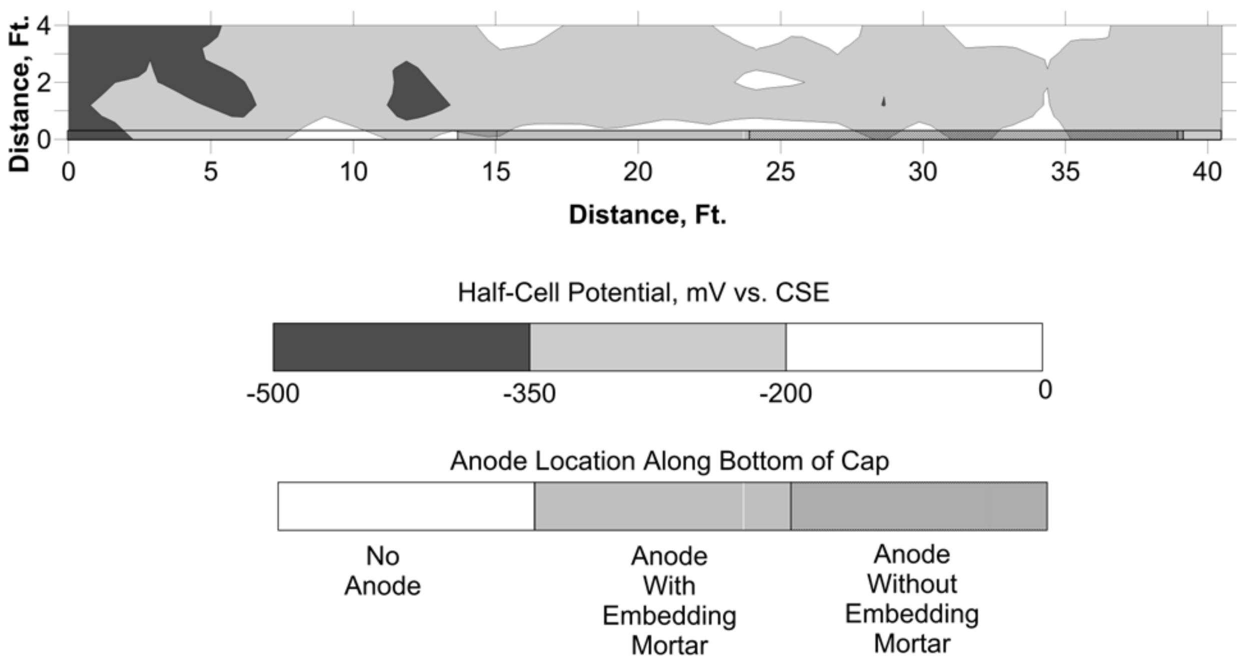
(a) Summer 2010 (Prior to Repair)



(b) 2011 (Post Repair)



(c) 2013 (Post Repair)



(d) 2014 (Post Repair)

Figure 16. Half-cell Potential Map of Road-Side Cap Face of Route 712 Bridge: (a) mid-summer2010 (includes some wrapping of cap and missing data); (b) 2011 (no data first 36 in and last 18 in along lower corner regions); (c) 2013 (no data at the end); (d) 2014. Anode locations are described earlier, with the coordinate 0,0 in these figures, indicating the most northern bottom region of the pier cap.

(a)

(b)

Figure 17. Cracks Evident in Upper Region of Pier Cap of Route 712 Bridge on Locations Separated From Embedment Anodes. Image of cracking in interior region is shown in (a), and cracks in end cap are shown in (b).

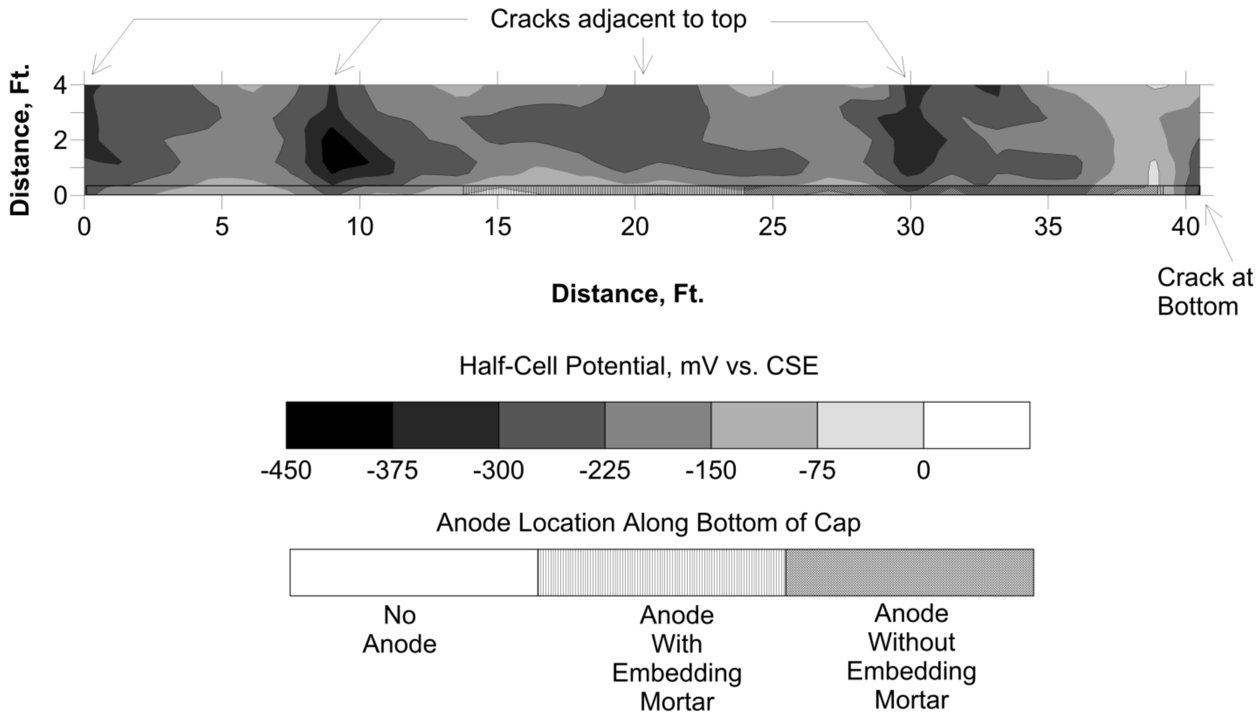


Figure 18. Recent Half-Cell Potential Map Showing Crack Location in Unrepaired Area Along Vertical Face of Pier Cap of Route 712 Bridge. Anode locations are described earlier, with the coordinate 0,0 in these figures indicating the northern bottom end of the cap.

This raises questions about the cost-effectiveness of always using galvanic anodes in repairs when adjacent chloride-contaminated concrete is not removed. From what was observed at Altavista, if the area with chloride-contaminated concrete is limited, as in the case of the end of a pier cap, galvanic anodes could be beneficial. However, if the chloride-contaminated concrete area is located too far away from the patch repair area, galvanic anodes will most likely not mitigate corrosion in the contaminated area. This will result in the need for future repairs in those remote chloride-contaminated areas. Moreover, in some cases, galvanic CP will not suffice, so other repair techniques, such as impressed current CP or even removal of a greater volume of chloride-contaminated concrete, should be considered.

Staunton District Bridges

I-81 Bridge

The fresh and hardened properties of the trial batch of concrete are shown in Table 7. The mixture contained a workability retaining admixture. The slump flow values with age show that the concrete remained self-consolidating during the repair.

Two batches of concrete were tested during placement, and the properties are summarized in Table 8. The strength values were varying, but satisfactory. The shrinkage values were lower than for Altavista as expected since the water and paste contents were lower.

Table 7. Fresh and Hardened Concrete Properties of Trial Batch for I-81 Bridge

Property	Age	Value
Slump flow (in)	15 min	28
	30 min	26¾
	45 min	25½
Air content (%)	-	5.4
Compressive strength (psi)	3 days	4,420
	7 days	4,870

Table 8. Fresh and Hardened Concrete Properties for I-81 Bridge

Property	Age	Batch 1 8/26/11	Batch 2 9/1/11
Air (%)	[a]	5.8	4.6
T_{20} (sec)	[a]	1.9	1.5
J-ring (in)	[a]	23	31
Density (lb/ft^3)	[a]	138.0	1,43.2
Compressive strength (psi)	1 day	2,150	2,030
	7 days	3,620	3,820
Splitting tensile strength (psi)	28 days	4,850	5,770
	7 days	370	430
Permeability (C)	28 days	490	460
	28 days	1,503	----

[a] Within 15 min.

The SCC repair was made to a pier cap and column. Three other pier caps were repaired at the same time using shotcrete. The repair with SCC resulted in a much smoother finish and was less noticeable aesthetically. Figure 19 shows the surface of the SCC and the shotcrete.

Figure 19. SCC With Smooth Surface (*left*) and Shotcrete Rough Surface (*right*) of Route 712 Bridge

Route 340 Bridge

Anode Placement

Anodes are present on both Pier 4 and Pier 5. On August 2, 2013, concrete was placed on the lower portion of Pier 4, and it partially covered the anodes, leaving two exposed in the upper portion of the pier. Because not enough time had elapsed, the recent condition survey conducted at 3.5 years would not have discerned differences between areas with and without the galvanic zinc anodes (experimental and control sections) adjacent to or near each other. Therefore, a follow-up study is recommended.

SCC Placement

Initially, the SCC placement was planned to be performed by pumping on the first pier (Pier 6) on July 19, 2013, but the pump malfunctioned. In addition, the forms bulged, so the placement was halted. Subsequently, the height of placement was limited to 8 ft and forms were secured more firmly to the piers. The air content of the mixture was high. To complete the upper portion of the first pier, the mixture was adjusted to reduce the air content by decreasing the amount of the air-entraining admixture. The pump never functioned, and placement resumed through the use of small buckets and manual pouring of the SCC from the top of the forms. The mixture was very workable and retained its workability until the completion of the project.

The fresh concrete properties of the SCC used in the piers of the Route 340 Bridge are shown in Table 9, which summarizes the results of tests conducted at the beginning, middle, and end of placement to ensure the concrete maintained the consistency of SCC during placement. A minimum slump flow of 25 in and a minimum air content of 4% were desired. When tested at the beginning, if the minimums were not achieved, admixtures were added to meet the minimums.

Table 9. Fresh Concrete Properties for Route 340 Bridge

Pier No.	**Pier 6**					**Pier 5**		
Region of Pier	**Lower**		**Upper**			**Lower**		
Date (Amount)	**7/19/13 (3.5 yd^3)**		**7/23/13 (2 yd^3)**			**7/26/13 (3 yd^3)**		
Time of Tests (A.M.)	**8:10**	**9:26**	**8:15**	**8:30**	**9:25**	**8:25**	**8:45**	**9:45**
Sample[a]	**1-a**[b]	**1-b**	**2-a**	**2-b**[b]	**2-c**	**3-a**[b]	**3-b**	**3-c**
Slump flow (in)	27.5	25.5	20.0	25.0	24.0	28.5	29.0	26.0
T_{20} (sec)	1.5	2.0	3.1	1.6	3.9	2.0	1.0	1.5
VSI	1.0	0.5	0.5	0.5	0.0	1.0	1.0	0.5
Air (%)	7.9	9.4	8.0	10.5	6.0	2.4	9.0	7.0
Density (lb/ft^3)	136.8	135.2	138.4	133.6	142.0	146.8	134.8	140.0
Concrete temp. (°F)	84	83	78	78	78	-	-	74

VSI = visual stability index.

[a] The letters a, b, and c indicate if tests were done at the beginning, at a middle point, or at the end of placement, respectively.

[b] Samples from which concrete for hardened specimens was taken.

The 28-day strength of Sample 2 with a high air content as shown in Table 9 was 7,010 psi (Table 10), much higher than the 3,000 psi required. Thus, an increase in air content was not expected to be of any concern. The compressive strengths of the other two batches given in Table 10 were high, averaging more than 5,500 psi.

On Friday, August 2, 2013, 3 yd^3 of SCC was batched at 9:15 A.M. and delivered to the jobsite. Table 11 shows the fresh concrete properties of this batch, Sample 4. This batch was distributed between the lower portions of Pier 5 and Pier 4 and a portion of Pier 6. Additional air-entraining admixture was included in the mixture after the initial measurement of air content for Sample 4-a.

On Friday, August 9, 2013, 3 yd^3 of SCC was batched at 6:55 A.M. and delivered to the jobsite. The fresh concrete properties of this mixture are listed as Sample 5 in Table 11. The piers that received the SCC repair were the upper rear of Pier 6, the upper rear of Pier 5, and the middle front and middle rear of Pier 4.

Strength and permeability values of the samples from Sample 4 and Sample 5 for the Route 340 Bridge are presented in Table 12.

Table 10. Hardened Concrete Properties for Route 340 Bridge

Pier No.	**Pier 6**		**Pier 5**
Region of Pier	**Lower**	**Upper**	**Lower**
Date Cast	**7/19/13**	**7/23/13**	**7/26/13**
Sample No.	**1**	**2**	**3**
Compressive strength (psi)			
3-day	3,480	2,600	3,730
7-day	3,600	5,250	3,960
28-day	5,580	7,010	5,940
Permeability (C) 28-day	1358	384	978

Table 11. Fresh Concrete Properties for Route 340 Bridge

Pier No.	**Pier 4, Pier 5, Pier 6**					
Date (Amount)	**8/2/13 (3 yd^3)**			**8/9/13 (3 yd^3)**		
Time of Tests (A.M.)	**10:30**	**10:42**	**12:00**	**8:25**	**8:33**	**9:24**
Sample No.[a]	**4-a**[b]	**4-b**	**4-c**	**5-a**	**5-b**	**5-c**[b]
Slump flow (in)	29	28	26.25	27	28.5	26
T_{20} (sec)	3.03	1.97	2.43	1.48	1.13	2.84
VSI	1.0	1.0	0.0 – 0.5	-	-	-
Air (%)	3.8	9.0	7.2	3.0	5.9	4.3
Density (lb/ft^3)	143.2	136.0	138.8	143.2	137.6	142.4
Concrete temp. (°F)	80.5	83.0	83.0	82.0	82.3	79.0

VSI = visual stability index.

[a] The letters a, b, and c indicate if tests were done at the beginning, at a middle point, or at the end of placement, respectively.

[b] Samples from which concrete for hardened specimens was taken.

Table 12. Hardened Concrete Properties for Route 340 Bridge

Pier No.	**Pier 4, Pier 5, Pier 6**	
Cast Date	**8/2/13**	**8/9/13**
Sample No.	**4**	**5**
Compressive strength (psi)		
3-day	3,610	3,420
7-day	4,410	5,040
28-day	5,860	6,000
Permeability (C) 28-day	1026	1596

On August 15, 2013, air-entraining admixture was added between the readings taken for Sample 6-a and Sample 6-b. On August 23, 2013, air-entraining admixture was added between readings taken for Sample 7-a and Sample 7-b. The air meter was not working; therefore, air contents were estimated based on density. The fresh concrete properties are given in Table 13, and the hardened properties in Table 14.

The results for the Route 340 Bridge show that the concretes, in general, had the high workability needed for placement in narrow areas and locations with heavy reinforcement and that they were stable with a visual stability index of 1 or less. There was high variability in air content, density, strength, and permeability. However, the 28-day compressive strengths were much higher than 3,000 psi, ranging from 4,560 to 7,010 psi with an average of 5,726 psi and a standard deviation of 766 psi, and the coefficient of variation was 13%. The permeability values were low or very low except for the last batch tested, which had a permeability in the moderate range. However, all values were less than the 3500 coulomb requirements for VDOT Class A3 concrete. This last batch also had the lowest density and compressive strength. The shrinkage values were also variable, as shown in Figure 13, based on three batches of concrete tested.

Even though the results of SCC testing showed that the product was highly variable, concretes used in the structure are air entrained and have the strength and permeability values to provide the expected service life. The relationship between air content and density for all projects is shown in Figure 20. There is a good correlation, and as expected, air content increases with a reduction in density.

Table 13. Fresh Concrete Properties for Route 340 Bridge

Pier No.	**Pier 4, Pier 5, Pier 6**					
Date (Amount)	**8/16/13 (3yd^3)**			**8/23/13 (3 yd^3)**		
Time of Tests (A.M.)	**8:05**	**8:19**	**10:17**	**8:05**	**8:25**	**9:50**
Sample No.	**6a**	**6b**	**6c**[a]	**7a**	**7b**	**7c**[a]
Slump flow (in)	30	31	29	-	29.5	28.5
T_{20} (sec)	1.75	1.10	1.07	-	0.47	0.50
Air content (%)	3.7	7.0	6.9	[b]	[b]	[b]
Density (lb/ft^3)	138.8	134.8	134.8	139.6	132.0	129.6
Concrete temp. (°F)	68	67	69	76	76	74

[a] Samples from which concrete for hardened specimens was taken.
[b] Since the air meter malfunctioned and did not take readings, air content was estimated based on density.

Table 14. Hardened Concrete Properties for Route 340 Bridge

Pier No.	**Pier 4, Pier 5, Pier 6**	
Cast Date	**8/16/13**	**8/23/13**
Sample No.	**6**	**7**
Compressive strength (psi)		
3-day	3,340	2,760
7-day	3,590	3,630
28-day	5,130	4,560
Permeability (C) 28-day	1259	3221

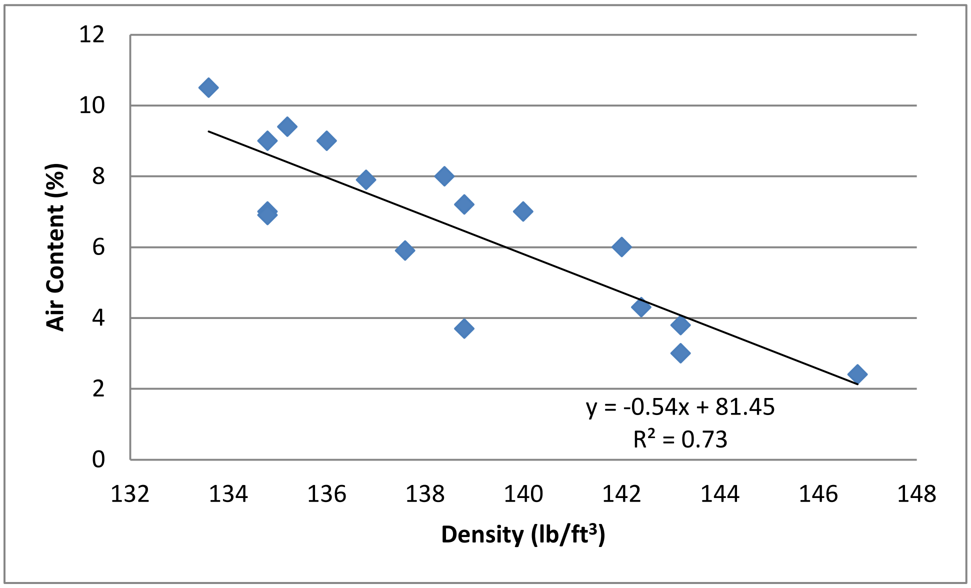


Figure 20. Density vs. Air Content for the SCC Mixtures

CONCLUSIONS

- *Despite the earlier placement problems that occurred in the field, the repairs with SCC were completed successfully and the repaired areas do not exhibit corrosion activity.*

- *SCC had high variability in air content, slump flow, strength, and permeability, indicating sensitivity to water; however, satisfactory values exceeding the minimums specified were obtained.*

- *Form bulging with SCC can be prevented by securing forms firmly and restricting the height of the placement.*

- *SCC can lose workability during placement. Slump flow checks during placement would indicate if SCC still had the desired workability. If the workability is reduced, mechanical vibration should be applied.*

- *With the use of workability retaining admixtures or retarding admixtures, workability of the SCC can be maintained during placement.*

- *SCC placement is facilitated by applying pressure through pumping or by pouring from a height. Placement by pouring from small buckets without any height would not provide the required flow through congested areas and narrow openings.*

- *After 7 years, chloride-contaminated concrete that was left in place during the original repair has initiated corrosion and requires repair.*

- *Seven years of experience has shown that anodes do provide protection to the steel in a limited area adjacent to the repair, but areas outside this limited region are not protected; corrosion is progressing in these unprotected areas and requires repair.*

RECOMMENDATIONS

1. *VDOT's Structure and Bridge Division and Materials Division should allow the use of SCC for substructure repairs.*

2. *VDOT's Structure and Bridge Division should provide guidance for how and when to use SCC for repairs.*

3. *VDOT's Structure and Bridge Division and Materials Division should consider replacing concrete surrounding reinforcement that shows corrosion activity. This should be done using high-quality SCC concretes, with or without anodes, to encapsulate the reinforcement completely. This option should be an alternative to making repairs with shotcrete or VDOT Class A3 concrete with anodes.*

4. *VTRC should continue to monitor the corrosion activity in the substructure elements that were repaired with and without anodes to determine how long the repairs last and corrosion is delayed.*

BENEFITS AND IMPLEMENTATION

Benefits

SCC is a workable quality concrete that can eliminate the commonly encountered consolidation problems. Poor consolidation results in reduced strength and durability, adversely affecting service life.

The high workability of SCC enables it to be an option in substructure repairs, thus providing another repair material in addition to shotcrete and concrete with regular consistency. Shotcrete placement requires skilled operators, and the surface finish is rough. Regular concrete is difficult to place in narrow openings and usually requires a protruding raised section that does not fit the rest of the geometry of the bridge element.

Implementation

Recommendation 1 has been implemented. VDOT's 2016 *Road and Bridge Specifications* (VDOT, 2016) include the use of SCC or VDOT Class A4 concrete for substructure surface repairs.

With regard to Recommendation 2, VDOT's Structure and Bridge Division will provide guidance for SCC for repairs in a special provision or best practice guide to be developed by the end of FY18.

With regard to Recommendation 3, the Structure and Bridge Division will provide guidance in the selection of repair areas and the use of anodes to be developed by the end of FY18. VDOT will consider selecting repair areas with preference for half-cell potential readings rather than only soundings for delaminations. Anodes should be used where a local area is distressed and application of a sacrificial coating or extensive concrete removal is not needed.

With regard to Recommendation 4, VTRC will make plans for visual observation and measurements of the current and voltage readings at 2-year intervals until repairs fail.

ACKNOWLEDGMENTS

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