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BRIDGE ASPHALT OVERLAY WITH WATERPROOFING MEMBRANE EFFECTIVENESS STUDY US-75/77 NEAR HOMER, NEBRASKA

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Abstract

When deicing chemicals penetrate bridge deck concrete to the depth of reinforcing steel, corrosion can occur causing bridge deck deterioration. Application of an asphalt overlay on top of a waterproofing membrane (ACC&M) applied to a bridge deck is one method to protect bridge decks from moisture and chlorides and extend the service life of bridge decks. To better understand the effectiveness of this deck preservation method, NDOT contracted with Vector Corrosion Services (VCS) to conduct an in-depth analysis of two bridges that had been in service with ACC&M for over forty years. The bridges that were selected had regions that were known to have had chloride exposure for more than 20 years prior to placement of the ACC&M and also regions that had ACC&M placed prior to opening to traffic and would be expected to have no chloride exposure if the waterproofing membrane was not breached. This circumstance allows for a case study to compare ACC&M effectiveness for preserving bridge decks that 1) have had previous chloride exposure and 2) have not had previous chloride exposure.

The bridges and their history

S075 17062 - a 60 foot, three-span concrete slab bridge constructed in 1938[1]
S075 17596 - a 152 foot three-span steel girder bridge constructed in 1933

Both bridges

- Carry traffic on US Highway 77 (concurrent with US 75 on this segment) and had a 2018 average daily traffic count of 6,630 vehicles per day with 13% heavy vehicles.
- The 1930s era bridge decks begin to be exposed to increased chloride concentrations in the mid-1950s as use of deicing treatments became more widespread
- In 1974 the bridges were widened by adding additional deck to each side and the existing deck and an asphalt overlay with a waterproofing membrane (ACC&M) was placed on the original and widened areas of the decks
- In 2010 the overlays and the waterproofing membranes were partially removed and replaced on a roadway repair project (CN 31903)
  - This work was not done to the current standard of ACC&M placement and it is considered likely that there were some areas where the waterproofing membrane would have been less effective
    - Some deck concrete was repaired with asphalt patches that were then covered with ACC&M
- In 2020 complete removal and replacement of the ACC&M in conjunction with a roadway paving project (CN 32309)
  - During the time that the overlay and membrane were removed, the decks on both bridges were tested to evaluate deck condition
  - The 1930s original concrete, and the concrete placed in the 1974 widening were evaluated separately
  - Asphalt patches placed in 2010 were removed and replaced with concrete patches
    - Some additional areas of unsound deck concrete were also repaired

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[1] Plan excerpts are shown in Appendix B

Test Methods & Results

This section describes the three testing methods used by VCS to evaluate the condition of concrete in the bridge decks and the results that were found by each method. All tests were conducted on the west side of each bridge (in the southbound traffic lanes).

Ground Penetrating Radar Survey

Ground penetrating radar (GPR) is a quick and effective way to identify the location and depth of metal objects within reinforced concrete. Steel reinforcement can be easily identified in a GPR scan due to the significant difference in the electromagnetic properties of steel and concrete.

GPR scans were collected along the length of each bridge to determine the cover-depth of reinforcement and the findings are presented in Table 1. For Bridge S075 17062, shallow reinforcing bars, with an average cover-depth of 2.1 in, were observed directly over the piers. Due to the shape of reinforcing steel that was used, as distance from the piers increases, the reinforcement goes deeper into the concrete section[2]. Therefore, the cover-depth for reinforcement over the piers was separated from the reinforcement in the spans in Table 1. A significant area of reinforcement over the piers had cover-depths below 2.0 in. However, it should be noted that reinforcement over the piers comprised only a small fraction of the total deck reinforcement of Bridge S075 17062. In the spans, the reinforcement was very deep. The average reinforcement cover-depth in regions away from the pier caps was 7.1 in, with no cover-depths below 2.0 in. Therefore, it is expected that the main reinforcement in the spans is well-protected from corrosion by the large cover-depth.

Bridge S075 17596 has two expansion joints, and there was no significant difference observed in the reinforcement cover-depth at the joints. The average cover-depth for this bridge was calculated to be 3.1 in. No cover-depths below 2.0 in were observed.

Table 1: Reinforcement Cover-Depth Statistics

Bridge	Location	Average (in)	Standard Deviation (in)	Minimum (in)	Maximum (in)	Less than 2.0 Inches (%)
S075 17062	Piers	2.1	0.4	0.9	2.8	33
	Spans	7.1	0.4	6.3	7.5	0
S075 17596		3.1	0.2	2.4	4.1	0

Electrical Continuity

Electrical continuity of the reinforcing is necessary for possible future corrosion mitigation by cathodic protection and to conduct efficient corrosion potential measurements. In most cast-in-place reinforced concrete structures conventional reinforcement is electrically continuous due to the crossing of bars and tie wires. If the reinforcement if found to be electrically isolated then continuity bonds will be required for the implementation of cathodic protection (CP). Electrical continuity is verified by contacting various steel elements with the lead wires from a high impedance multi-meter using the DC millivolts and/or resistance

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[2] Plan excerpt for S075 17062 showing reinforcement can be seen in Appendix B

settings. As per ACI 222R-19 Standard in Section 5.3.1.6, if the potential difference between the reinforcing elements is less than one (1) mV, then the reinforcing steel is deemed electrically continuous.

The reinforcement in Bridge S075 17062 was determined to be electrically continuous in the original section within each span and between spans. The reinforcement in the widened section was determined to be electrically continuous within each span, but was determined to be electrically isolated between spans. The reinforcement in the widened section was electrically continuous with the reinforcement in the original section within each span, but was electrically isolated between spans.

For Bridge S075 17596, the reinforcement in the original and the widened sections was found to be electrically continuous within each span. However, reinforcement was discontinuous between spans.

If a form of CP were to be applied to these decks, a more robust evaluation of electrical continuity would be required during the construction phase of the CP. However, it is expected that limited continuity corrections would be required and would mostly focus on correcting electrical continuity between spans.

Corrosion Potential Survey

In order to identify locations with a high probability of active corrosion, corrosion potential measurements were collected per ASTM C876 *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*[3]. A copper/copper sulfate (CSE) reference electrode was used to collect corrosion potential measurements. The CSE reference electrode was placed on the concrete surface with a saturated sponge used to make an electrical couple with the concrete. The reference electrode was then connected to the negative terminal of a volt-meter. The positive terminal of the volt-meter was connected to the embedded reinforcement of the structure, and the potentials at various points along the bridge deck were recorded. The magnitude and spatial variation of the measured potentials provides the probability for active corrosion at the test location.

A generally accepted interpretation of normalized CSE measurements is provided in the appendix of ASTM C876 (Table 2 and Figure 1). It is important to understand that the interpretation values provided in ASTM C876 are a general guideline based on values normalized to 72 degrees Fahrenheit, and are not absolute values. The threshold values can shift based on the concentration of moisture and oxygen in the concrete, as well as other environmental factors like temperature.

Table 2: ASTM C876 Interpretation of Data

Corrosion Potential	Probability of Active Corrosion
< -350 mV	90%
- 350 mV to -200 mV	Uncertain
> −200 mV	10%

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[3] CPT is also known as a Half-Cell Potential test. For a more detailed description see https://fhwaapps.fhwa.dot.gov/ndep/DisplayTechnology.aspx?tech_id=12

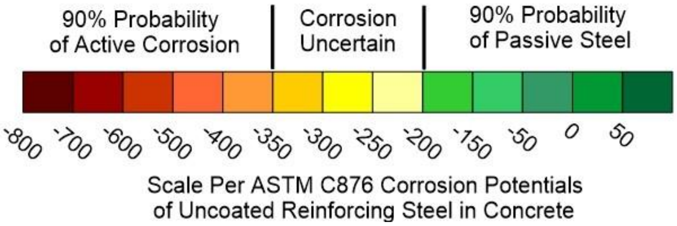

Figure 1: Corrosion Potential Survey Scale for Color Maps

Corrosion potential survey maps of the west lanes of Bridges S075 17062 and S075 17596 are presented in Figure 2 and Figure 3, respectively. The predominant green color of the widened sections in both bridges indicates that most the reinforcement in the widened sections is at a low risk for corrosion. The original section of Bridge S075 17596 is also generally at a low probability of corrosion (Figure 3). However, for Bridge S075 17062 the probability of corrosion for the original deck section was uncertain as indicated by the predominantly yellow color of this section in Figure 2.

Statistical analysis of the data presented in Table 3 indicated that approximately 90% of the widened area in each bridge was at a 90% probability of passive steel, and the corrosion activity was uncertain in less than 10% of the area. No measurements at 90% probability of active corrosion were observed in the widened section of Bridge S075 17062, and the area indicating 90% probability of active corrosion in the widened section of Bridge S075 17596 was very small, approximately 1%.

As for the original sections, majority of the area in Bridge S075 17062 had uncertain probability of corrosion, with only 24% indicating 90% probability of passive steel. This is surprising, considering a high cover-depth for most of the reinforcement in this bridge (Table 1). The original section of Bridge S075 17596, on the other hand, had 75% of the area indicating 90% probability of passive steel. Considering the similar age of these bridges at the time of the waterproofing membrane and asphalt overlay application, it is not clear why the original section of Bridge S075 17596 was in better condition than the other bridge.

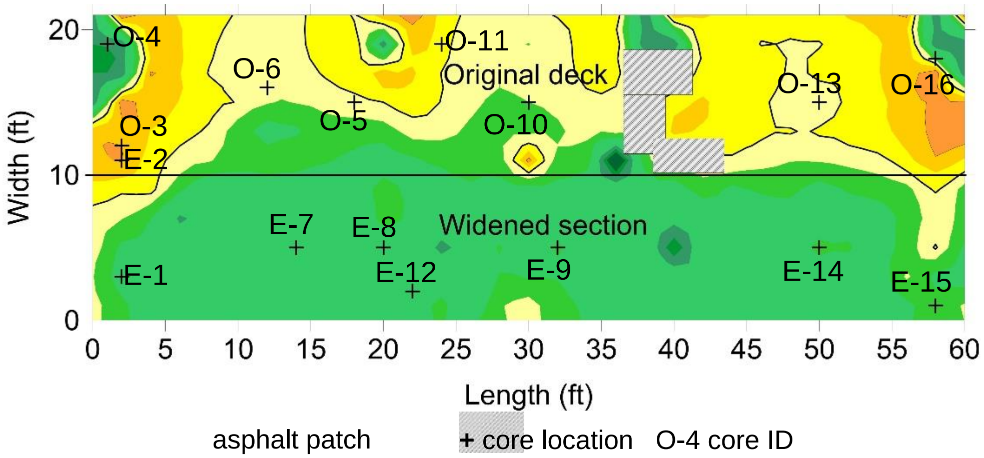

Figure 2: Corrosion Potential Survey Map of the West Lane of Bridge S075 17062 Deck

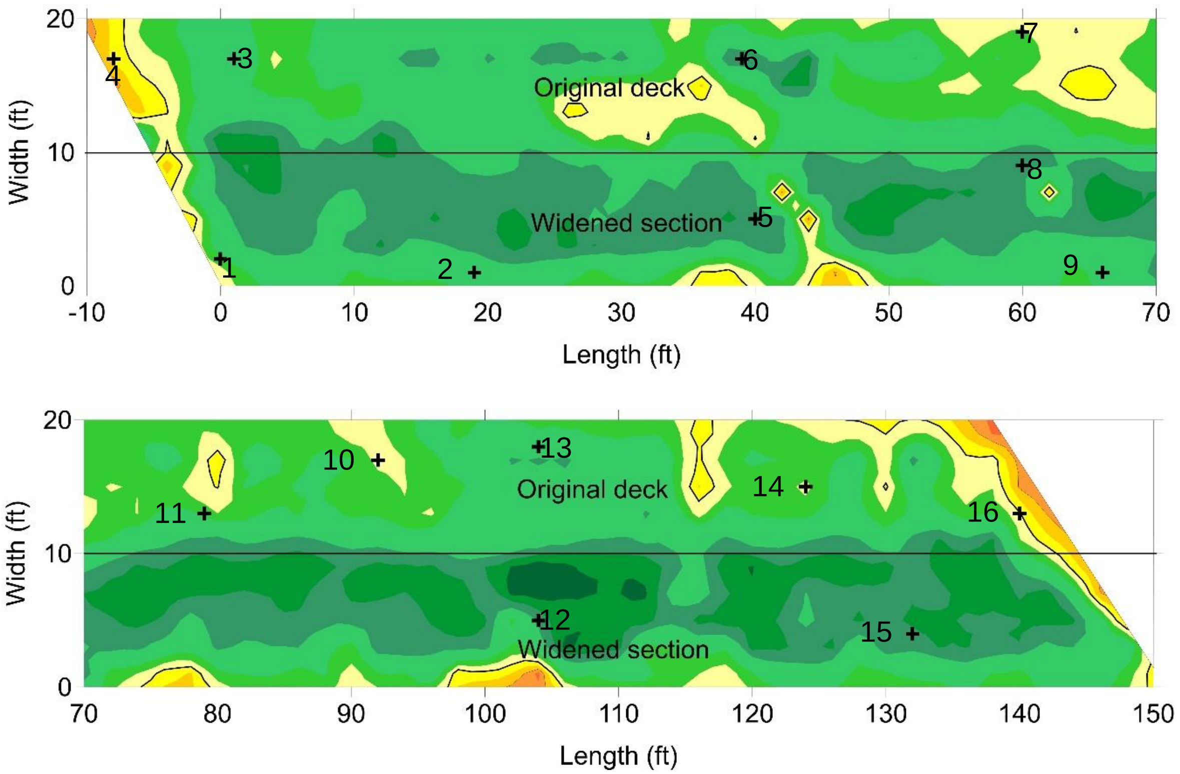

Figure 3: Corrosion Potential Survey Map of the West Lane of Bridge S075 17596 Deck

Table 3: Corrosion Potential Survey Statistics

Bridge	Section	Area Indicating 90% Probability of Active Corrosion (%)	Area Indicating Uncertain Corrosion Activity (%)	Area Indicating 90% Probability of Passive Steel (%)
S075 17062	Original	2.4	73.3	24.3
	Widened	0.0	6.1	93.9
	Overall	1.3	43.3	60.3
S075 17596	Original	6.3	18.3	75.4
	Widened	1.3	9.9	88.9
	Overall	3.8	14.1	82.1

Corrosion Rate Measurement

An estimation of corrosion rate in reinforced concrete can be obtained by galvanostatic pulse method[4]. In this technique, a low current is applied to reinforcement for 5-10 seconds, and the resulting change in potential over time is measured by a reference electrode. The measured concrete resistance, polarization and applied current are then used to estimate the corrosion current.

Corrosion rate measurements were collected at the location of each collected core. Table 4 provides the generally accepted interpretation of the corrosion rate values and Table 5 provides the individual corrosion rate values measured for each bridge. All but one measurement indicated a negligible rate of corrosion. One measurement collected at the

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[4] For more detailed description of Corrosion Rate Testing see https://fhwaapps.fhwa.dot.gov/ndep/DisplayTechnology.aspx?tech_id=11

location of core E-1, Bridge S075 17062, indicated a slow rate of corrosion. Statistical analysis of the corrosion rate measurements in presented in Table 6 and indicates that the corrosion rate is negligible in both bridge decks. No significant difference in corrosion rate was observed between the original and the widened sections of both bridges.

Table 4: Corrosion Rate Threshold Values

Measurement	Corrosion Rate
< 0.5 μA/cm^2	negligible
0.5 - 5 μA/cm^2	slow
5 - 15 μA/cm^2	moderate
> 15 μA/cm^2	high

Table 5: Measured Corrosion Current Values

Bridge S075 17062		Bridge S075 17596	
Core	**Corrosion Current (μA/cm^2)**	**Core**	**Corrosion Current (μA/cm^2)**
E-1	0.6	1	0.0
E-2	0.0	2	0.1
O-3	0.1	3	0.0
O-4	0.1	4	0.0
O-5	0.2	5	0.0
O-6	0.2	6	0.0
E-7	no reading	7	0.0
E-8	0.0	8	0.0
E-9	0.0	9	0.0
O-10	0.0	10	0.0
O-11	0.2	11	0.0
E-12	0.0	12	0.0
O-13	0.1	13	0.0
E-14	0.0	14	0.0
E-15	0.5	15	0.0
O-16	0.0	16	0.0

Table 6: Statistical Analysis of the Corrosion Rate Measurements

Bridge	Average Corrosion Current (μA/cm^2)	Standard Deviation (μA/cm^2)	Minimum (μA/cm^2)	Maximum (μA/cm^2)
S075 17062	0.1	0.2	0.0	0.6
S075 17596	0.0	0.0	0.0	0.1

Concrete Material Sampling

In addition to non-destructive methods, concrete material sampling was conducted to gain an understanding of the concrete composition and the amount of chloride ion contamination within the concrete matrix from de-icing salt application. Samples were collected in the form of 4-inch diameter cores drilled from the top of the deck. The location of all the cores is indicated in Figure 2 and Figure 3. Appendix A provides a photo log of the collected cores.

Carbonation Depth

The depth of carbonation into the concrete can indicate the risk for corrosion activity. Carbonation lowers the concrete's pH as carbon dioxide diffuses into moist concrete. If the pH of the concrete surrounding the reinforcing steel is lowered below pH 11, depassivation of the reinforcing begins and general corrosion initiates. Carbonation can cause corrosion in concrete that has not been contaminated with chlorides and can also propagate through crack surfaces. In chloride-contaminated concrete, carbonation can work in tandem with chlorides to initiate corrosion much more quickly.

To identify the depth of the carbonation front in concrete, a pH indicator solution is sprayed onto freshly extracted and cleaned concrete cores. The indicator solution changes to a pink/purple color at pH greater than 9.5. If the solution is clear, that is an indication of carbonated concrete. If the solution turns purple or pink on the concrete then that is an indication of uncarbonated or alkaline concrete.

Cores were tested for carbonation in the field immediately after coring. The carbonation depth for all the cores was negligible, less than 0.25 in. There is little to no risk of corrosion due to carbonation of the concrete.

Concrete Chloride Sampling

Reinforcing steel in concrete is protected from corrosion by the high alkalinity of the concrete pore solution, typically greater than a pH of 12. The high pH of the pore solution causes formation of a passivating film on the surface of rebar, effectively sealing it and preventing corrosion. Corrosion of reinforced concrete exposed to chloride-containing environment, such as deicing salts or marine exposure, is typically initiated by chloride ions, which have the ability to break down the passivating film. Chloride ions diffuse from the concrete surface, and once their concentration at reinforcement depth reaches a threshold value, corrosion is initiated. The quantity of chlorides required to depassivate the steel is known as the threshold concentration. Chloride threshold is a critical value in determining the initiation time for the service life model. In the literature, threshold concentrations for chloride in concrete can vary significantly and depend on a number of factors. ACI 222R-19 indicates that for acid-soluble chloride testing the generally accepted chloride threshold in the United States is between 1.0 and 1.5 lbs of chloride per cubic yard of concrete (263 to 395 ppm assuming a concrete density of 3,800 lbs/yd^3).

For the service life modeling, VCS typically implements a chloride threshold of 350 ppm with a standard variation of 50 ppm. It is important to consider the variation of the threshold as chloride threshold is not a single value. Due to many influencing factors, corrosion of steel in concrete can initiate at a range of chloride concentrations. As a result, it is important to take into consideration this variation. There are many environmental and concrete material conditions that can cause the corrosion to initiate at a lower or higher threshold. For example, if the moisture content is high in an area then corrosion may initiate at a lower chloride concentration. If the concrete is drier in an area it may take more chloride to cause corrosion.

In addition to chlorides entering the concrete from the environment, concrete can also contain background chlorides, which were either admixed into fresh concrete or are naturally present in cement products or aggregates. Admixed chlorides could be added to the concrete mix through the use of chloride-containing chemical admixtures or the use of seawater instead of potable water. Admixed chlorides and chloride ions that diffuse into the concrete from the environment are referred to as “free” chlorides and are responsible for chloride-induced corrosion in reinforced concrete. Chlorides present in the aggregate are chemically bound and are not able to initiate corrosion.

Concrete samples were collected to evaluate the level of chloride penetration and risk for corrosion activity of the steel reinforcing per ASTM C1152 *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*, which measures the concentration of both free and bound chlorides. The concrete samples were collected in the form of cores which were sliced in 0.5-inch depth increments and then pulverized into concrete powder samples.

Figure 4 through Figure 7 present the chloride concentration profiles for all the collected samples. The average reinforcement cover-depth and the average plus or minus one standard deviation are indicated in the plot by vertical green lines, and the chloride threshold value of 350 ppm is plotted as a horizontal red line. For Bridge S075 17062, the cover-depth over the piers is shown on the plots since it was not possible to core to the reinforcement depth in the spans.

When concrete is exposed to an environment containing chloride ions, either from deicing salt application or marine exposure, chlorides will diffuse into the concrete from the surface. This results in the highest chloride concentration at the surface and a decreasing chloride concentration with increasing depth as observed in the original sections of the two decks (Figure 4 and Figure 6).

For Bridge S075 17062, almost all the samples from the original deck (7 out 8 or 88%) had chloride concentrations above the corrosion initiation threshold in the depth range 1.7 to 2.5 in, which is the average cover-depth plus/minus one standard deviation for the reinforcement over the piers. This indicates that the reinforcement over the piers is at a high risk for chloride-induced corrosion. However, since the majority of the reinforcement in this bridge had significantly higher cover-depths (7.1 in) and the chloride concentrations at that depth are expected to be well below the threshold, a majority of the reinforcement away from the piers is not expected to be at risk for chloride-induced corrosion. Additionally, though chloride concentrations in the original concrete were above thresholds that are typically considered likely to initiate corrosion, the corrosion potential shown in Figure 2 and Table 3 indicate, that only 2.4% of this bridge deck has a probability of active corrosion. The total area of shallower concrete cover is 15.8% of the deck. As can be seen in Figure 2, the higher areas of corrosion potential occur near the supports (abutments and piers) where the reinforcing steel has shallower concrete cover.

Although surface chloride concentrations in the original section of Bridge S075 17596 were high in some of the cores, at the rebar depth they were below the threshold for corrosion initiation in all the collected samples. This explains the low probability of corrosion indicated by the corrosion potential measurements in this deck compared to the original section of Bridge S075 17062 (Figure 2 and Figure 3).

The chloride concentrations in the original section of Bridge S075 17062 were generally higher than in Bridge S075 17596. This was a little surprising, since it is expected that both bridges would have been exposed to the same amounts of deicing salts, and Bridge S075 17062 is about 5 years younger. The apparent higher rate of chloride penetration in the original section of Bridge S075 17062 could be due to differences in concrete mix design used for this bridge compared to Bridge S075 17596.

In the widened sections, no significant change was observed in chloride concentration with increasing depth, which indicates that the chlorides present in the widened sections are background chlorides naturally present in cement and aggregates. It is clear that the waterproofing membrane was very effective in protecting the concrete from chloride ingress. Reinforcement in the widened sections is not at risk for chloride-induced corrosion.

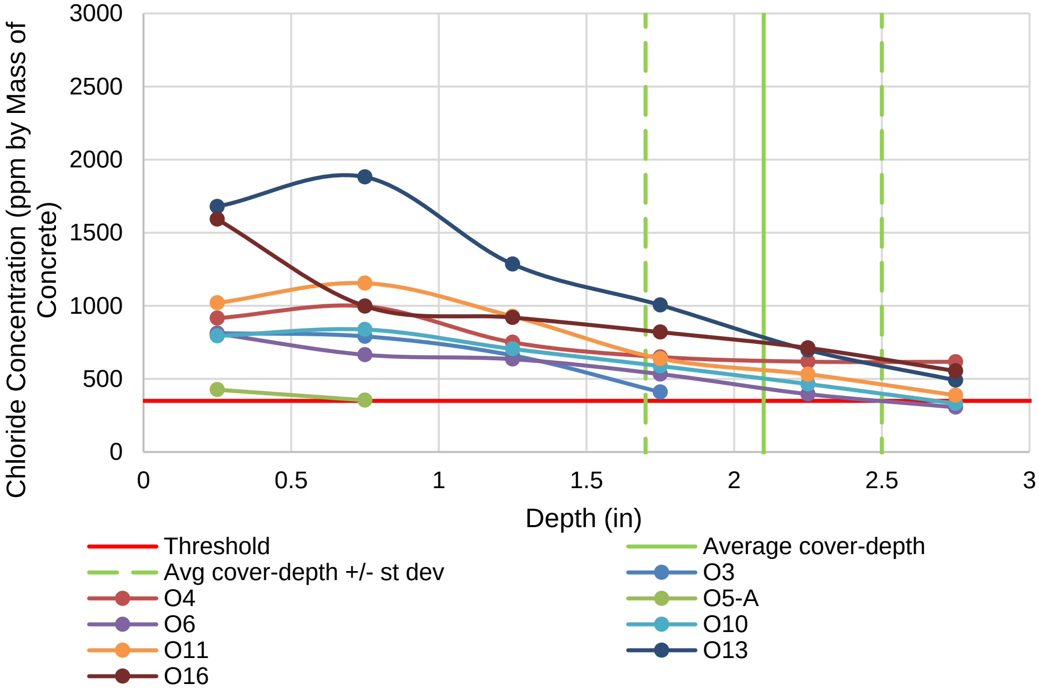

Figure 4: Chloride Concentration Profiles for the Original Section of the Bridge S075 17062 Deck

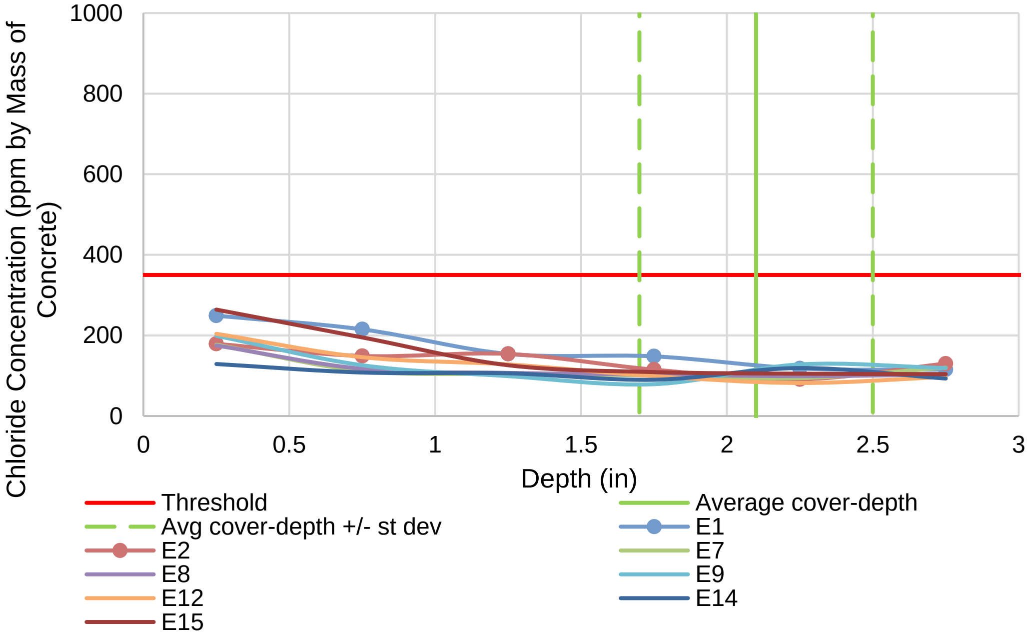

Figure 5: Chloride Concentration Profiles for the Widened Section of the Bridge S075 17062 Deck

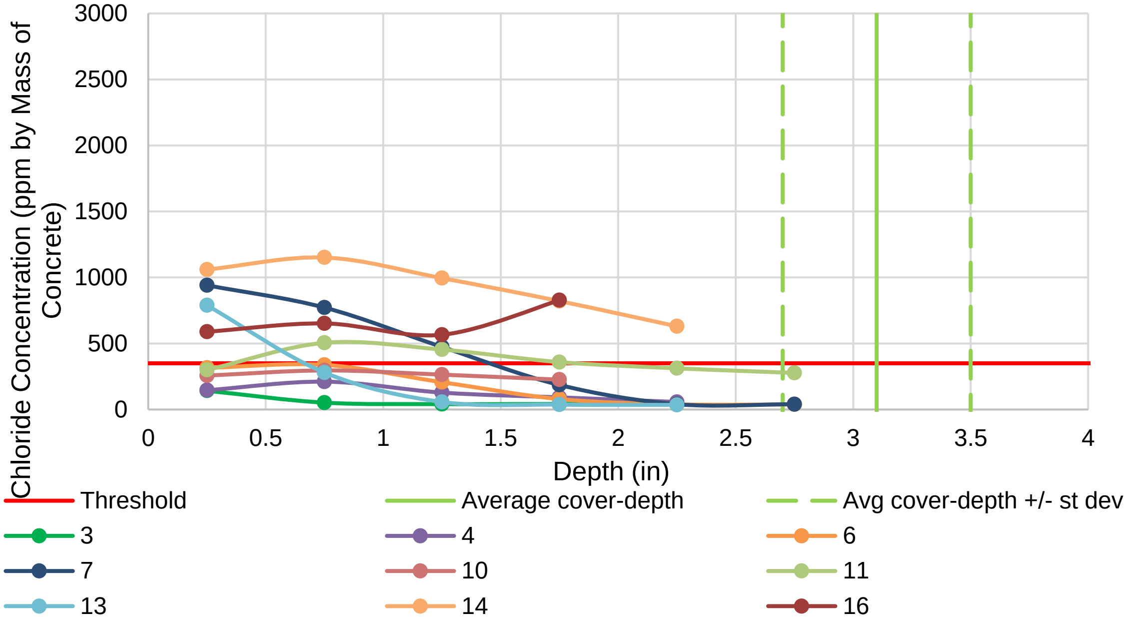

Figure 6: Chloride Concentration Profiles for the Original Section of the Bridge S075 17596 Deck

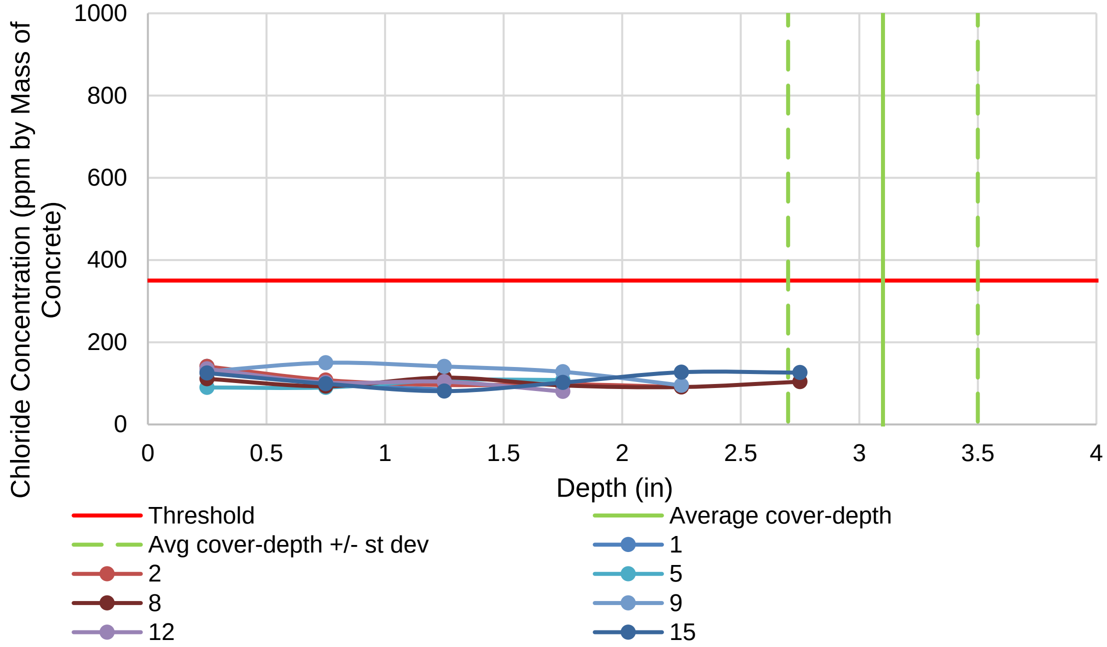

Figure 7: Chloride Concentration Profiles for the Widened Section of the Bridge S075 17596 Deck

Visual Inspection of Reinforcing Steel

It was not possible to core over the reinforcement in Bridge S075 17062 due to the high average cover-depth. However, in Bridge S075 17596 a number of cores were extracted over the reinforcing steel to visually inspect the condition of the reinforcement. The bars were observed to be in good condition with only minor surface rust. Typical images presented in Figure 9.

Figure 8: Typical Reinforcement Condition in Bridge S075 17596

Conclusions

Based on the data collected by VCS, the following conclusions can be made regarding the waterproofing membrane and asphalt overlay effectiveness in protecting Bridges S075 17062 and S075 17596 from chloride ingress and chloride-induced corrosion.

1. For the widened sections where the waterproofing membrane and asphalt overlay were applied immediately after construction, minimal chloride ingress into the concrete was observed. The chloride concentration profiles were mostly flat, indicating that the chlorides present were bound chlorides that were naturally present in the concrete at the time construction. There is no risk of chloride-induced corrosion at the present time or in the future, as long as the concrete surface continues to be protected from chloride ingress.
2. The original sections of both bridges were exposed to chlorides for approximately 20 years prior to application of the overlay. While the waterproofing membrane and the asphalt overlay protected the original decks from deicing chemical from 1974 onward, the chlorides that penetrated into the surface of the concrete between the mid-1950s and 1974 remained in the concrete and continued to diffuse into the deck cross-section.
    a. Nevertheless, chloride concentration in the original section of Bridge S075 17596 at rebar depth was below the typical corrosion initiation threshold. Service life modeling indicated that at the average cover-depth of 3.1 in, there is little to no risk of chloride-induced corrosion in the foreseeable future as long as the deck continues to be protected from deicing salts.
        i. Visual observation of the rebar exposed during coring indicated that the rebar is in good condition with only minor surface corrosion.
    b. The shallow rebar that was observed over the piers in Bridges S075 17062 is at risk of chloride-induced corrosion at the present time. However, there is no risk of chloride-induced corrosion to the rebar away from the piers due to their high cover-depth.
    c. Chloride concentrations for bridge S075 17062 would indicate that in the areas of shallower concrete cover (near the abutments and piers), the reinforcing steel is at risk of chloride-induced corrosion. However, the corrosion potential survey found that only 2.4% of the deck has a high probability of active corrosion and the total area of shallower concrete over the supports is 15.8%.
3. The overlay was very effective is protecting both the original and the widened sections from carbonation.

Thank you for the opportunity to work with you on this project and if you have any questions please do not hesitate to contact me directly.

Sincerely,

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NDOT Comment

Concrete Permeability

Concrete Permeability was not studied as part of this analysis but is known to play an important role in the propagation rate of chlorides in concrete. In general, bridges of similar age are constructed under similar concrete mix specifications, but variations in the applied concrete can occur as a result of construction conditions such as temperature, curing conditions, curing time and other factors.

A Comparison of Chloride Concentrations in Bridges of Similar Age on the State Highway System

Historically, NDOT has tested many bridges for chloride concentrations on unprotected concrete bridge decks prior to placing concrete deck overlays. The Table below shows chloride concentrations at the level of reinforcing steel for a group of ten bridges that, at the time of testing, had exposed deck concrete (no overlay) and had been in service for a similar length of time to the widened concrete of the two bridges in this study.[5]

NDOT has also tested a few bridges for chloride concentrations after a period of service with a concrete overlay in place[6]. The Table below also includes results of these tests for comparison[7].

As can be seen in the Table below, the widened concrete that was protected with ACC&M had the lowest concentrations of chloride and even the original concrete, though much older, had lower chloride concentrations than bridges with no overlay and bridges with concrete overlays.

Description	Deck Surface	avg Cl top mat ppm	avg Cl top mat LB/CY	avg yr deck placed	avg age of deck
10 Bridges - bare deck	Concrete	935	3.554	1963	44
7 Bridges - concrete overlays*	Concrete/Conc Overlay	533	2.026	1972	40
Widened Concrete	ACC&M	104	0.394	1974	45
Original Concrete	Concrete/ACC&M	451	1.905	1935	83

* Average age at placement of overlay was 29.1 years

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[5] See Appendix C for Average Chloride analysis and a more detailed comparison.

[6] Testing bridges with concrete overlays for chlorides after a period of service is rare because typically the next action, when a concrete overlay reaches end of service life, is to replace the bridge deck or replace the entire bridge.

[7] See Appendix C for more detailed analysis

Appendix A
Core Chloride Concentrations

Deck chloride concentration for the original part of bridge S075 17062									Average	Average
depth	O3	O4	O5-A	O6	O10	O11	O13	O16	(ppm)	(lb/cy)
0.25	813	915	426	809	794	1020	1679	1592	1006	3.82
0.75	791	997	354	665	838	1155	1881	998	960	3.65
1.25	661	750		635	705	928	1285	920	841	3.19
1.75	410	650		533	588	639	1005	820	664	2.52
2.25		618		396	465	532	698	712	570	2.17
2.75		617		306	332	388	491	554	448	1.70

Deck chloride concentration for the widened part of bridge S075 17062									Average	Average
depth	E1	E2	E7	E8	E9	E12	E14	E15	(ppm)	(lb/cy)
0.25	249	179	177	175	199	204	129	264	197	0.75
0.75	215	149	112	117	126	146	108	195	146	0.55
1.25	154	154	104	107	99	128	106	126	122	0.46
1.75	148	115	101	103	79	99	90	109	106	0.40
2.25	118	91	94	99	128	82	119	105	105	0.40
2.75	115	130	120	103	119	97	93	104	110	0.42

Deck chloride concentration for the original part of bridge S075 17596										Average	Average
Depth	3	4	6	7	10	11	13	14	16	(ppm)	(lb/cy)
0.25	142	146	316	941	257	300	789	1060	589	504	1.92
0.75	52	211	338	772	296	505	280	1151	652	473	1.80
1.25	41	128	207	474	264	454	57	995	566	354	1.35
1.75	42	92	79	186	226	359	37	821	828	297	1.13
2.25	45	57	39	40		312	35	630		165	0.63
2.75			38	40		277				118	0.45

Deck chloride concentration for the widened part of bridge S075 17596								Average	Average
Depth	1	2	5	8	9	12	15	(ppm)	(lb/cy)
0.25	141	141	90	111	129	135	125	125	0.47
0.75	99	108	90	93	150	102	99	106	0.40
1.25	85	96	108	114	141	104	81	104	0.40
1.75		98	108	95	128	80	102	102	0.39
2.25		91		91	95		127	101	0.38
2.75				104			126	115	0.44

Core Photo Log

Core E-1

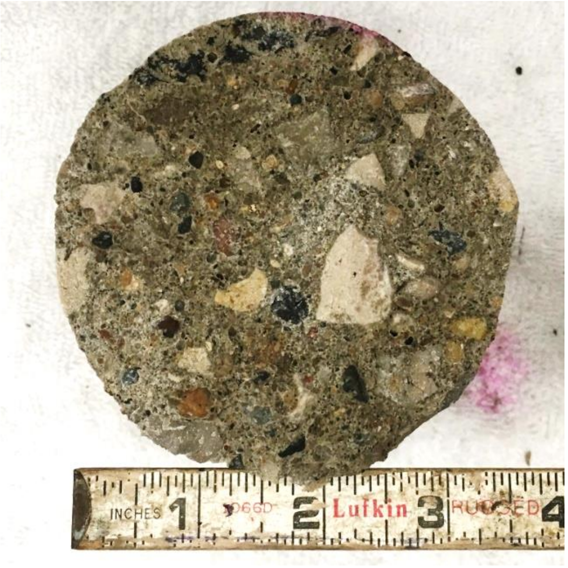
Side A

Side B

Side C

Core E-1

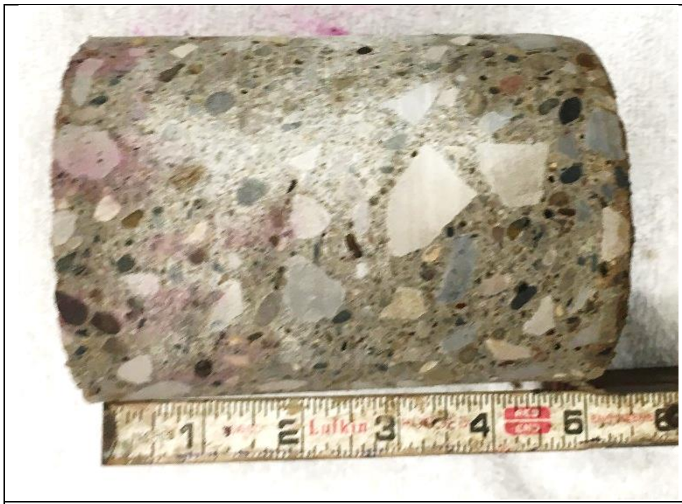
Top



Bottom

Core E-2

Side A



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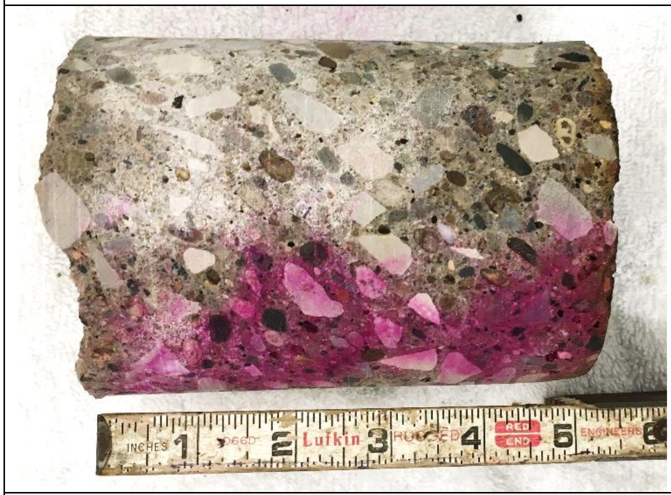


Side C

Core E-2

Top

Bottom

Core E-7

Side A

Side B

Side C

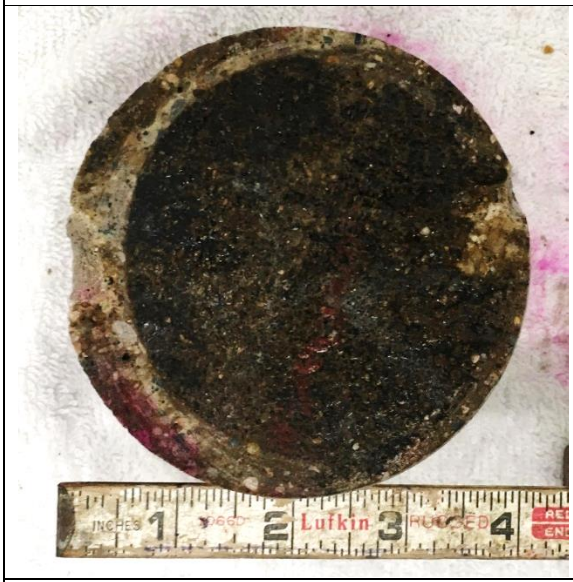
Core E-7	
	
Top	Bottom

Core E-8

Side A

Side B

Side C

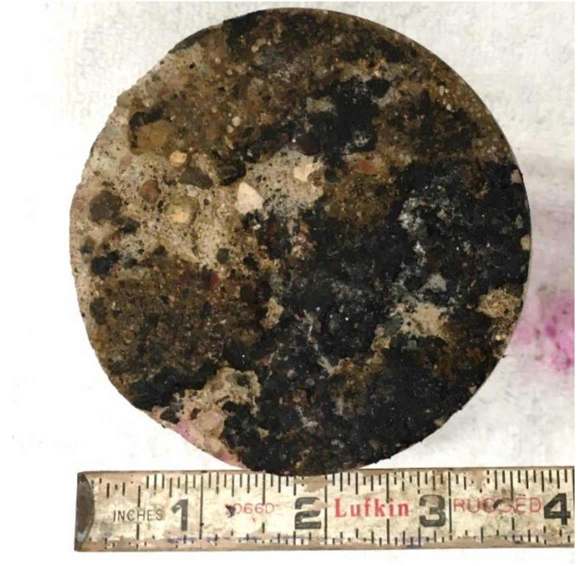
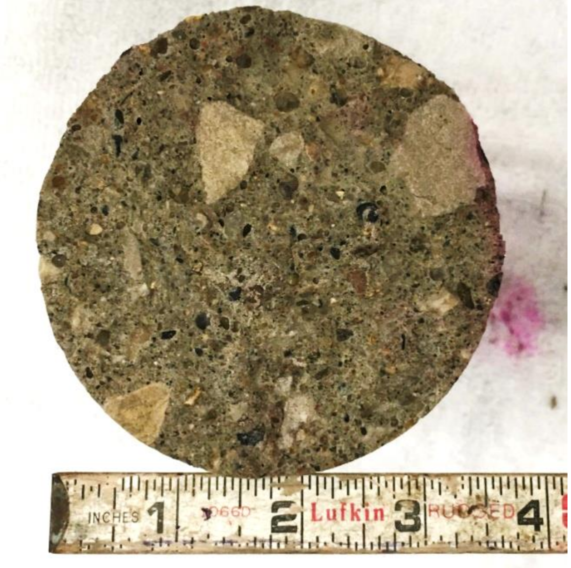
Core E-8	
	
Top	Bottom

Core E-9

Side A

Side B

Side C

Core E-9	
	
Top	Bottom

Core E-12

Side A

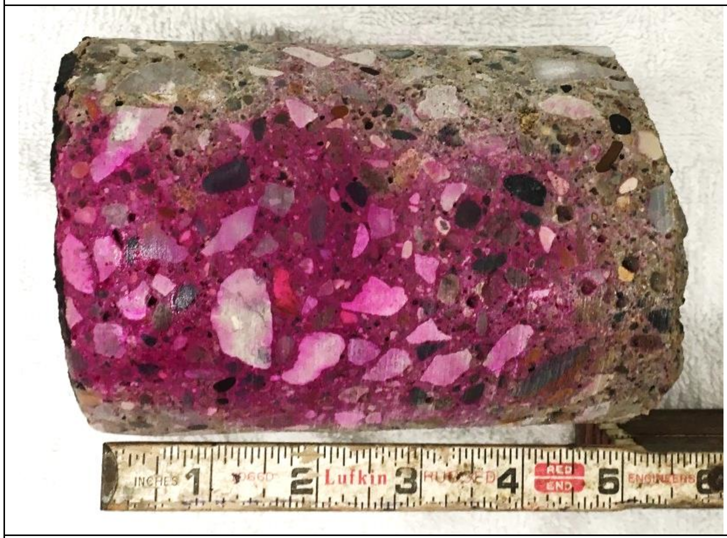

Side B

Side C

Core E-12	
missing	*missing*
Top	Bottom

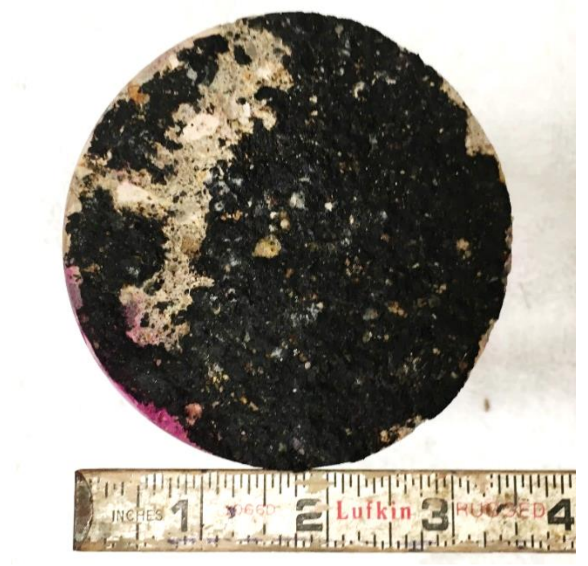
Core E-14

Side A



Side B

Side C

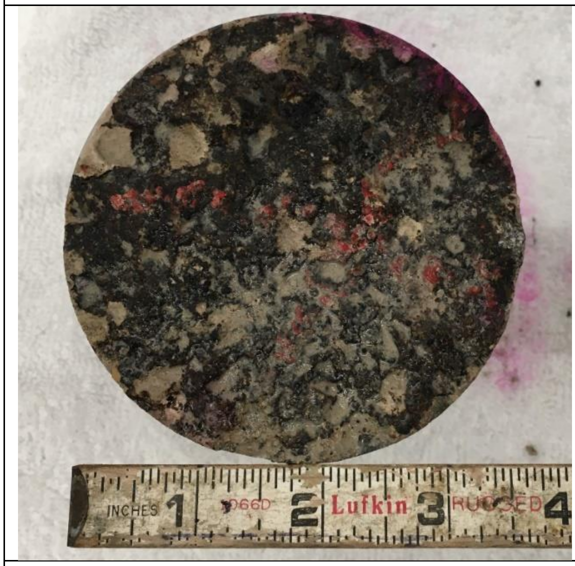
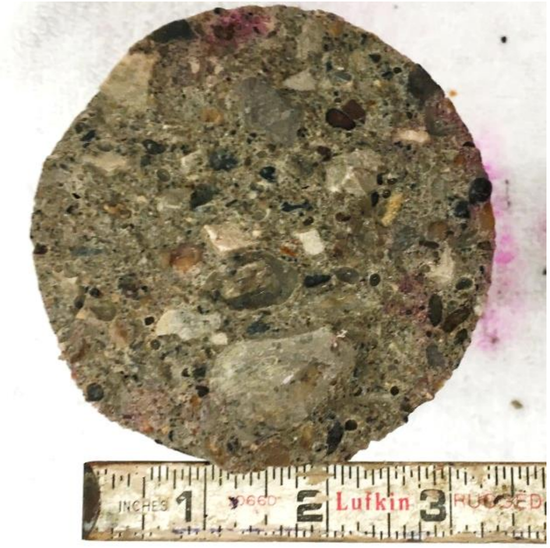
Core E-14	
	
Top	Bottom

Core E-15

Side A

Side B

Side C

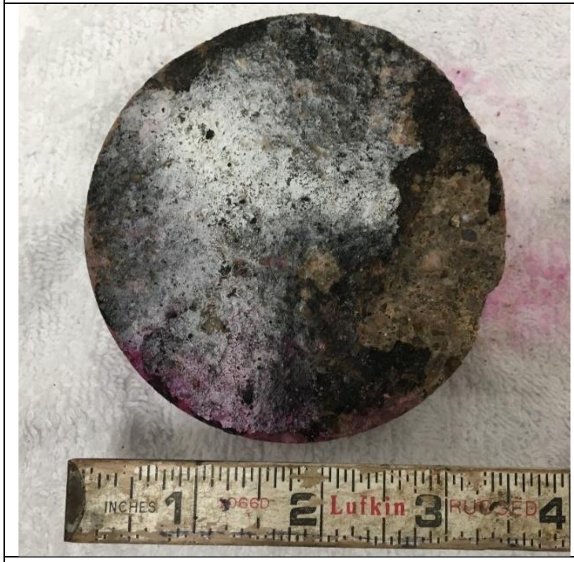
Core E-15	
	
Top	Bottom

Core O-3

Side A

Side B

Side C

Core O-3	
	
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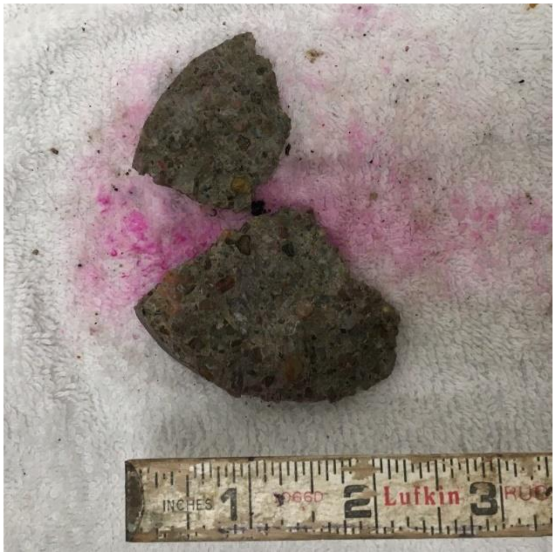
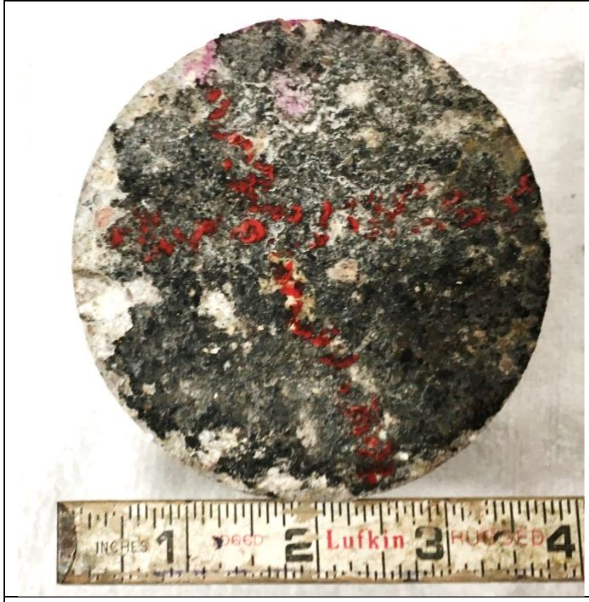
	
Bottom-1	Bottom-2

Core O-4

Side A

Side B

Side C

Core O-4	
	
Top	Bottom

Core O-5

Side A

Side B

Side C

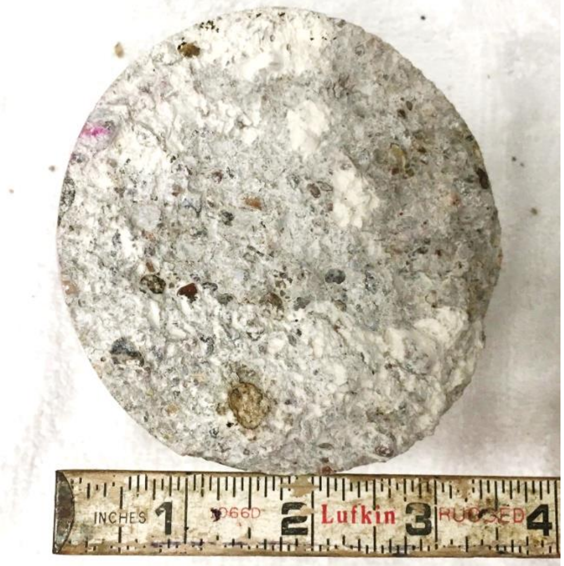
Core O-5	
	
Top-1	Top-2
	
Bottom-1	Bottom-2

Core O-6

Side A

Side B

Side C

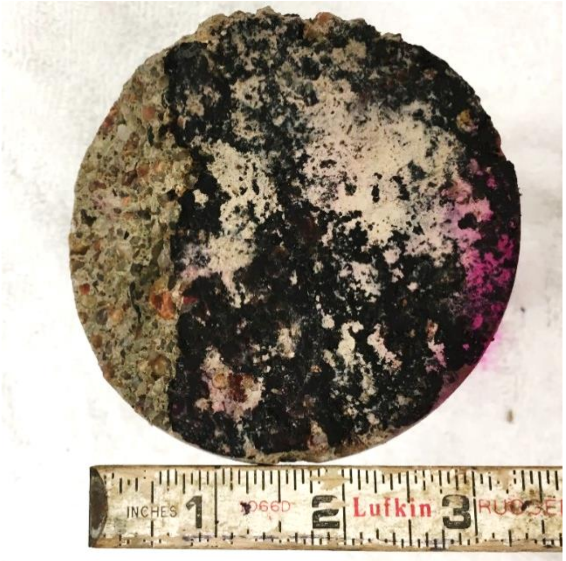
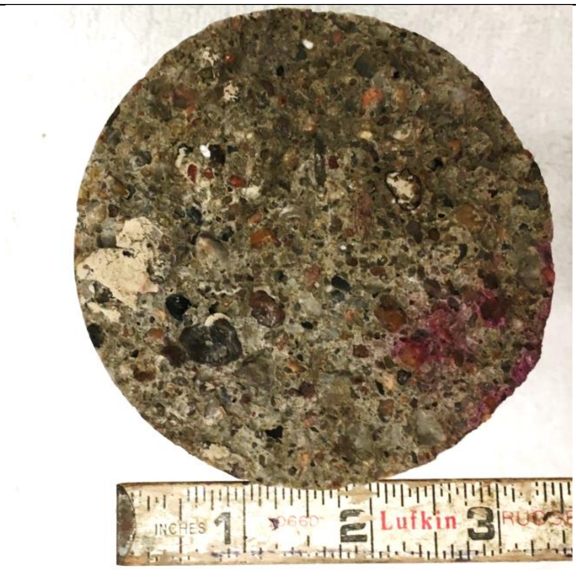
Core O-6	
	
Top	Bottom

Core O-10

Side A

Side B

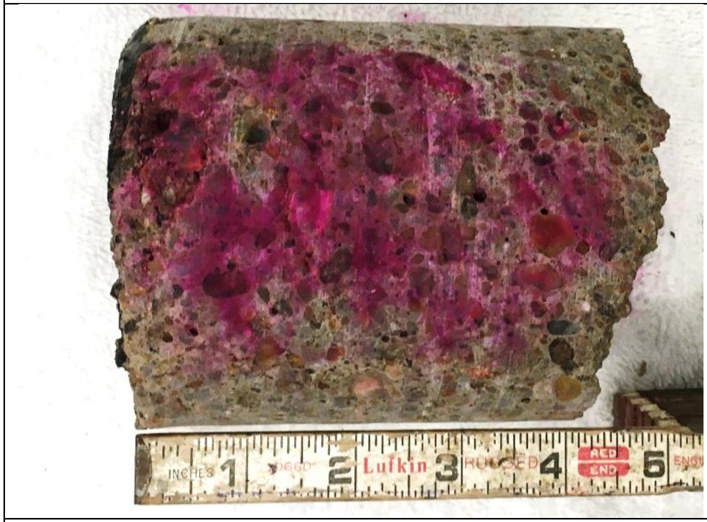
Side C

Core O-10	
	
Top	Bottom

Core O-11

Side A

Side B

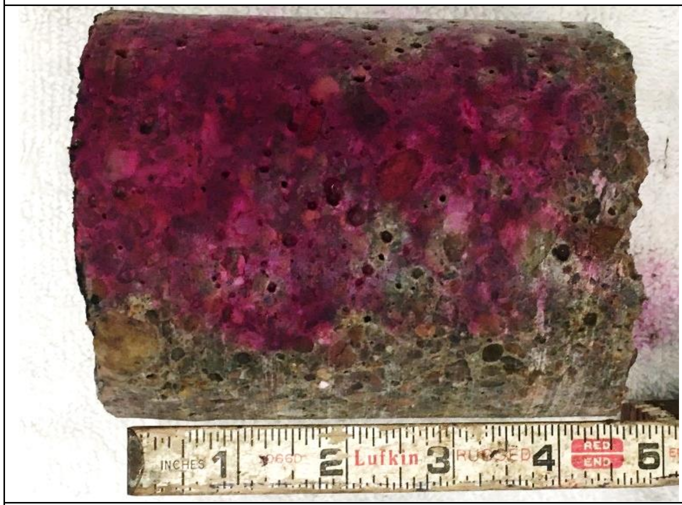


Side C

Core O-11	
missing	*missing*
Top	Bottom

Core O-13

Side A



Side B

Side C

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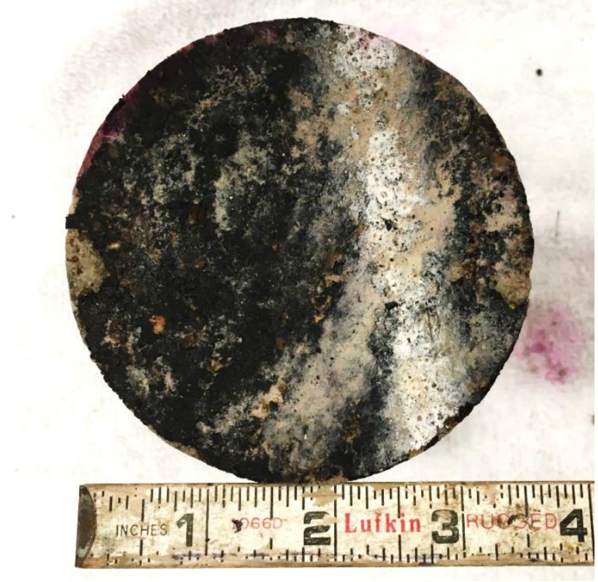
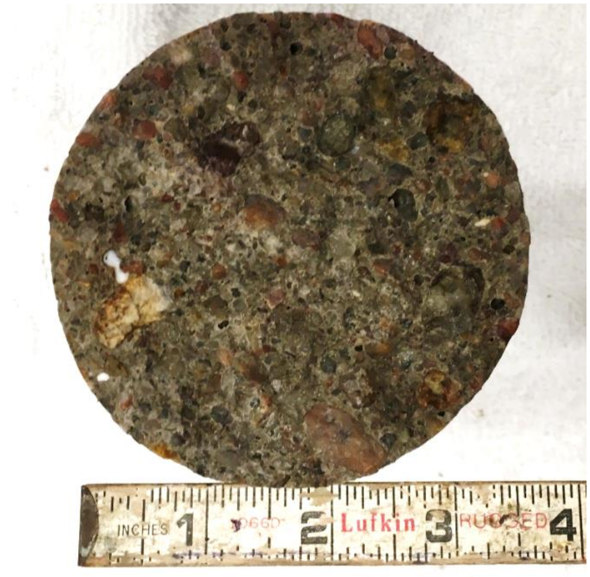
<tr><td> </td><td> </td></tr>
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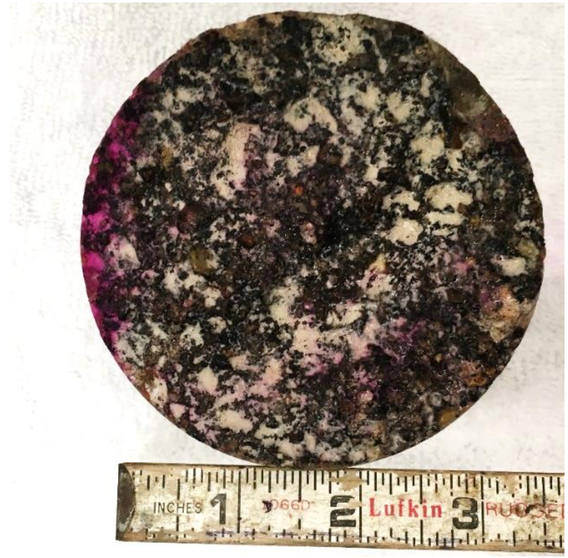
Core O-16

Side -A

Side-B

Side-C

Core O-16	
	
Top	Bottom

Core 1

Side A

Side B

Side C

Core 1	
	
Top	Bottom

Core 2

Side A

Side B

Side C

Core 2	
	
Top	Bottom

Core 3

Side A

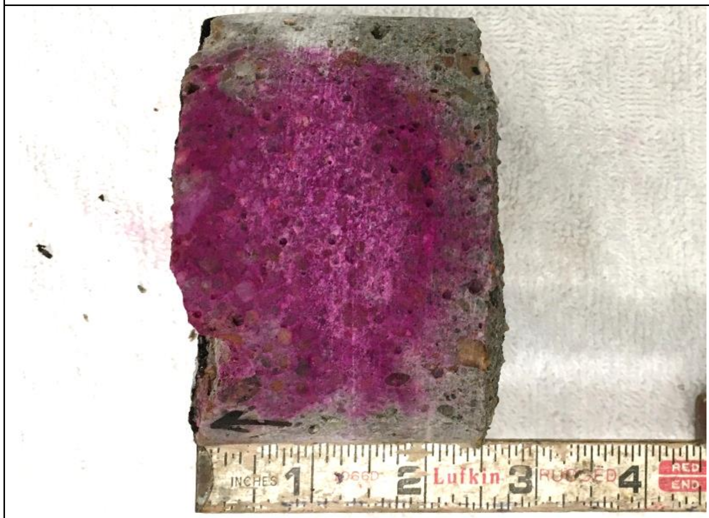
Side B

Side C

Core 3	
	
Top	Bottom

Core 4

Side A



Side B

Side C

Core 4	
	
Top	Bottom

Core 5

Side A

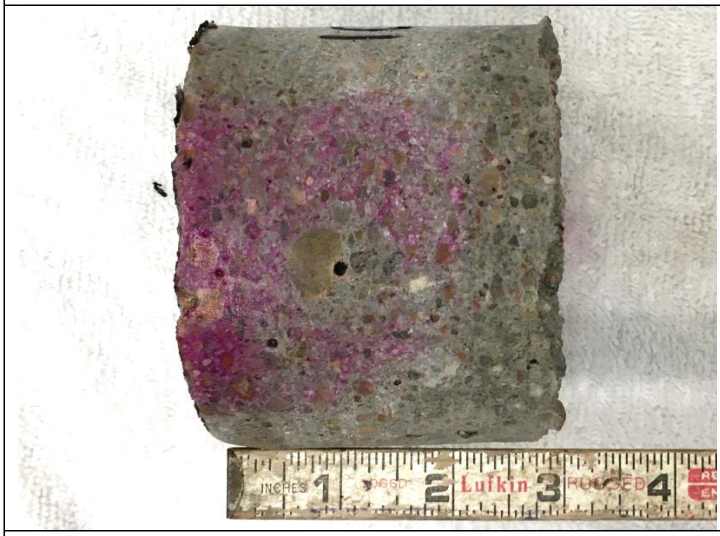
Side B

Side C

Core 5	
missing	*missing*
Top	Bottom

Core 6

Side A



Side B

Side C

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<tr><td>Top</td><td>Bottom</td></tr>
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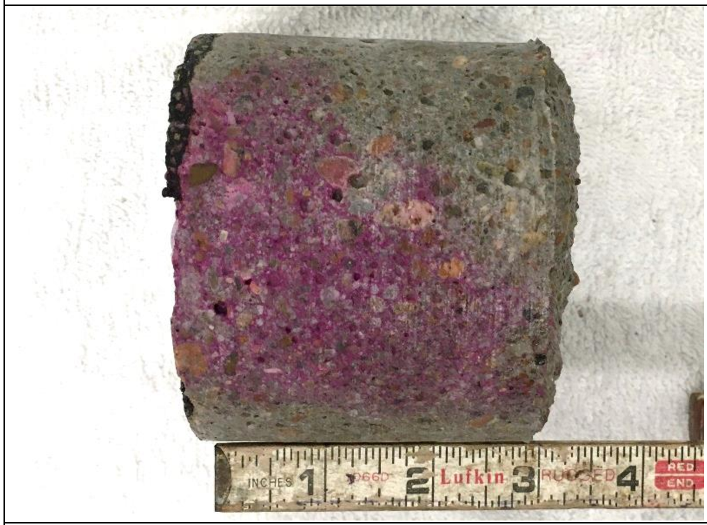
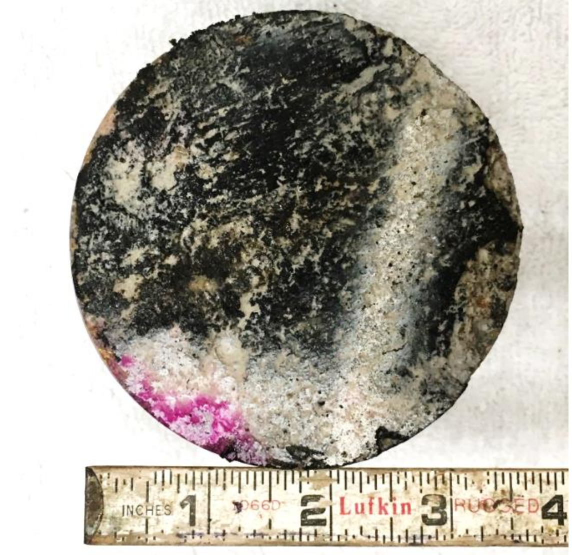
Core 7

Side A



Side B

Side C

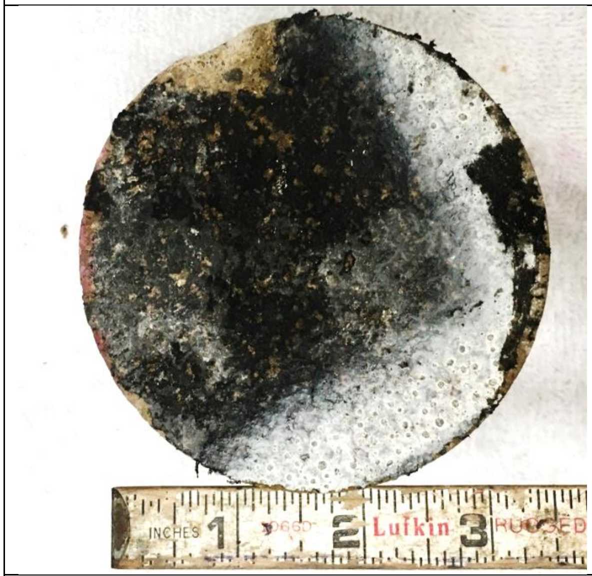
Core 7	
	
Top	Bottom

Core 8

Side A

Side B

Side C

Core 8	
	
Top	Bottom

Core 9

Side A

Side B

Side C

Core 9	
missing	*missing*
Top	Bottom

Core 10

Side A

Side B

Side C

Core 10	
	
Top	Bottom

Core 11

Side A

Side B

Side C

Core 11	
	
Top	Bottom

Core 12

Side A

Side B

Side C

Core 12

Top-1

Top-2

Bottom-1

Bottom-2

Core 13

Side A

Side B

Side C

Core 13	
	
Top	Bottom

Core 14

Side A

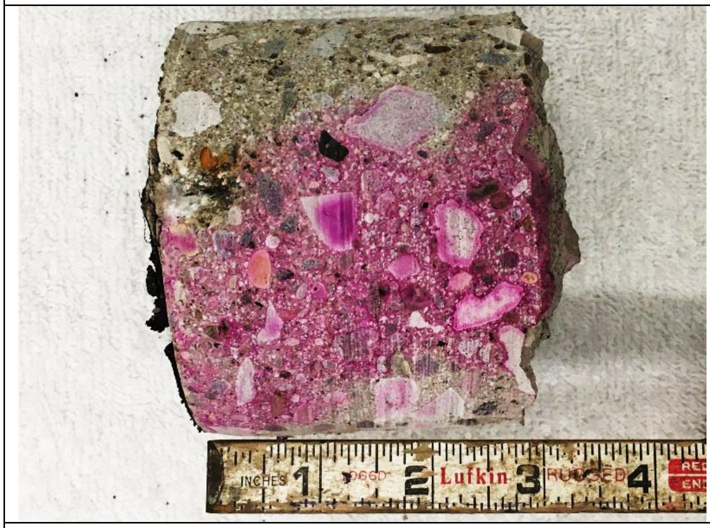
Side B

Side C

Core 14	
	
Top	Bottom

Core 15

Side A



Side B

Side C

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<tr><td>Top</td><td>Bottom</td></tr>
</table>

Core 16

Side -A

Side-B

Side-C

Core 16	
	
Top	Bottom

Appendix B
Plan Excerpt for S075 17062

Figure 10 shows the reinforcement placement from the original 1935 plan for S075 17062. There was no top mat of reinforcing steel. Note the 1 ¾" minimum clearance near the supports for the A and B reinforcement bars. Bending diagrams for these bars are shown in Figure 11.

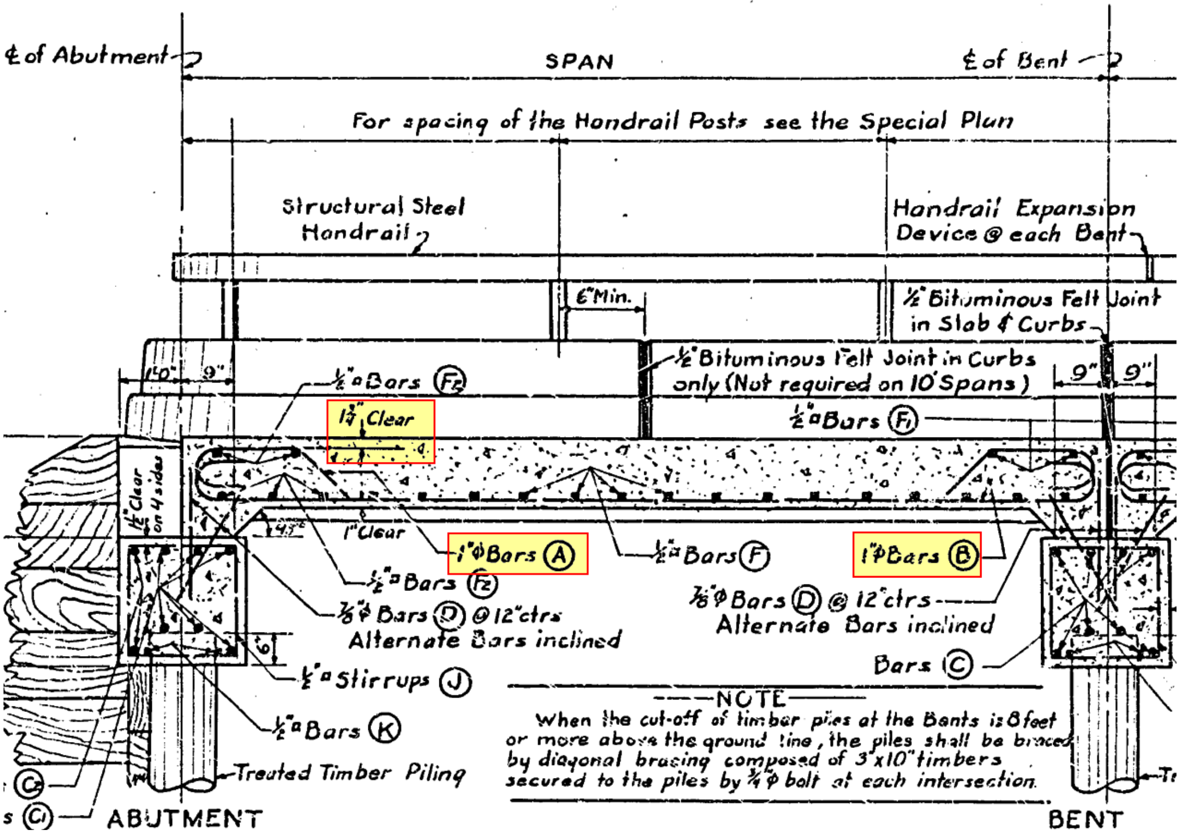

Figure 9 Excerpt from 1935 plan showing reinforcement

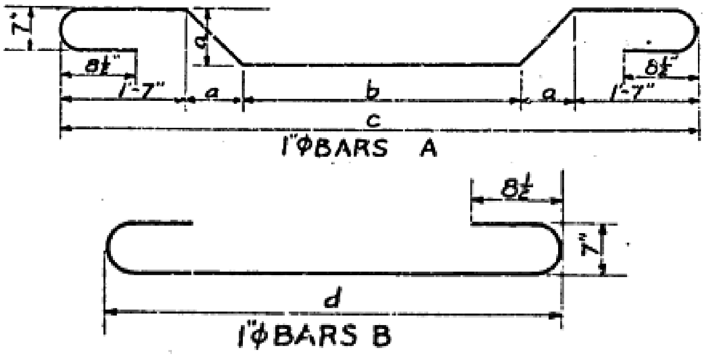

Figure 10 Showing bending diagrams for A and B bars from 1935 plan

The figure below shows the 1974 widening project. Note that minimum clearance of S801 and S805 bars near the supports was 1 ½" and no top mat of reinforcing steel is present in the midspan areas.

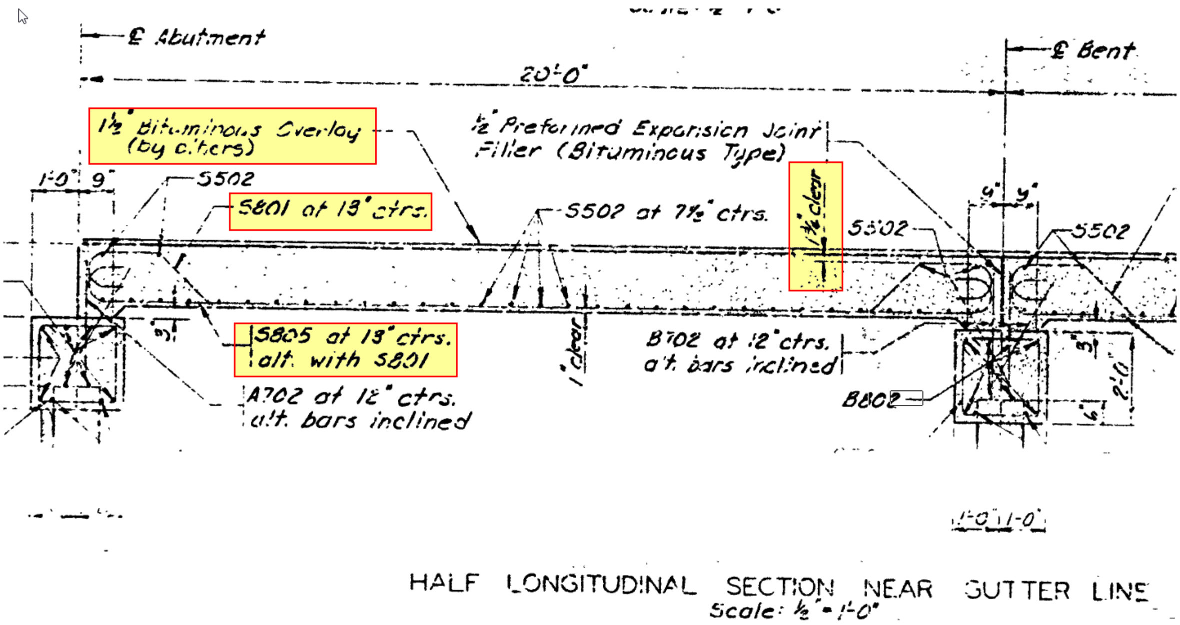

Figure 11 Showing reinforcement placement for 1974 widening plan

Appendix C
Chloride test results

Unprotected bridge decks

All tests taken at the depth of the top mat of reinforcing steel following ASTM C 1218.

Structure	avg Cl at top mat ppm	avg Cl at top mat LB/CY	year deck placed	age of deck
S008 14151	324	1.23	1963	50
S012 15268	1463	5.56	1961	47
S034 23671	145	0.55	1969	40
S045 00360	1221	4.64	1960	43
S045 00552	637	2.42	1960	43
S077 01143	150	0.57	1957	44
S080 34812	2324	8.83	1966	42
S091 21901	1658	6.3	1964	46
S103 05579	437	1.66	1966	44
S275 18888	995	3.78	1968	40
count	10	10		
average	935	3.55	1963	44
std dev	728	2.77		
avg - std dev	207	0.79		
avg + std dev	1663	6.32		

Bridge Decks with concrete overlays

Structure	year Deck placed	year Overlay placed	Type of Overlay	Year of first Cl test	chlorides before overlay ppm	chlorides before overlay LB/CY	Year of second Cl test	chlorides after overlay ppm	chlorides after overlay LB/CY
S080 07661L	1971	1989	4.00	1988	124	0.47	2011	603	2.29
S080 07661R	1971	2016	4.01	1988	37	0.14	2011	371	1.41
S080 07690L	1971	1989	4.00	1988	287	1.09	2012	645	2.45
S080 07690R	1971	2016	2.00	1988	382	1.45	2012	847	3.22
S080 05035L	1974	2000	2.00	1999	158	0.6	2013	342	1.3
S080 05035R	1974	2000	2.00	1999	129	0.49	2013	324	1.23
S080 05466L	1974	2000	2.00	1999	124	0.47	2013	600	2.28
count	7								
average	1972.3	2001.4		1992.7	177	0.67	2012.1	533	2.03
std dev					117	0.44		195	0.74
avg - std dev					60	0.23		339	1.29
avg + std dev					294	1.12		728	2.77

Appendix D

Service Life Modeling

Rebar corrosion in reinforced concrete is typically divided into two stages: initiation and propagation. The initiation period is the time it takes for the chloride ions to diffuse from the concrete surface to the rebar and reach a threshold concentration (350 ppm) above which rebar will start to corrode. Propagation time is the time from the corrosion initiation to when enough corrosion product has formed at the steel surface to create a concrete crack or delamination.

Most practitioners will assume between 3 to 6 years for the propagation time (NCHRP 558) (*Life-365 Service Life Prediction Model and Computer Program for Predicting the Service Life and Life-Cycle Cost of Reinforced Concrete Exposed to Chlorides Version 2.2.1 User's Manual*, the Life-365 Consortium III, January 2014). Literature has tried to calculate the propagation time using variables such as the cover-depth, concrete tensile strength, diameter of the reinforcement bar and percent of section loss. These calculations determine the amount of corrosion product created with section loss and then determine the amount of section loss needed to create enough internal stress in the concrete to create a crack or delamination. This calculation can be quite laborious and is generally not implemented regularly. Since a propagation time of 3 to 6 years is short in relation to the overall service life of concrete, assuming a value for propagation time does not have a significant effect on the overall service life model. That is why most practitioners will assume a value between 3 and 6 years.

The initiation phase is primarily modeled by the transport of chloride into concrete using Fick's Laws of Diffusion. Fick's Laws are based on kinetic theory and the random motion of molecules. Fick's First Law of Diffusion indicates that the rate at which chloride transports through concrete is proportional to the concentration per unit distance and a constant that is based on the concrete properties. Fick's Second Law of Diffusion provides a time dependency to the model. Using the Fick's Laws, an equation for the chloride concentration with depth of a permeable solid element can be developed (Equation 1).

Using Equation 1 the time to reach the chloride threshold at the depth of steel can be determined by making $C_{(x,t)}$ equal to the chloride threshold, x equal to the cover-depth and then solving for t. As a result, Equation 1 is rearranged into the form presented in Equation 2, which is used to determine the duration of the initiation phase.

$$C_{(x,t)} = C_o\left(1 - erf\,\frac{x}{2\sqrt{D_c t}}\right) \qquad \text{Equation 1}$$

erf	Mathematical error function
$C_{(x,t)}$	Chloride concentration at depth x after exposure time t
C_0	Constant chloride concentration at the surface
D_c	Diffusion coefficient
t	Time
x	Depth from exposure surface

$$t = \frac{1}{4D_c}\left(\frac{X_{cover-depth}}{inverse\ erf\left(1 - \frac{C_{threshold)}+C_O}{C_O}\right)}\right)^2 \qquad \text{Equation 2}$$

ACI Life 365 and NCHRP 558 *Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements* use chloride profiles of cores collected in the field to determine C_0 and D_c for modeling the initiation phase (Equation 1). These variables are determined by conducting a least square error regression analysis of the chloride profile using Equation 1.

In order to understand the risk of chloride-induced corrosion for the Bridges S075 17062 and S075 17596, the first step was to use the chloride test data to determine C_0 and D_c for each core (Table 7). Core O-3, 5 and 16 were not used in this analysis because, O-3 was too short, and cores 5 and 16 did not follow Fickian diffusion. For cores 5 and 16, most likely the chloride diffusion was influenced by a nearby crack or joint.

Then, statistical analysis of the data was carried out for the original and widened section of each bridge (Table 8). As can be seen from Table 8, the calculated surface chloride concentrations in the widened sections for both bridges were below the corrosion initiation threshold. Therefore, if the surface of the bridges is protected from the chlorides, such as by reapplication of a waterproofing membrane, there is no risk of chloride-induced corrosion of the reinforcement in the widened sections.

For the original sections, the remaining length of the propagation period from 2020 was calculated for diffusion coefficients ranging from the average minus standard deviation to average plus standard deviation. Also, a range of cover-depths was used in this calculation from 1.7 in to 7.5 in for Bridge S075 17062 (representing average cover-depth minus one standard deviation over the piers to one average cover-depth plus one standard deviation in the spans) and from 2.9 in to 3.3 in for Bridge S075 17596 (representing the average cover-depth plus/minus one standard deviation). As can be seen in Figure 12a, the reinforcement over the piers in Bridge S075 17062 is at risk for the chloride-induced corrosion at the present time or in the near future. However, for the deeper reinforcement in the spans there is no risk of corrosion for the next 100 years. For Bridge S075 17596, there is also no risk of chloride-induced corrosion in the foreseeable future as long as the deck continues to be protected from deicing salts (Figure 12b). Exposing the deck to deicing salts will increase the chloride surface concentration and change the rate of diffusion thus reducing the remaining service life of the deck.

Table 7: Calculated Surface Concentration and Diffusion Coefficient Values

Bridge S075 17062		
Core	**Chloride Surface Concentration (ppm)**	**Diffusion Coefficient (in^2/yr)**
E-1	275	$12.9\cdot10^{-3}$
E-2	190	$14.2\cdot10^{-3}$
O-3	938	$33.9\cdot10^{-3}$
O-4	1075	$83.2\cdot10^{-3}$
O-5	N/A	N/A
O-6	869	$44.2\cdot10^{-3}$
E-7	233	$1.2\cdot10^{-3}$
E-8	217	$1.7\cdot10^{-3}$
E-9	256	$1.7\cdot10^{-3}$
O-10	1086	$36.7\cdot10^{-3}$
O-11	156	$26.6\cdot10^{-3}$
E-12	235	$4.2\cdot10^{-3}$
O-13	2607	$21.8\cdot10^{-3}$
E-14	142	$2.3\cdot10^{-3}$
E-15	314	$4.9\cdot10^{-3}$
O-16	1528	$40.5\cdot10^{-3}$

Bridge S075 17596		
Core	**Chloride Surface Concentration (ppm)**	**Diffusion Coefficient (in^2/yr)**
1	307	$0.2\cdot10^{-3}$
2	170	$1.2\cdot10^{-3}$
3	230	$0.9\cdot10^{-3}$
4	360	$8.1\cdot10^{-3}$
5	N/A	N/A
6	711	$5.8\cdot10^{-3}$
7	1,182	$9.4\cdot10^{-3}$
8	186	$0.1\cdot10^{-3}$
9	149	$29.5\cdot10^{-3}$
10	353	$63.5\cdot10^{-3}$
11	611	$60.1\cdot10^{-3}$
12	176	$0.7\cdot10^{-3}$
13	1,156	$2.0\cdot10^{-3}$
14	1,451	$46.5\cdot10^{-3}$
15	266	$0.2\cdot10^{-3}$
16	N/A	N/A

Table 8: Surface Chloride Concentration and Diffusion Coefficient Statistics

Bridge	Section	Variable	Average	Standard Deviation	Minimum	Maximum
S075 17062	original	C_0	1,381 ppm	605 ppm	869 ppm	2,607 ppm
		D_c	$41.0\cdot10^{-3}$ in^2/yr	$20.1\cdot10^{-3}$ in^2/yr	$21.8\cdot10^{-3}$ in^2/yr	$83.2\cdot10^{-3}$ in^2/yr
	widened	C_0	221 ppm	44 ppm	142 ppm	274 ppm
		D_c	$5.5\cdot10^{-3}$ in^2/yr	$5.6\cdot10^{-3}$ in^2/yr	$1.2\cdot10^{-3}$ in^2/yr	$14.2\cdot10^{-3}$ in^2/yr
S075 17596	original	C_0	758 ppm	456 ppm	230 ppm	1,461 ppm
		D_c	$24.6\cdot10^{-3}$ in^2/yr	$27.2\cdot10^{-3}$ in^2/yr	$0.9\cdot10^{-3}$ in^2/yr	$60.1\cdot10^{-3}$ in^2/yr
	widened	C_0	209 ppm	63 ppm	149 ppm	307 ppm
		D_c	$4.6\cdot10^{-3}$ in^2/yr	$11.0\cdot10^{-3}$ in^2/yr	$0.0\cdot10^{-3}$ in^2/yr	$29.5\cdot10^{-3}$ in^2/yr

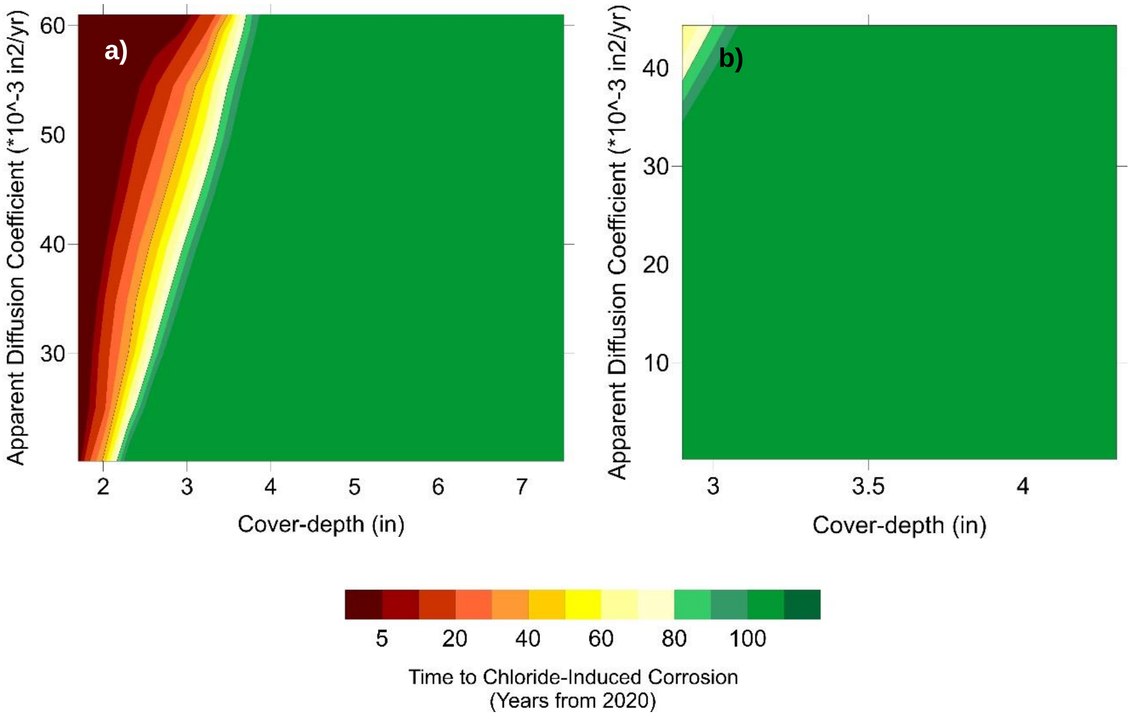

Figure 12: Projected Time to the End of the Propagation Period. Showing Bridge S075 17062 at left and Bridge S075 17596 at right