Character, Extent, and Severity of Corrosion in Continuously Reinforced Pavements in South Dakota

Allen L. Jones, 1 Nadim Wehbe, 2 and Stephanie Klay 3

ABSTRACT

A major drawback of continuously reinforced concrete pavement (CRCP) is the potential for the steel reinforcement to corrode. The corrosion process is initiated and accelerated when deicing chemicals, used extensively during winter maintenance, penetrate through the cracks and reach the steel reinforcement. The expansion of the corroded reinforcement leads to spalling of concrete and rapid degradation of the pavement section.

CRCPs constructed in South Dakota as part of the original Interstate Highway System (pre-1995) have performed well with little to no observed corrosion of steel reinforcement. However, recent surveys conducted by the South Dakota Department of Transportation (SDDOT) indicate a strong likelihood of potential future problems in newer CRCPs (post-1995) with corrosion of the reinforcing steel. SDDOT is interested in determining cost-effective maintenance and rehabilitation strategies to maintain and extend the life of in-service CRCPs exhibiting unexpected levels of cracking and distress for the post-1995 CRCPs.

An extensive field and laboratory testing program was initiated to define the character, extent, and severity of corrosion in CRCPs constructed in South Dakota since 1995 and to identify factors and interactions among factors that contribute to observed levels of corrosion. Field evaluations included detailed crack mapping, half-cell potential measurements, dust sample profiling, and core sampling. Dust samples were tested for chloride content, and the core samples were examined under a scanning electron microscope for imaging, X-ray spectrum, semi-quantitative analysis of the X-ray spectrum, and elemental mapping. The results of the field and laboratory testing were used to evaluate the reinforcement condition of eight representative sites across South Dakota.

The results of this study show that the distress in CRCP is not necessarily the result of reinforcement corrosion. Corrosion of the reinforcement occurred only at locations of cracks in the CRCP where chlorides could penetrate and reach the reinforcement. In intact (uncracked)

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concrete, the chloride content from deicing salts was insignificant and was below the threshold for corrosion at the level of the reinforcement. The half-cell potential Numeric Magnitude Technique was not a good indicator of the probability of corrosion in CRCP. However, in general, a strong correlation was observed between the crack density in the pavement and elevated half-cell potential readings.

INTRODUCTION

Both jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavement (CRCP) have been used extensively for interstate highways in the upper Midwest. In contrast to JPCP, where reinforcement is only present in the form of dowel bars placed at transverse joints, CRCP does not incorporate transverse joints but is instead reinforced with a mesh of longitudinal and transverse steel bars throughout the section. In CRCP, the thermal effects cause frequent tight transverse cracks, typically spaced at intervals of 2 to 4 ft (0.6 to 1.2 m), to develop in the concrete pavement. The amount of steel reinforcement is designed to control further widening of these cracks.

Prior to the 1960s, no CRCP projects had been constructed in South Dakota (Johnston 2009). Due to the lack of data on the performance of CRCP in the State, two test sections of CRCP were constructed in 1963 on Interstate 90 (I-90) in South Dakota. After 5 years of monitoring, the SDDOT observed smaller average crack widths and less frequency of cracks than what was considered in the design of the pavement; furthermore, after 45 years of being in service, neither section of pavement had required significant rehabilitation.

A subsequent analysis of performance and life-cycle costs of concrete pavements in South Dakota was conducted in the 1990s (Johnston 2009). Due to the favorable results exhibited by CRCP, the SDDOT adopted CRCP as a high-quality construction alternative to JPCP for interstate pavement. Between 1995 and 2009, the SDDOT replaced over 250 mi (400 km) of two-lane interstate highway with CRCP. The goal of using CRCP was to create high-performance roadway sections that required limited maintenance. However, after being in service for less than 15 years, several of these pavement sections showed signs of undesired distress, including Y-cracking, network cracking, and cluster cracking. At some locations, the crack interval was as frequent as 1 ft (300 mm), which led to loss of concrete in the form of moderate-severity to high-severity punchouts. It was thought that the punchout failures could become more frequent due to the cracking patterns that promote this type of failure.

Based on these observations, the SDDOT conducted a study entitled “Impact, Cause, and Remedies for Excessive Cracking in CRC Pavement” (Johnston 2009). The purpose of the study was to identify design, construction, and material alternatives that would optimize future CRCP construction. Subsequently, modifications to concrete gradation, steel content, centerline chair assembly, steel depth, aggregate preparation due to environmental factors, cement content, and curing compound application rates were incorporated into projects that were scheduled for the following construction season. The study also concluded that the impact on performance from progressing corrosion of the reinforcing steel due to deicing chemicals was uncertain and, therefore, the service life estimates for in-place CRCP may have been too optimistic at the time. The study concluded that there was uncertainty of the effect of deicing chemical on the steel
reinforcement in CRCP built in South Dakota between 1995 and 2009 and recommended that the extent of corrosion in CRCP in South Dakota interstate pavements and potential corrosion mitigation strategies be evaluated, if appropriate. That is the focus of this study.

The study presented in this paper was undertaken to assess the reinforcement corrosion extent in CRCP in South Dakota. The main objectives of this study were to 1) determine the character, extent, and severity of corrosion in CRCP constructed in South Dakota since 1995; 2) identify factors and interactions among factors that contribute to observed levels of corrosion; and 3) if appropriate, develop cost-effective maintenance and rehabilitation mitigation strategies for treatment of CRC pavements with corrosion problems so that the service life of the pavements can be achieved. Work on the third objective is still in progress and is not reported in this paper.

EXPERIMENTAL METHODS

To evaluate the condition and extent of corrosion of CRCP in South Dakota, a combination of field and laboratory methods were used. Field methods included general observations, crack mapping, concrete dust sampling, concrete core sampling, and half-cell potential measurements of CRCP sections across South Dakota. Laboratory work consisted of chloride ion analysis on concrete cores and dust samples, and scanning electron microscope (SEM) analyses on the reinforcement in core samples obtained from CRCP.

The increase in the use of deicing salts on roadways since the 1960s has caused significant corrosion to reinforced concrete structures (ACI 2001; Stratfull 1973; Stark 1989; Virmani and Clemeña 1998). Since the most common method for chlorides to penetrate concrete is by diffusion, chloride ion testing of the concrete was performed. However, for cracked CRCP, crack persistence and width can also have an effect on the severity and rate of corrosion. Therefore, crack mapping was also performed.

Given that the chloride ion testing is a destructive test, a complementary nondestructive test was needed for the project in evaluating statewide CRCP. The half-cell potential test served this purpose. Electrical potential has been shown to be an indicator of the corrosion process in reinforcing steel (Richardson 2002; Smith and Hashemi 2005; Virmani and Clemeña 1998). The copper–copper sulfate electrode was employed for this study by using a half-cell potential device. The half-cell potential measurements can be analyzed using the Numeric Magnitude Technique (NMT) and/or the Potential Difference Technique (PDT) (ASTM 2009). The NMT assigns potential value ranges that indicate the probability of corrosion (table 1) where more negative half-cell potential values indicate a higher probability that the steel reinforcement is corroding. The PDT compares the relative difference between measurements.
Table 1. Probability of Corrosion Based on Copper-Copper Sulfate Electrode (After ASTM 2009)

<table>
<thead>
<tr>
<th>Potential Measurement</th>
<th>Indication</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; -200 mV</td>
<td>90% probability that no reinforcing steel is corroding</td>
</tr>
<tr>
<td>-200 mv to -350 mV</td>
<td>Corrosion activity is uncertain</td>
</tr>
<tr>
<td>&lt; -350 mV</td>
<td>Greater than 90% probability steel is corroding</td>
</tr>
</tbody>
</table>

In addition to chloride ion determination and half-cell potential measurements, a SEM was used to analyze reinforced concrete specimens obtained from the cores. High-resolution images of specimens can be used to determine the degree of corrosion based on loss of cross-sectional area of the reinforcement. The SEM can also be used to create a chemical profile of the sample in evaluating corrosion.

A condition evaluation of CRCP pavements in the field was performed in three stages: investigation of baseline sites, general observation sites, and a statewide evaluation. Investigation at baseline sites included an in-depth evaluation of sites that were near a metropolitan area (Sioux Falls, South Dakota), as well as less rigorous evaluations of two other sites. General observations sites were used to obtain data when SDDOT was removing concrete as part of pavement repair. A statewide evaluation of the severity and extent of corrosion on interstate highways at representative sites throughout South Dakota was conducted to determine the overall condition of CRCP in the State. The results from the statewide CRCP evaluation were compared to the baseline sites to determine the character, extent, and severity of corrosion in CRCP in South Dakota. Factors that contribute to observed levels of corrosion were then identified.

Baseline Sites

Three CRCP sites in the Sioux Falls area were chosen as baseline sites for a detailed CRCP evaluation. Table 2 summarizes the CRCP information for the three sites.

Table 2. Pavement Design Information for the Baseline Sites

<table>
<thead>
<tr>
<th>Attribute</th>
<th>Baseline Site Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I-29N, MRM 87</td>
</tr>
<tr>
<td>Concrete thickness (in.)</td>
<td>8</td>
</tr>
<tr>
<td>Cover thickness (in.)</td>
<td>4</td>
</tr>
<tr>
<td>Longitudinal bar size (No.)</td>
<td>6</td>
</tr>
<tr>
<td>Design longitudinal bar spacing (in.)</td>
<td>8</td>
</tr>
<tr>
<td>Design longitudinal bar depth (in.)</td>
<td>3</td>
</tr>
<tr>
<td>Transverse bar size (No.)</td>
<td>4</td>
</tr>
<tr>
<td>Design transverse bar spacing (in.)</td>
<td>36</td>
</tr>
</tbody>
</table>

1 in. = 25.4 mm
Crack mapping at the sites was conducted by laying out a grid on the pavement surface for a 100-ft-long by 12-ft-wide (30-m-long by 4-m-wide) traveling lane and tracing the crack distribution on a scaled grid. The widths of the cracks were also measured using a crack-width gauge. The crack maps were digitally recreated, and the average crack width and crack density were determined for each site. The crack density was defined as follows:

\[
\text{Crack Density} = \frac{\sum \text{Crack lengths for entire pavement section}}{\text{Pavement surface area surveyed}}
\]  

Four core samples were collected from each of the three sites for the purpose of general observation, chloride ion testing, and SEM analysis. For each site, three cores were obtained at crack locations and one core at an uncracked pavement location. Cores were prepared for chloride ion analysis by slicing the core horizontally at 1-inch (25 mm) increments down to the level of the reinforcement followed by vertical slicing at 0.5-inch (13 mm) increments away from crack locations. Each of the resulting sections were subsequently pulverized for chloride ion testing. Figure 1 shows the horizontal slices of one of the cores with vertical sectioning shown in the rightmost horizontal section. Chloride testing was performed to determine the spatial distribution of the chlorides vertically from the pavement surface and laterally from crack location. The chloride test samples in this study were tested using an alternative potentiometric method (Clemeña and Apusen 2002).

A variable-pressure field-emission SEM was used to analyze the reinforcement in the core samples. The reinforcement sample was polished prior to placing it in the SEM. Testing using the SEM included secondary electron images, backscattered electron images, and X-ray element distribution maps. The results were used to determine elemental composition (presence or absence of chlorine) and extent of corrosion, if any, within each sample.

Four concrete dust composite samples were collected near core sample locations at all three sites. Dust samples were obtained using a roto-hammer to pulverize the concrete and a custom dust
vacuum device developed for this project. The dust samples consisted of four individual dust samples collected from pulverizing 1-inch concrete thickness at depths of 1, 2, 3, and 4 inches (25, 50, 75, and 100 mm) from the pavement surface. The four composite samples were then obtained by combining the dust samples of equal depths from three closely spaced drilled holes. Although dust samples were tested using the alternative potentiometric method, inductively coupled plasma mass spectrometry (ICP-MS) testing was also conducted for the purpose of method verification and to compare the results between the two methods. It should be noted that in some cases, the ICP-MS did not have low enough detection limits to provide useable results.

Half-cell potential measurements were obtained using a copper–copper sulfate reference probe with connections made to one reinforcing bar that was exposed at the edge of the pavement and sealed with epoxy. The test was conducted in accordance with ASTM C876-09 (ASTM 2009). One measurement was taken at each of the intersections of the grid lines used for crack mapping.

**General Observation Sites**

Other interstate pavement sections were also surveyed using visual observations (notes and photography) as well as crack mapping. One section was removed and reconstructed as part of scheduled maintenance, allowing for detailed observation of the reinforcing. Crack map surveys were completed along sections of pavement that were to be removed, with the reinforcement subsequently exposed for observation of general condition. Other sections of CRCP were also undergoing maintenance repairs including removal and replacement of full-depth sections at locations that exhibited severe deterioration. Notes and photographs were obtained at several locations to evaluate the condition of the reinforcement and pavement surface.

**Statewide Assessment Sites**

Eight sites of CRCP installed since 1995 in South Dakota were selected and evaluated for corrosion using general observation, half-cell potential measurements, and chloride ion analysis from concrete dust samples. Selection of the eight sites was based on the following criteria: pavement condition, pavement age, precipitation, pavement maintenance activities, and amount of deicer application. A rating system that reflected the selection criteria listed above was devised to compare the vulnerability to corrosion of the CRCP installed since 1995. The rating system is explained in detail in Klay (2011). Higher numerical rating indicates a pavement section that is less vulnerable to corrosion. Location of the sites was also considered to ensure wide geographic representation of the CRCP in South Dakota. Four regions outside of the Sioux Falls area were chosen to represent general geographic areas for the entire State of South Dakota. Two sites from each of these four regions were selected for the statewide CRCP evaluation. One of the two sites selected within each geographic region had a relative high rating (low susceptibility to corrosion), and one had a relative low rating (high susceptibility to corrosion). The four regions that contained the eight recommended sites that were subjected to additional evaluation are shown in figure 2.

The eight statewide sites were surveyed to compare half-cell potential and chloride ion results with the measurements taken at the baseline sites near Sioux Falls. At each of the eight sites, the traveling lane was examined by obtaining half-cell potential measurements and collecting dust
samples from the pavement section. The dust samples were obtained at gridline intersections with half-cell measurements that corresponded to the most negative 10 percent of readings.

![Figure 2. Regions for statewide evaluation.](image)

**RESULTS**

Results of the field measurements consisted of general observations, crack mapping, and half-cell potential measurements with results of the laboratory analyses consisting of chloride ion analysis and SEM testing.

**Baseline Sites**

*Crack Mapping*

The crack mapping showed cracking patterns similar to those reported in an earlier study by Johnston (2009); crack maps are shown in Klay (2011). A summary of the measured crack data is presented in table 3. The minimum crack widths, maximum crack widths, and crack densities for the three sites were all within the same order of magnitude.
Table 3. Crack Width Data for the Initial CRCP Sites

<table>
<thead>
<tr>
<th>Site</th>
<th>Minimum Crack Width (in.)</th>
<th>Maximum Crack Width (in.)</th>
<th>Crack Density (ft/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-29N, MRM 87</td>
<td>0.006</td>
<td>0.033</td>
<td>0.681</td>
</tr>
<tr>
<td>I-29N, MRM 68</td>
<td>0.010</td>
<td>0.023</td>
<td>0.331</td>
</tr>
<tr>
<td>I-90W, MRM 411</td>
<td>0.004</td>
<td>0.020</td>
<td>0.814</td>
</tr>
</tbody>
</table>

0.001 inch = 0.0254 mm; 1 ft/ft² = 3.28 m/m²

Chloride Content

If reinforced concrete is introduced to sufficiently high concentrations of chlorides from deicing salts, the protection of the embedded steel reinforcement bars is compromised (Virmani and Clemeña 1998). The concentration at which this occurs is referred to as the chloride threshold level (Böhni 2005). Previous work performed by the Federal Highway Administration (Clear 1976) suggests a value of 0.033 percent $\text{Cl}^-$ by mass of concrete or 0.2 percent chloride by mass of binder material. The chloride threshold value of 1.244 lb of chloride per cubic yard (0.738 kg/m³) of concrete was used in this study to determine if the chloride content in the concrete would be a cause of concern with respect to corrosion.

The measured vertical and lateral chloride content distributions for the Sioux Falls sites are presented in figure 3 and figure 4, respectively. These results are based on the potentiometric chloride testing. Figure 3 shows that the chloride ion concentration is typically above the threshold within the top 1 inch (25 mm) of the pavement section, and from depths of greater than 1 inch (25 mm) the chloride concentrations reduce below the threshold. Figure 4 shows the lateral distribution of chloride concentration away from cracks locations at a depth of 1.5 inch (38 mm) from the pavement surface. The chloride content was above the threshold in the concrete within 1 inch (25 mm) from the crack surface. At more than 1 inch from the crack, the chloride content was below the threshold.
Figure 3. Chloride concentration versus depth in the dust samples collected at baseline sites.
(1 in. = 25.4 mm, 1 lbs/yd³ = 0.5933 kg/m³)

Figure 4. Lateral chloride data at depth of 1.5 inches from core samples obtained at baseline sites.

**Half-Cell Potential**

Figure 5 presents the cumulative frequency distribution for half-cell potential at the baseline evaluation sites. The figure also shows the ranges of the corrosion probabilities suggested in ASTM C876-09. Note that for the baseline sites MRM 68 and 87, the majority of data are in the high probability of steel corroding or in the uncertain range. For site MRM 411, the data are
nearly distributed between the two ranges. No data were in the less than 90 percent probability of corrosion occurring.

![Figure 5. Half-cell cumulative frequency distribution at baseline sites (spring 2011).](image)

Equipotential contour maps were also constructed using the half-cell data. The equipotential contour map for the measurements at MRM 87 (spring 2011) is presented in figure 6 as an example of the NMT method presented in ASTM. The white lines on the map represent crack traces.
Figure 6. MRM 87 equipotential contour map.
**SEM Testing**

A representative SEM test result is presented in figure 7 for site MRM 87. The secondary electron image in figure 7 shows a thin layer of possible corrosion on the outer edge of the rebar. Further examination of the SEM elemental mapping showed that the layer consisted of iron oxides, silicon, calcium, aluminum, and manganese. The absence or low presence of chlorine was most likely due to the formation of a passive layer on the surface of the steel bar. Also, since chlorine levels were low or absent, it is unlikely that deicers had any corrosive effects on the reinforcement at this location. The energy dispersive spectrum of the steel bar sample showed only the presence of iron peaks. With the exception of one core at MRM 411, these results were typical of reinforcement in the core samples taken from the baseline sites. Figure 8 shows a core slice through a longitudinal reinforcing bar at site MRM 411. The sampling section intercepted several cracks in the concrete, and corrosion was detected at locations around the bar perimeter. The loss of cross-sectional area of the polished section of reinforcement due to corrosion was determined to be approximately 0.026 in² (16.8 mm²), or 6.0 percent, and was considered inconsequential in relation to pavement performance.

![Possible corrosion (??)](https://example.com/image.png)

*Figure 7. Secondary electron image of possible corrosion zone (MRM 68).*
General Observation Sites

The CRCP at several other sites was observed by the researchers prior to demolition as part of prescheduled reconstruction along I-29 and I-90. General observation (notes and photographs) and crack mapping revealed that several sections of pavement had been previously patched. Most patches were filled with asphalt, while some were filled with concrete. The reinforcement steel in some of the patched locations showed signs of corrosion. Within a patch, localized corrosion resulting in a noticeably reduced cross-sectional area of reinforcement was observed, which likely indicated that deicing salts or water intrusion contributed to the corrosion process. However, minimal localized corrosion was observed on steel reinforcement from intact CRCP that had been demolished (figure 9).

Figure 8. Corrosion observed in core at site MRM 411.
Observation at other CRCP repair sites showed several signs of distress, including longitudinal cracking, transverse cracking, and spalling. In addition to the surface distress, noticeable loss of cross section in the steel reinforcement was present at several locations within the sections that were exposed for repair. At some locations, nearly half of the cross section had deteriorated, as shown in figure 10. Figure 10 also shows observed rust stains and corrosion that occurred at the intersections of the longitudinal bars and transverse cracks.
Statewide Evaluation Sites

Crack Mapping

The crack densities of the various sites are presented in table 4. Transverse and Y-cracking was observed. However, little to no longitudinal cracking was typically identified within the 100-ft (30 m) sections that were observed and mapped. The crack densities in table 4 are within the same order of magnitude as the crack densities measured at the three baseline sites.

Table 4. Crack Densities for the Various Statewide Assessment Sites

<table>
<thead>
<tr>
<th>Site</th>
<th>MRM33</th>
<th>MRM44</th>
<th>MRM25</th>
<th>MRM54</th>
<th>MRM222</th>
<th>MRM246</th>
<th>MRM168NB</th>
<th>MRM168SB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack density (ft/ft²)</td>
<td>0.670</td>
<td>0.453</td>
<td>0.383</td>
<td>0.244</td>
<td>0.334</td>
<td>0.290</td>
<td>0.371</td>
<td>0.322</td>
</tr>
</tbody>
</table>

1 ft²/ft² = 3.28 m²/m²
Chloride Content

The results from the chloride analysis of the dust samples are presented in figure 11. Dust samples were obtained at the locations with the most elevated (most negative) readings, unless the traffic on the passing lane imposed a safety concern for collecting a dust sample. The data shows that the chloride values are generally above the threshold within the top 1 inch (25 mm) of the pavement surface and below the threshold at depths of more than 1 inch. It should be noted that the data points where the chloride content exceeded the threshold at depths of more than 1 inch corresponded to dust samples that were taken very close to a crack.

![Figure 11. Vertical chloride distribution for statewide CRCP evaluation.](image)

Half-Cell Potential

Figure 12 presents the cumulative distribution for half-cell potential at the statewide assessment sites. Note that the data for five of the sites were nearly within the 90 percent probability of corrosion occurring. One of the sites was distributed between 90 percent probability of corrosion occurring and the uncertain region, and two of the sites mostly within the uncertain range.
ANALYSIS AND DISCUSSION

General Observations

Transverse and Y-cracking of CRCP was observed at the baseline sites, however, longitudinal cracking was observed at only two of the three baseline sites (MRM 87 and MRM 411). The variation in crack density at the baseline sites was attributed to differing construction periods. The pavements at MRM 68, MRM 87, and MRM 411 were built in 2001, 1999, and 1997, respectively. The newest pavement section had not been exposed to as much traffic or as many winter maintenance seasons as the other two sections. Also, the cover depth at MRM 68 (5.5 inches (140 mm)) is greater than the cover depths of the other two sections (3.5 to 4.0 inches (89 mm to 102 mm)); this may have prevented additional cracking.

The cracks observed at the baseline sites were generally wider than those observed during the statewide evaluation. Also, the two baseline sites that had crack densities greater than 0.5 ft/ft² (1.64 m/m²) exhibited longitudinal cracking. Conversely, only one of the eight statewide evaluation sites surveyed had a crack density of greater than 0.5 ft²/ft² (1.64 m²/m²), and no longitudinal cracks were observed in any of the sites included in the statewide evaluation. On average, the crack densities at the statewide evaluation sites were 44 percent lower than the crack densities at the baseline sites.
densities at the baseline sites. The increased crack densities observed at the baseline sites could make it easier for chlorides to reach the reinforcement and subsequent corrosion.

At visual observation, I-29 and I-90 repair sites also showed signs of corrosion. Exposure of the embedded reinforcing bars revealed that corrosion occurred at crack locations or at locations of cold joints where patches were present in repaired CRCP. This type of corrosion was localized and varied in severity from rust stains to severe loss of the bar cross section. Away from the cracks and cold joints, no signs of localized corrosion were observed. Due to the corrosion observed, it is reasonable to conclude that cracks or other discontinuities in the concrete surface (i.e., patched locations) allowed chlorides to intrude at these locations and subsequently corrode the reinforcement.

The general observations imply that cracking is an important factor when considering corrosion in the reinforcement of CRCP. Although it is theoretically possible for corrosion to instigate cracks in a reinforced concrete section, this is likely not the case in this study. Since no areas of localized corrosion were observed at nondistressed locations in the concrete, it is concluded that cracks preceded the corrosion that was visually observed.

**Chloride Ion Results**

The chloride analysis of concrete cores and dust samples provides further evidence that a cracked section of CRCP is more susceptible to corrosion than an intact section. In general, the chloride content was above the chloride threshold within 1 inch (25 mm) of the surface of the pavement, and decreased to below the threshold at a depth of more than 1 inch in the pavement section. Analysis of the vertical chloride profiles of the dust samples showed that only 2 of the 28 composite dust samples had chloride ion concentration values above the threshold value of 1.244 lbs/yd³ (0.738 kg/m³) at a depth of 3 to 4 inches (76 to 102 mm). Therefore, the reinforcement at these locations in the pavement was likely not susceptible to corrosion initiation.

The lateral chloride profiles accomplished via core slicing showed a similar trend in the chloride data. The chloride content exceeded the threshold near a crack location and decreased away from the crack in all but one core slice tested. Also, the chloride concentration was above the threshold within the first 0.5 inch (13 mm) of the crack in all but two of the lateral chloride profiles. Furthermore, the core slices closest to the reinforcement showed chloride concentrations above the threshold for the first full inch (25 mm) away from the crack. This shows that the chloride concentration in the pavement section will only reach the threshold concentration deep in the section if it is near a crack, even though the concrete above this sample has not reached the chloride threshold.

**SEM Results**

Corrosion was detected in one of the four reinforcement samples analyzed using SEM. The sample containing the corroded reinforcement exhibited corrosion at crack locations but with only a 6.0 percent loss in cross section. Since several cracks intersected this reinforcement location and corrosion was evident at these crack locations, it is likely that chlorides intruded through these cracks. The absence of corrosion in the reinforcement of the other two cracked
cores that were tested indicates that a section of pavement can be cracked but the reinforcement may not be corroded.

No corrosion or chlorides were present in the reinforcement of the uncracked core samples; this indicates that chlorides did not penetrate through the concrete matrix to reach the reinforcement and initiate corrosion. Finally, if corrosion of the reinforcement was caused by chlorides entering through the concrete matrix, one would have expected to find elevated chloride contents at the steel/concrete interface.

It was difficult to draw additional, definite conclusions regarding the presence of corrosion in CRCP statewide from these analyses since only four reinforcement samples were tested using SEM methods. The limited SEM results showed that corrosion did not occur at every cracked location in the CRCP surface, but it was possible for chlorides to intrude through cracks and discontinuities in the pavement to cause corrosion.

### Half Cell Potential Results

Recall that there are two methods used to interpret the results obtained from the half-cell test: NMT and PDT. Using the NMT, the cumulative frequency distributions of the half-cell potential measurements indicate that all sites have a high probability of corrosion or that the corrosion activity is uncertain. These results generally contradict the results of the general observations, chloride ion results, and SEM results. This brings into question the validity of the corrosion probability ranges of the NMT. There are several factors that influence half-cell potential readings, including concrete resistance, moisture content, oxygen concentration at the steel interface, and concrete cover depth (Gu and Beaudoin 1998; Elsener 2003). It is also important to note that much of the research performed on the half-cell technique was based on results from structural surveys of bridge decks, bridge columns, and buildings in which the concrete cover is much less than that used in pavements. It is also not clear from the literature if the NMT also applies to a reinforced pavement section on a soil subgrade. Due to the inability to accurately quantify the effects of concrete cover and soil subgrade, the NMT was regarded as an inappropriate method to evaluate the sites for corrosion.

The analysis of the values using the PDT was then considered. With this technique, the half-cell values are used to evaluate the relative difference between values in identifying areas of elevated half-cell measurements. This was accomplished by spatially comparing the change in half-cell potential with time and identifying areas of relative high and low half-cell potential. The areas with more negative half-cell potential values likely indicate areas of higher potential of corrosion. Since concrete resistance, moisture content, oxygen concentration at the steel interface, and concrete cover depth were likely consistent over the length of pavement within the same site, these variables will have a reduced effect on the interpretation of the half-cell results. Therefore, this approach was deemed more appropriate to determine areas that have a high potential of corrosion. The most negative, or elevated, half-cell potential readings were identified as areas of high corrosion potential.

To make such a comparison useful, analyses were considered to evaluate the half-cell potential data; this included correlating half-cell potential values to variables of interest. For this analysis, a percentage of the most negative half-cell readings were considered to be elevated readings. To
determine the percentage to be used in a correlation, several values were tested, including 10, 20, and 30 percent of the most negative readings at each site. After analyzing the data, it was determined that the most negative 20 percent of the half-cell readings recorded for each site had strong correlations with crack density. The half-cell measurements at each grid location were compared with the average crack density; this comparison resulted in a positive increase in the percentage of elevated half-cell measurements within a given crack density range as the crack density increased. This is shown in figure 13 for the three baseline sites. The established model is subsequently discussed.

Correlation models were established and tested to determine the significance of the relationship between crack density and half-cell potential measurements. The models were established by using the method of ordinary least squares. Correlation models were generated for each site using the following equation:

\[ P = p_0 \times e^{r \times C_d} \]  

(2)

where:

- \( P \) = Percent of elevated readings
- \( p_0 \) = Initial percent (parameter) of elevated readings
- \( r \) = Rate of increase in percent of elevated readings caused by crack density (parameter)
- \( C_d \) = Crack density

**Figure 13. Correlation models for baseline sites.** (1 ft/ft² = 3.28 m/m²)
The parameters of the established models were evaluated using the t-test statistic. The results of the correlations of the sites surveyed are summarized in Table 5. The table shows that all of the p-values indicate that the relationship between crack density and the \( p_0 \) parameter is insignificant. However, in this study the rate of the change in percent of elevated half-cell readings (\( r \)) was of more interest than the intercept of the equation (\( p_0 \)) since the goal was to determine if there was a correlation, as opposed to determine a model equation to relate half-cell potential to crack density. The \( r \) parameter was considered significant, moderately significant, or very significant at 8 of 11 CRCP sites surveyed. At seven of these eight sites, the correlation between crack density and elevated half-cell potential readings was positive. Finally, six of the seven sites that had significant, moderately significant, and very significant positive correlations had \( R^2 \) values above 0.70.

### Table 5. Summary of Half-Cell and Crack Density Correlation Results

<table>
<thead>
<tr>
<th>Site</th>
<th>( p_0 ) Estimate</th>
<th>( p_0 ) P-value</th>
<th>Level of Significance</th>
<th>( r ) Estimate</th>
<th>( r ) P-value</th>
<th>Level of Significance</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRM 87</td>
<td>0.003</td>
<td>0.32</td>
<td>Insignificant</td>
<td>5.199</td>
<td>&lt; 0.01</td>
<td>Very significant</td>
<td>0.92</td>
</tr>
<tr>
<td>MRM 68</td>
<td>0.054</td>
<td>0.40</td>
<td>Insignificant</td>
<td>3.527</td>
<td>0.17</td>
<td>Insignificant</td>
<td>0.31</td>
</tr>
<tr>
<td>MRM 411</td>
<td>0.006</td>
<td>0.52</td>
<td>Insignificant</td>
<td>3.890</td>
<td>0.02</td>
<td>Moderately significant</td>
<td>0.76</td>
</tr>
<tr>
<td>MRM 33</td>
<td>0.003</td>
<td>0.50</td>
<td>Insignificant</td>
<td>5.361</td>
<td>0.01</td>
<td>Very significant</td>
<td>0.85</td>
</tr>
<tr>
<td>MRM 44</td>
<td>0.174</td>
<td>0.43</td>
<td>Insignificant</td>
<td>0.624</td>
<td>0.69</td>
<td>Insignificant</td>
<td>0.03</td>
</tr>
<tr>
<td>MRM 25</td>
<td>1.184</td>
<td>0.20</td>
<td>Insignificant</td>
<td>-4.799</td>
<td>0.07</td>
<td>Significant</td>
<td>0.39</td>
</tr>
<tr>
<td>MRM 54</td>
<td>0.116</td>
<td>0.30</td>
<td>Insignificant</td>
<td>2.422</td>
<td>0.31</td>
<td>Insignificant</td>
<td>0.11</td>
</tr>
<tr>
<td>MRM 222</td>
<td>0.023</td>
<td>0.49</td>
<td>Insignificant</td>
<td>5.797</td>
<td>0.09</td>
<td>Significant</td>
<td>0.57</td>
</tr>
<tr>
<td>MRM 246</td>
<td>0.018</td>
<td>0.29</td>
<td>Insignificant</td>
<td>6.182</td>
<td>&lt; 0.01</td>
<td>Very significant</td>
<td>0.75</td>
</tr>
<tr>
<td>MRM 168NB</td>
<td>0.028</td>
<td>0.15</td>
<td>Insignificant</td>
<td>4.249</td>
<td>0.00</td>
<td>Very significant</td>
<td>0.70</td>
</tr>
<tr>
<td>MRM 168SB</td>
<td>0.007</td>
<td>0.19</td>
<td>Insignificant</td>
<td>8.529</td>
<td>0.00</td>
<td>Very significant</td>
<td>0.93</td>
</tr>
</tbody>
</table>

### SUMMARY AND CONCLUSIONS

A baseline evaluation of CRCP was conducted to determine the extent of possible corrosion of CRCP on selected interstate highways in South Dakota. A CRCP evaluation was also performed on selected CRCP sites statewide to assess corrosion of other interstate sites relative to the baseline sites. The results of this assessment were compared to the results of the baseline evaluation of CRCP, and conclusions with respect to corrosion were formulated from these evaluations. The following are the conclusions established in this study.
Corrosion was only observed at cracked or patched locations. Reinforcement corrosion was not evident at areas away from cracked or patched locations, except in the case of the longitudinal reinforcement placed near the longitudinal joint. However, there were areas of severe spalling and cracks that did not show signs of corrosion. This leads to the conclusion that pavement distresses observed are not necessarily the effect of corroded reinforcement. This conclusion can aid in assessing the need for maintenance activities of CRCP in maintaining expected design life.

The chloride analysis showed that chlorides have the ability to penetrate through cracks and patched locations of CRCP. Therefore, crack density is an appropriate means of determining a CRCP section’s susceptibility to corrosion. The chloride analysis also showed that chloride contents were above the threshold level within the first lateral half-inch (13 mm) of the pavement section. This leads to the conclusion that only localized reinforcement at cracked locations is susceptible to corrosion caused by deicing salts. The chloride analysis of the dust samples that were obtained away from crack locations showed that chlorides have not penetrated to the reinforcement through intact concrete during the lifetime of the pavements surveyed in this study. Thus, in general, an intact pavement section is not susceptible to corrosion caused by deicing salts, and this finding can be factored into the assessment of long-life concrete pavements.

The SEM analysis of the reinforcement in the cores of two of the three samples that were obtained at crack locations showed no signs of corrosion. The reinforcement sample that was corroded showed evidence of chloride intrusion, however the corrosion was limited. This further confirms the conclusions that pavement distresses observed are not necessarily the effect of corroded reinforcement and that reinforcement at cracked locations is susceptible to corrosion caused by deicing salts.

The half-cell potential measurements at all sites were determined to fall in either the high probability or uncertain range with respect to corrosion according to ASTM C876-09. Many unquantifiable factors affect the half-cell potential test including concrete resistance, moisture content, oxygen concentration at the steel interface, and concrete cover depth. Although the half-cell potential method has been a valid indicator of corrosion for concrete buildings and bridge decks, by itself it was inconclusive in determining the probability of corrosion in CRCP during this study. Therefore, we recommend that more research be conducted on the half-cell potential device, as well as other alternatives in identifying a reliable nondestructive test in assessing corrosion in CRCP. It was also shown that there was a strong correlation between crack density and elevated half-cell potential readings. Therefore, crack density can be used as an indicator to the susceptibility of a CRCP site to corrosion.

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