DEPARTMENT OF TRANSPORTATION

Evaluation of Electrochemical Chloride Extraction, Fiber Reinforced Polymer Wraps, and Concrete Sealers for Corrosion Mitigation in Reinforced Concrete Bridge Structures

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16. Abstract (Limit: 250 words)

Reinforced concrete bridge substructures in Minnesota and other northern climates possess an elevated risk for chloride-induced corrosion damage, which can reduce capacity and shorten service life. Research was performed in 1997 to investigate new strategies for corrosion mitigation, including electrochemical chloride extraction (ECE) and installation of fiber reinforced polymer (FRP) wrap, which were applied on several corrosion-damaged piers on a 30-year-old bridge in Minneapolis. This report presents the results of follow-up research performed to assess the condition of the treated piers after 20 additional years of service, in order to understand the long-term effectiveness of the strategies implemented.

The combination of ECE treatment and FRP wrap installation was found to be very effective, with no concrete distress or probable corrosion activity identified in the treated elements. Poor or mixed performance was observed with all other strategies, including both ECE treatment followed by application of a penetrating sealer and FRP wrap installation that was not accompanied with ECE. In addition, significant chloride contamination occurred in all of the subject piers within the 20 years since the initial study, indicating that neither FRP wrap nor concrete sealers prevented the ingress of new chlorides in the manner in which these systems were installed in the initial study. The findings indicated that performing ECE treatment, or installing FRP wraps, did not alone eliminate the risk of future corrosion activity. The most effective corrosion mitigation strategy to extend the service life of reinforced concrete bridge substructures was to minimize water and chloride exposure.

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FINAL REPORT

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Chloride-induced corrosion is the primary deterioration mechanism for reinforced concrete bridge structures in Minnesota and many other states that utilize heavy volumes of deicing salts and chemicals to clear snow and ice from roadway surfaces in winter months. The deicing salts contain chlorides, which dissolve in water and can then penetrate concrete surfaces that are wetted. Corrosion can initiate if chlorides penetrate to the depth of embedded steel reinforcement in accumulations at or above critical concentrations. Although bridge decks are the most susceptible, bridge substructures can experience significant moisture and chloride exposure during service as a result of leaking expansion joints, leaking drainage systems, or positioning within plow zones. As a result, reinforced concrete bridge substructures in Minnesota and other northern climates possess an elevated risk for chloride-induced corrosion damage. Corrosion damage typically consists of concrete cracking, delaminations, or spalls, and loss of section at reinforcing bars and can result in reduced capacity and shortened service life.

Concrete repairs can be performed to address the areas of corrosion damage that develop. Repair work traditionally consists of removing and replacing areas of unsound concrete. However, the repairs can be short-lived if the remaining concrete that surrounds the new patch material is chloride contaminated. Additional corrosion mitigation strategies may be required to prevent continued damage and extend the service life of the repaired structure. The most common strategies are cathodic protection, which involves providing a sacrificial anode that will corrode preferentially to the embedded steel reinforcement and providing surface protection to prevent the ingress of new moisture and chloride into the repaired elements.

Electrochemical chloride extraction (ECE) is a relatively new strategy for corrosion mitigation. ECE incorporates the principles of cathodic protection into a temporary treatment process intended to remove chloride ions from sound, but contaminated, concrete. The short-term effectiveness of ECE treatment has been investigated in multiple field and laboratory studies, and significant chloride content reductions have been reported. However, little information is available regarding the long-term performance of in-service bridge structures that have been treated with ECE. Similarly, fiber reinforced polymer (FRP) wraps are a relatively new technology that is most commonly used for strengthening applications but may also prevent moisture and chloride ingress. Several studies have reported that FRP wraps may function as impermeable barriers and might also significantly decrease corrosion rates. However, the presence of the wrap can inhibit traditional assessment techniques and conceal the presence of any new or recurrent distress or corrosion activity within a treated element.

In 1997, the Minnesota Department of Transportation (MnDOT) commissioned a research project in association with the University of Minnesota to study the effectiveness of ECE treatment and FRP wrap installation as corrosion mitigation strategies for chloride-contaminated reinforced concrete bridge substructures. The project focused on portions of the substructure of MnDOT Bridge No. 27831, a bridge that was constructed in 1967 and carries Trunk Highway 394 over Dunwoody Boulevard just west of downtown Minneapolis. Several reinforced concrete piers that exhibited chloride contamination and areas of corrosion damage were selected for study. The main objective of the research was to assess the

benefits of ECE and its effectiveness at slowing or stopping the corrosion process, both when performed as a standalone treatment and when combined with various surface protection measures installed to prevent future moisture ingress. The surface protection measures evaluated included three different types of carbon or glass FRP wraps and three different types of concrete sealers. Half of the elements included in the study received ECE treatment and the other half did not. The partner ECE-treated and non-treated elements with similar surface protection measures were evaluated to allow for relative comparisons of performance. These comparisons were primarily derived from the results of field surveys, half-cell potential testing, and chloride content testing, which were performed both before and after the ECE treatment process. The ECE process was found to have significantly reduced the level of chloride contamination that had been present within the treated elements and had re-passivated the embedded reinforcement reducing the risk of future corrosion. However, at some locations, post-ECE chloride contents remained above corrosion threshold levels. Continued monitoring and chloride sampling was recommended to evaluate the long-term performance of the treatments and to determine the most effective combinations.

This report presents the results of a follow-up research project sponsored by MnDOT to evaluate the condition of the five piers of Bridge 27831, included in the initial study, after they had experienced 20 additional years of service. The evaluation tasks that were performed generally mirrored those of the initial research, including field inspection, delamination surveys, non-destructive testing, and laboratory analyses of chloride content. The collection of similar types of data and information, including locations of distress, corrosion potentials and chloride content levels, allowed the current conditions to be directly compared to those that existed before and immediately after the different corrosion mitigation strategies were installed 20 years prior. The primary objective of this project was to evaluate the long-term effectiveness of the different corrosion mitigation strategies to collect information that could better inform future decisions by MnDOT regarding rehabilitation of chloride-contaminated and corrosion-damaged reinforced concrete bridge substructures.

The following generally summarizes the conclusions drawn from this follow-up research project:

- All of the pier sections that received ECE treatment and FRP wrap installation exhibited very good performance, including no new or recurrent concrete distress and no evidence of probable corrosion activity after 20 years of service. Although highly effective, this strategy was judged to be the least cost-effective in comparison to other approaches studied at this bridge.
- The highest rate of recurrent distress (88 percent) was observed in areas that received ECE treatment followed by the application of a penetrating sealer. These results indicate that ECE treatment does not eliminate the risk of future corrosion activity, and the effectiveness of the treatment can be short-lived if chloride and moisture exposure persists.
- Mixed performance was observed at sections where ECE treatment was not performed, but FRP wrap was installed. No new or recurrent distress was identified in pier sections where no obvious evidence of deck joint leakage problems was apparent, but a significant rate of recurrent distress and areas of highly negative corrosion potential were observed at another section below a leaking deck joint. The installation of FRP wrap, alone, was not effective at mitigating corrosion in areas of high moisture exposure.

- Three pier sections were maintained as controls (“no action”) and received no ECE treatment or surface protection. The majority of the distress, which was repaired as part of the initial study, recurred after 20 years. However, this approach was judged to be the most cost-effective corrosion mitigation strategy.
- ECE treatment resulted in re-passivation of the reinforcing steel and significant reductions in the extent of chloride contamination that was present in the elements treated. These conditions were not sustained. After 20 years, chloride levels exceeded pre-ECE treatment levels at almost all locations sampled, and areas of moderate or high-risk corrosion potentials were identified in most sections that received ECE treatment.
- Significant chloride contamination occurred in all five piers within the past 20 years. Chloride levels typically exceeded the corrosion threshold at the depth of the reinforcing steel. These results indicate that none of the FRP wrap types or concrete sealer products prevented the ingress of new chlorides into the concrete in the manner in which these systems were installed for this study.

The following generally summarizes the recommendations drawn from this study:

- The most effective corrosion mitigation strategy to extend the service life of reinforced concrete substructures was to minimize water and chloride exposure through diligent maintenance, effective repair, or timely replacement of bridge deck joints and deck drainage systems.
- FRP wrap systems may not function as waterproofing barriers when installed on existing bridge elements. The presence of an FRP wrap will obscure the concrete surface, inhibiting visual inspection, and alone will not prevent corrosion activity and distress from developing or recurring behind the FRP.
- Combining ECE treatment with FRP wrap installation was the most effective corrosion mitigation strategy evaluated in this study (the only treatment that resulted in no recurring distress), but it also was the most expensive. This strategy may only warrant consideration for applications or structures with special circumstances that would make periodic interventions for traditional deck or substructure repairs impractical.

CHAPTER 1: INTRODUCTION AND BACKGROUND

1.1 INTRODUCTION

The majority of bridge structures in Minnesota are constructed of reinforced concrete. The service life of a reinforced concrete bridge is significantly influenced, and often controlled, by the extent of deterioration that develops in the bridge deck and substructure over time. Deterioration typically manifests as a result of exterior exposure. The most common deterioration mechanism is corrosion of the steel reinforcement, which is embedded within a concrete element, and the most common cause of corrosion is chloride contamination. The primary sources of chloride contamination are deicing salts and chemicals used to clear roadway surfaces in the winter months. Because deicing salts and chemicals have been used extensively in northern climates over the past several decades, the prevention and repair of chloride-induced corrosion damage in concrete bridge structures have been significant challenges in Minnesota and many other states, for many years.

In 1997, the Minnesota Department of Transportation commissioned a research project to evaluate promising new techniques that were developed for the rehabilitation of corrosion-damaged concrete bridge structures. These techniques included a temporary electrochemical treatment capable of permanently removing chloride ions from contaminated elements and, by doing so, created a potential for significant extension of bridge service life. The research project was implemented on select portions of a bridge located in Minneapolis that exhibited significant chloride contamination and corrosion damage following 30 years of service and exposure.

This report presents the results of a follow-up research project that was performed to evaluate the condition of the treated bridge elements after 20 additional years of service. The goal of this project was to evaluate the long-term performance and effectiveness of the different corrosion mitigation techniques that were implemented in the hope that the findings would better inform future decisions regarding the repair of other chloride-contaminated or corrosion-damaged reinforced concrete bridge substructures in Minnesota.

1.2 BACKGROUND

1.2.1 Corrosion in Reinforced Concrete Bridge Structures

Corrosion of steel reinforcement is a common cause of degradation in concrete bridge structures, an example of which is shown in Figure 1.1. Initially, steel reinforcement embedded in concrete is protected from corrosion by a stable thin protective oxide film that develops on the surface of the bars as a result of the highly alkaline environment (pH of 12.5 to 13.5) produced by the cement hydration process [1]. Corrosion will not occur as long as this passive film remains intact. However, the loss of the film can allow active corrosion to occur in the presence of moisture and oxygen. There are two primary mechanisms that can destroy the passive film and permit corrosion of the reinforcement to initiate — carbonation of the surrounding concrete and chloride contamination.

Figure 1.1 Typical corrosion damage in concrete bridge substructures

Carbonation of concrete occurs when carbon dioxide present in the atmosphere diffuses through pores in the concrete and reacts with moisture and cement hydration products [1]. As such, carbonation typically starts at an exposed surface of the concrete and progresses inward over time. The risk of carbonation-induced corrosion generally increases with age but it is primarily influenced by the depth of concrete cover over the reinforcing steel — greater cover equates to lower risk. The effect of the carbonation reaction is a lowering of the pH of the pore solution within the concrete. The protective passive film on the reinforcement will begin to break down once the pH of the surrounding concrete falls below about 10 or 11. Breakdown of the passive film can allow corrosion to develop.

In the absence of carbonation, the accumulation of chloride ions above a critical concentration (known as the chloride corrosion threshold) at the level of the reinforcement will also cause a breakdown of the protective passive film and allow corrosion to initiate. The most common source of chloride contamination within bridge structures in northern climates is exposure to deicing salts. The chlorides in deicing salts dissolve in water and the chloride ions can penetrate into concrete surfaces that are wetted, entering through the pore structure or cracks. The onset of corrosion is typically then governed by the time required for chlorides to penetrate to the depth of steel reinforcement in accumulations at or above the threshold concentration. Concrete bridge decks are directly exposed to chlorides during service. Bridge substructures are also susceptible to significant chloride exposure during service below leaking expansion joints or within plow exposure zones. Accordingly, both bridge decks and substructures possess an increased risk for corrosion activity and damage.

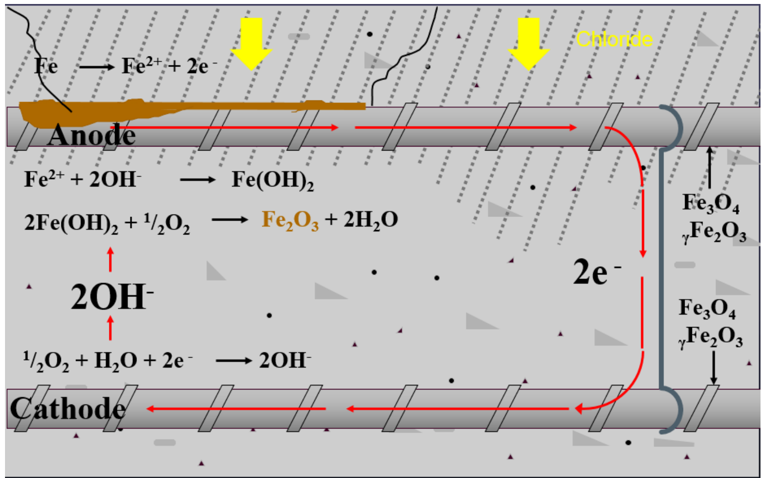


Figure 1.2 Schematic of macrocell corrosion in reinforced concrete

The corrosion process generates corrosion product (rust), which accumulates at the surface of the embedded reinforcing bar, as shown above in Figure 1.2. The corrosion product occupies a significantly greater volume than the original steel, which was consumed (oxidized). The increase in volume creates internal expansive pressures that can eventually result in cracking, delamination, and ultimately spalling of the cover concrete. The presence of concrete distress will accelerate both the rate and severity of corrosion damage due to increased availability of moisture and oxygen. If not addressed in a timely manner, corrosion will reduce the cross-sectional area of the steel reinforcement. The continued loss of concrete and reinforcement area will diminish structural capacity over time.

Concrete repairs can be performed to address areas of corrosion damage. Repairs involve removing concrete that is unsound, and removing corrosion from reinforcing bars, prior to placing new concrete. Repairs may not be durable if the remaining concrete behind or around the patch location is chloride contaminated. Although the reinforcing steel within the patch area will be protected by the new placed concrete material, these sections of steel will become cathodic and promote corrosion of the steel in the surrounding concrete that may still contain high levels of chlorides. This phenomenon is commonly referred to as the “ring anode effect” and results from the large chloride gradient that can occur between areas adjacent to a repair (chloride contaminated) and the repair itself (chloride-free). Additional corrosion mitigation strategies, such as cathodic protection, may be required to enhance the durability of the repaired structure and reduce the likelihood of perpetual maintenance and repair.

1.2.2 Chloride-Induced Corrosion

The onset of chloride-induced corrosion is governed by the time required for chloride to penetrate through the concrete cover and build up at the bar depth to a threshold value. The chloride corrosion threshold is a theoretical concentration of chlorides in concrete at which steel corrosion may initiate. The specific chloride threshold at any given location on the steel surface is dependent on a number of conditions within the concrete, including cement content and chemistry, moisture conditions, steel chemistry and surface conditions, and proximity and condition of other embedded steel elements.

Uncoated mild steel embedded in concrete typically has a corrosion threshold of approximately 0.20 percent total (acid-soluble) chloride by weight of cement in non-carbonated concrete [1]. This is the lowest chloride corrosion threshold at which corrosion may be expected to initiate if all other conditions conducive to corrosion are present. The likelihood, severity, and rate of corrosion increases with increases in chloride concentrations above this threshold.

However, it is important to recognize that corrosion of the reinforcing is not certain at chloride concentrations at and above the corrosion threshold, since multiple environmental factors affect the influence of chloride concentration on corrosion. Further, since testing of existing structures is performed on samples of concrete not cement, a conversion is needed based on the content of cement in the concrete mix and the unit weight of the sampled concrete. For typical normal weight concrete, a value of 0.030 to 0.035 percent by weight of concrete is often cited as the chloride threshold, as this provides a conservative limit to prevent corrosion [1]. At the same time, published literature contains widely varying statements about the chloride corrosion threshold, indicating that there is not a consensus within the industry and practice. For the purposes of this report, a chloride concentration of 0.035 percent by weight of concrete is the relevant corrosion threshold for uncoated steel reinforcement that will be used. This value is based on available research and the authors' experience.

1.2.3 Electrochemical Chloride Extraction

Electrochemical chloride extraction (ECE) is a corrosion mitigation technique intended to remove chloride ions from concrete and to re-passivate steel reinforcement bars. This technique is typically employed to supplement a traditional concrete repair approach (i.e., local removal and replacement of spalled and delaminated concrete) and treat chloride-contaminated concrete that remains. The primary objective of ECE treatment is to reduce the risk of ring-anode corrosion in unpatched areas by removing chloride ions from sound, but contaminated, concrete. ECE is similar in principle to impressed current cathodic protection systems, except that it is a temporary process that is performed and then removed, eliminating the need and costs associated with long-term cathodic protection system maintenance and continuous power supply requirements [1].

A typical ECE treatment consists of installing a temporary sacrificial metal anode on the exterior surface of a concrete element, encapsulating it within a conductive media (e.g., cellulose fibers soaked with an electrolyte), establishing an electrical connection from the external anode to the internal reinforcing steel, and passing a high current between the two [2]. Figure 1.3 shows installation of the anode and conductive media. The treatment runs continuously for up to 2 months. After the completion of the ECE treatment, the external anode and conductive media is removed.

Figure 1.3 In-progress installation of ECE system

The ECE process utilizes a low-voltage, direct-current (DC) electric field to drive free chloride ions away from the internal reinforcing steel and toward the externally mounted sacrificial anode, often a mild steel mesh. This is termed ionic migration. Refer to Figure 1.4. During the process, the rebar is negatively charged to repel negatively charged chloride ions, while the exterior mesh located on the surface of the contaminated element is positively charged. The current passed between the internal reinforcing steel and the external anode during ECE is approximately 50 to 500 times the current that would be supplied in an impressed cathodic protection system. In addition to reducing chloride levels, the electrochemical reactions that occur during the ECE process create hydroxyl ions at the internal reinforcing steel [2]. Hydroxol ions increase the alkalinity of the surrounding concrete, and this is claimed to re-passivate the embedded reinforcement through the regeneration of a thin protective oxide film around the bars.

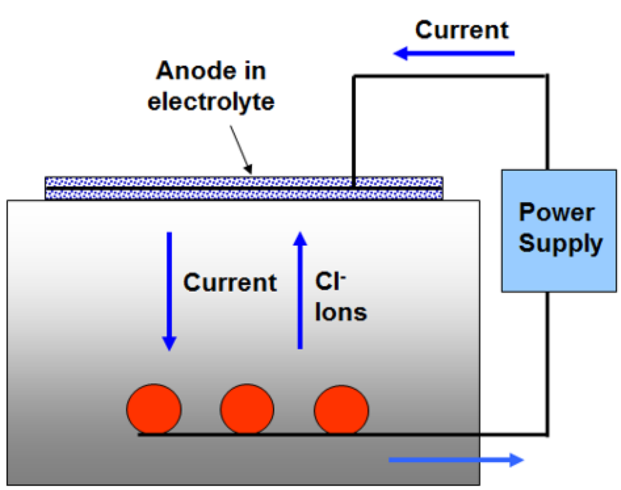


Figure 1.4 Schematic of the ECE process

The short-term effectiveness of the ECE process has been investigated as a component of several field and laboratory studies. These studies determined that ECE can significantly reduce the overall chloride content of contaminated structures, potentially in excess of 70 percent [3] [4]. However, and particularly in heavily contaminated elements, the level of remaining chlorides still approached or exceeded the corrosion threshold in certain locations. In addition, some studies have identified evidence or concerns that chlorides were being driven further inward behind the reinforcing steel instead of being extracted [5]. For these reasons, the long-term effectiveness of ECE treatments has remained in question.

There is little information available regarding the long-term performance of bridge structures that have been treated with ECE. However, the combination of reduced chloride content and re-passivation of the reinforcing steel is believed to suspend corrosion activity for some time. Resumption of the corrosion process would only be expected if chloride ions again accumulate at the depth of reinforcing steel in concentrations at or above threshold levels (though the threshold level itself may be influenced by the prior corrosion and subsequent treatment). Corrosion could occur as a result of residual chlorides diffusing back to the steel, or from new chloride ions that re-contaminate the treated element during continued service, or from a combination of these conditions.

1.2.4 Surface Protection

Surface protection measures are installed to limit the exposure of the concrete to moisture and deicing salts, mitigating the risk of new chloride ingress and possibly reducing moisture and oxygen ingress. This may act to prevent the initiation of corrosion or slow the process where corrosion has initiated. Available options for surface protection of concrete include, but are not necessarily limited to, the application of waterproof or water-resistive penetrating sealers, film forming coatings, or fiber reinforced polymer (FRP) composite wrap systems. The selection of an appropriate surface protection measure, if any, is a project- and condition-specific decision that should consider applicable advantages and disadvantages of each system. Surface protection measures may be installed with traditional concrete repairs and, if used, with cathodic protection measures or ECE treatments.

Most concrete sealers are transparent and allow for continued visual inspection of the concrete surface for distress and corrosion activity. Sealers work by making pores within the concrete less receptive to water (hydrophobic). Periodic reapplications are required to maintain sealer effectiveness. Film-forming coating systems typically offer more durable water penetration resistance and can themselves be inspected for evidence of staining or distress, which may be indicative of underlying problems. However, coating systems or surface finishes are typically opaque, inhibiting direct inspection of the underlying concrete. Delaminations in concrete are identified most routinely by hammer sounding the concrete surface and listening for hollow-sounding areas. Hammer sounding on thick coating systems can make determination of hollow sounding areas more difficult. Greater scrutiny is required to determine whether the coating or underlying concrete, or both, are unsound.

Figure 1.5 Typical FRP wrap installation on a bridge column

FRP wrap installation can create an impermeable surface and offer additional benefits, including strengthening and control of crack widening. FRP wraps may function as a water barrier as long as the material remains bonded to the concrete surfaces. Several studies have reported that confining composite wraps will not arrest corrosion but may significantly decrease corrosion rates [6, 7, 8]. However, FRP coverings may conceal all evidence of any existing distress or active corrosion activity. FRP wrap systems typically include an epoxy resin, the fiber reinforced polymer material, and a coating system that provides ultraviolet protection, producing a system that is both thick and opaque. The presence of an FRP wrap can effectively inhibit traditional concrete assessment techniques such as visual inspection, sounding, and half-cell potential testing. See Figure 1.5. Concrete surface distress may also be restrained by the confining nature of the FRP wraps, unlike a coating system where such distress would be expected to reflect through. As such, it is plausible that corrosion activity could continue unchecked within an element wrapped with FRP [9, 10].

Regardless of type, any surface coatings or systems that prevent water ingress may also introduce risks of trapping moisture and chloride within the subject element. The presence of oxygen and trapped moisture could facilitate the re-initiation of the corrosion process, even without new water or chloride ingress during service. Corrosion re-initiation following repair or ECE treatment is influenced by the level of chloride contamination that remains within the concrete.

1.2.5 1997 to 2000 MnDOT and University of Minnesota Research

In 1997, the Minnesota Department of Transportation (MnDOT) commissioned a research project in association with the University of Minnesota to study the effectiveness of ECE treatment as a corrosion mitigation strategy for chloride-contaminated reinforced concrete bridge substructures. The project focused on portions of the substructure of MnDOT Bridge No. 27831, which carries Trunk Highway 394 over Dunwoody Boulevard just west of downtown Minneapolis, MN. Reinforced concrete pier caps and columns that exhibited areas of corrosion damage were selected for study. These elements contained chloride content levels above the corrosion threshold due to many years of exposure to moisture and deicing salts resulting from leaking joints and drainpipes. This research project will herein be referred to as the 1998 study.

The main objective of the 1998 study was to assess the short- and long-term benefits of ECE and its effectiveness at slowing or stopping the corrosion process, both when performed as a standalone treatment and when combined with various surface protection measures installed to prevent future moisture ingress. The surface protection measures evaluated included three different types of carbon or glass FRP wraps and three different types of concrete sealers. Half of the elements included in the study received ECE treatment and the other half did not. The partner ECE-treated and non-treated elements with similar surface protection measures were evaluated to allow for relative comparisons of performance. These comparisons were derived from the results of field surveys, non-destructive evaluation, and laboratory studies, including half-cell potential testing and the collection of numerous chloride samples both before and after the ECE treatment process.

A comprehensive summary of the 1998 study was published in MnDOT Report No. MN/RIC-2000-24 titled "*Evaluation of Electrochemical Chloride Extraction and Fiber Reinforced Polymer (FRP) Wrap Technology*" [11] This report included background information regarding corrosion in concrete structures and the history, chemistry and past research associated with ECE. This background information was not repeated herein. The report also presented preliminary findings obtained by the date of publication. In general, the ECE process was found to have significantly reduced chloride concentrations and re-passivated embedded reinforcement within the treated elements. However, at some locations, post-ECE chloride contents remained above threshold levels and the increase in passivity was less pronounced. Continued monitoring and chloride sampling was recommended to evaluate the long-term performance of the treatments and to determine the most effective combinations.

1.3 RESEARCH OBJECTIVES AND GOALS

In 2017, Collins Engineers was retained by MnDOT to perform both special and routine inspections of Bridge No. 27831. The work included visual inspections and delamination surveys of the entire bridge substructure, among other tasks. In conjunction with this project, MnDOT retained the authors of this report in 2018 to perform follow-up research associated with the pier and column groupings that had been included in the 1998 study. The objective of the research was to evaluate the performance of the ECE treatments and surface protection measures as corrosion mitigation strategies for reinforced concrete bridge elements after 20 years of service. The goal of the research was to obtain information that could better inform future decisions by MnDOT regarding the implementation of repair and corrosion mitigation strategies for other reinforced concrete bridge substructures suffering from chloride contamination and corrosion damage in Minnesota. The research will be termed herein as the 2018 study.

The work performed by the authors as a component of the 2018 study generally mirrored the assessment work that was performed in the 1998 study, including limited field inspection, non-destructive testing, and laboratory analyses of chloride content of the subject bridge elements. Information provided by Collins Engineers, including visual and delamination survey results, was also analyzed. The collection of similar types of data and information allowed the current conditions to be compared to those that existed before and immediately after ECE treatment and the installation of various surface protection measures.

1.4 REPORT ORGANIZATION

This report includes five chapters and an afterword. Chapter 2 provides a description of the project site, a more detailed synopsis of the 1998 study, and a summary of site conditions documented by MnDOT between the 1998 study and the 2018 study. Chapter 3 summarizes the field and laboratory investigation work performed as a component of the 2018 study. The investigation work included visual inspection and sounding, half-cell potential testing, concrete resistivity testing, FRP bond testing, and chloride content testing of 46 core samples, which were extracted by MnDOT personnel from locations immediately adjacent to the locations of chloride sampling associated with the 1998 study. Chapter 4 presents an analysis of findings, including comparison of current distress, chloride, and corrosion conditions to those documented in 1998 both before and after ECE treatment. Chapter 5 presents conclusions and recommendations.

An afterword is included after the end of Chapter 5 written by Paul Pilarski, the MnDOT Technical Liaison for this project. The afterword is intended to provide an owner/agency perspective on the motivation for the 1998 and 2018 research studies and the findings of these projects.

CHAPTER 2: PROJECT SITE CONDITIONS AND HISTORY

2.1 BRIDGE 27831

2.1.1 Description

MnDOT Bridge Number 27831 carries seven lanes of traffic on Trunk Highway 394, travelling in the eastbound and westbound directions, over Dunwoody Boulevard in Minneapolis, Minnesota. See Figure 2.1. The bridge has 50 spans and a total length of approximately 2,700 feet, with over 400,000 square feet of bridge deck area on two parallel bridge structures, one eastbound and one westbound. The typical bridge construction consists of a reinforced concrete deck supported by prestressed precast concrete girders, spaced at 10 feet on center, which typically span approximately 50 feet between multi-column reinforced concrete piers. See Figure 2.2. Strip seal expansion joints are present in the bridge deck directly above approximately every third pier.

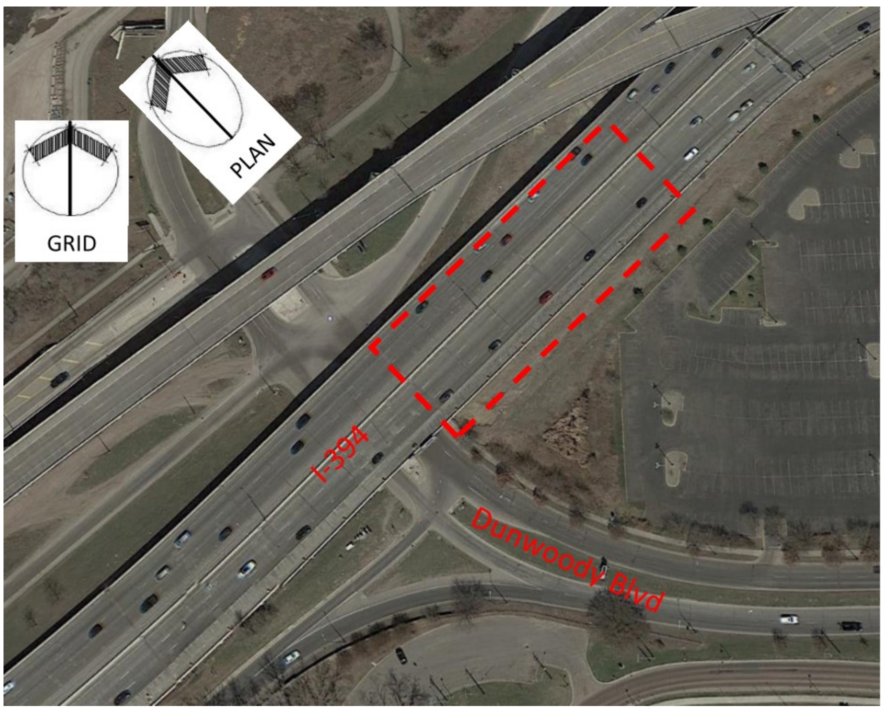


Figure 2.1 Aerial image of Bridge 27831 (obtained from Google® Earth)

The piers include a reinforced concrete pier cap which measures approximately 3 feet wide by 3 feet 9 inches deep. The pier caps are longitudinally reinforced with top and bottom layers of #11 reinforcing bars, typically spaced at 3-1/2 inches on center, with #5 stirrups typically spaced at approximately 21 inches on center. The pier caps are supported by either three or four reinforced concrete round columns that are 32 inches in diameter with either #9 or #11 reinforcing bars equally spaced around the perimeter, behind #3 spiral reinforcement. The specified clear cover for the reinforcement at the columns and pier caps was 2 inches. Refer to Figure 2.3.

Figure 2.2 Bridge 27831 substructure, looking east at Pier 34WB and Pier 34EB

A nomenclature system was established by MnDOT to uniquely identify the elements comprising the substructure of Bridge No. 27831 for inspection, maintenance and repair purposes. This nomenclature system has been utilized herein. Each of the piers was identified numerically, from west to east. Although the bridge is oriented at a skew relative to grid north, the pier caps of the westbound and eastbound substructures are distinguished by the designations “WB” or “EB.” Columns are identified alphabetically, starting from the north (i.e., W. Linden Avenue) to the south.

2.1.2 Initial Service History (1967 to 1997)

Bridge No. 27831 was originally constructed in 1967. In 1977, a low slump concrete overlay was installed on the bridge deck in concert with a deck repair project that included the replacement of the original strip seal deck joints with a “Type H” joint. In 1989, the bridge was widened as a component of a rehabilitation project. The scope of the rehabilitation work again included the replacement of all strip seal glands at deck joints. No contract substructure repair projects were performed at the three piers which are the subject of this research project (Pier 34, Pier 37, and Pier 40) between the original construction of the bridge and the initiation of the 1998 study.

Bridge Inspection Reports have been prepared by MnDOT personnel in conjunction with the routine inspections of Bridge No. 27831 since 1971. The reports provide history of the conditions that have been observed and documented by bridge inspection teams, including element ratings, data and distress. The following paragraphs summarize the general and specific conditions which were reported between 1971 and 1997 for the elements which are the focus of this report: the strip seal deck joints and the reinforced concrete pier caps and columns of the substructure. Pertinent dates when conditions were first observed or noted by MnDOT are cited in parentheses, as applicable.

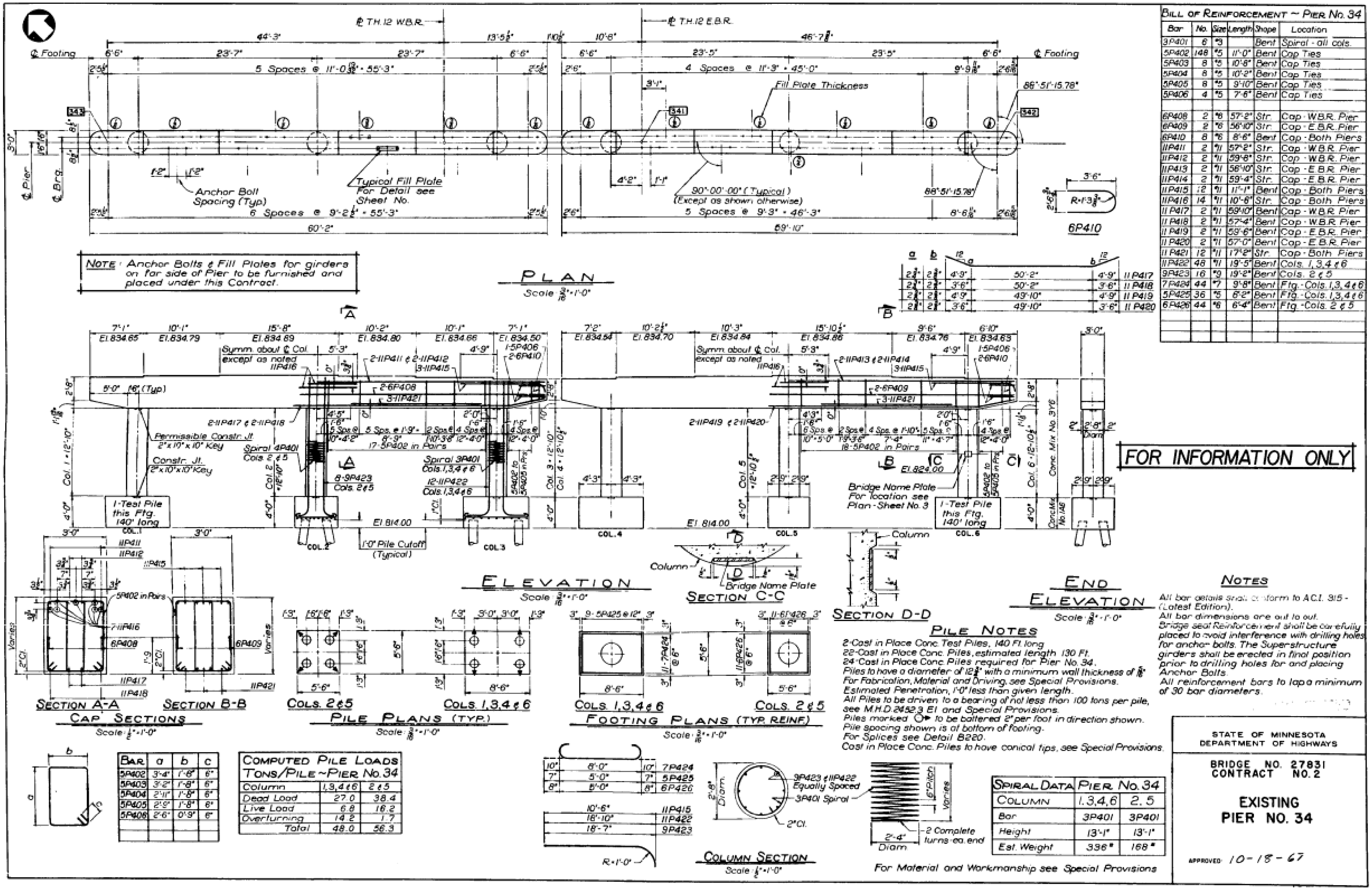


Figure 2.3 Bridge Drawings - Pier 34

Between 1971 and 1988, the deck joints were typically assigned an overall condition rating of 7, defined as “non-structural items in need of repair” or “minor items in need of repair by maintenance forces.” Inspection notes described general evidence of leakage (1972) at joints and deck cracks, conditions which persisted despite biennial deck crack sealing efforts and the 1977 overlay and strip seal gland replacement project. Strip seal gland material was also described as having “fallen out over many of the piers,” including specific mention of Pier 34 WB (1977). Discrete instances of torn or cut joint gland material were also reported (1980). Below deck level, observations of “rust stains” (1974) and then spalling and exposed corrosion strand (1977) were noted at the ends of prestressed girders below joints.

Cracking of pier caps was observed below fixed bearing locations near the west end of the bridge shortly after construction (1971) but was believed to be due to restraint issues. However, discrete spalling and delamination conditions which exposed corroded reinforcing steel developed at various pier caps shortly thereafter (1973). Approximately 10 years after original construction, more global observations of “rust stains” (1979), pier caps having cracked “badly” (1980) or “severely” (1982), and a progressive worsening of distress conditions were described in general notes, including that “areas of unsound concrete on pier caps and columns seem to be getting worse and more new ones are appearing” (1982). Pier cap spalling conditions grew to an estimated aggregate of 150 square feet of deterioration, concentrated “under expansion bearings” (1988). Similarly, below the pier caps, spalling and delamination conditions developed at the columns, including a note that “one column at Pier 34 has a 6’ by 1’-2’ by 2” deep vertical rebar popout” (1977).

The barriers and strip seal deck joints were replaced as a component of the 1989 rehabilitation project. The bridge deck was also widened, and the west end of the deck was removed and reconstructed. Between the completion of that project and the start of the 1998 study, the strip seal joints were consistently rated at Condition State 1, defined as “little to no deterioration” and “no leakage.” However, concrete distress continued to advance at the ends of the prestressed girders and the pier caps below the bridge deck. Documented distress included 90 to 100 square feet of delamination at Pier 34 (1992), 50 square feet of delamination at Pier 37 (1992), and 70 square feet of spalling and delamination at Pier 40 (1992). Cracking of the north column at Pier 40 was also noted (1992).

By 1997, the substructure of Bridge No. 27831 exhibited widespread concrete distress consistent with chloride-induced corrosion. Corrosion damage was concentrated below the expansion joints and strip seal joints in the bridge deck and included cracking, spalling, delamination and visible corrosion of reinforcement at the ends of prestressed girders, across the surfaces of the pier caps, and around the circumference of the concrete columns. These problems had persisted despite various repair and rehabilitation attempts. The pattern and nature of the distress was consistent with long-standing leakage of water through the deck joints, and from the deck drainage systems. The deck drainage systems included scuppers with downspouts which discharged below deck to traversing open troughs, positioned at the ends of approximately every third pier cap. The troughs frequently clogged with debris or build-up that prevented the free flow of water, instead producing splattering, leakage or spillage which wetted the pier caps and columns to which they were mounted. Joint and trough leakage allowed water to flow down and over the surfaces of the concrete elements located below, especially near gutter-line and scupper drainage areas. Discharged water included meltwater laden with chlorides from road salt applications during the winter months. In summary, in 1997, after 30 years of service, the substructure of Bridge No. 27831 was heavily chloride contaminated and corrosion damage was accelerating.

2.2 1998 STUDY

2.2.1 Overview

MnDOT partnered with the University of Minnesota in 1997 to conduct a study to research the effectiveness of ECE and evaluate the ability of the technique to slow or stop the corrosion process in chloride contaminated reinforced concrete bridge substructures in the state of Minnesota. While available research indicated that ECE was capable of significantly reducing chloride concentrations, the long-term effectiveness of the treatment was not fully understood. For this study, ECE treatments were followed by the installation of various surface protection systems intended to prevent future moisture ingress. The study was configured to allow the performance of different techniques to be evaluated, both initially and over time, through comparisons to similar elements with similar baseline conditions which did or did not receive similar treatments. The study was anticipated to run five years.

The substructures chosen for the study were multi-column piers under strip seal expansion joints, each of which exhibited corrosion damage. The study incorporated all or portions of five different pier caps and twelve columns directly below. The elements included in the study represented, by total surface area, approximately five percent of the bridge substructure. The selected elements included the entire pier cap and all three columns of Pier 34WB, Pier 34EB, and Pier 37WB, as well as the north end of the pier cap and column (D) at Pier 37EB and the north and south ends of the pier cap and columns (A and C) at Pier 40WB. See Figure 2.4.

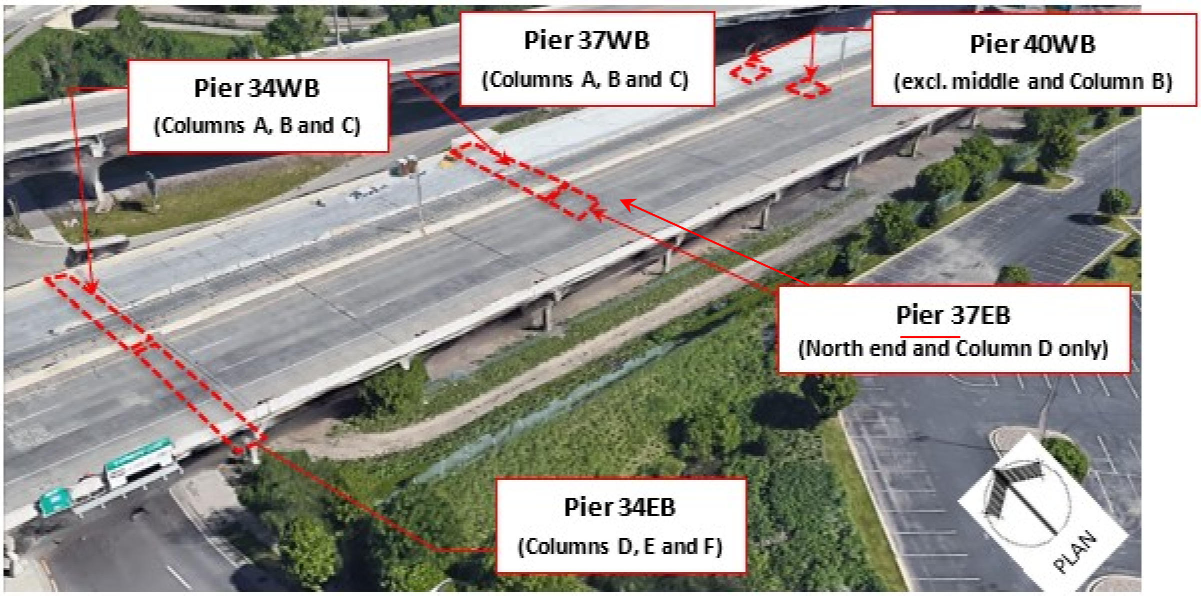


Figure 2.4 Aerial view of the portions of Bridge No. 27831 included in the 1998 study

2.2.2 Documentation of Initial Conditions

Initial condition assessments were performed on each of the twelve columns and five pier caps which were included in the 1998 study to identify all existing concrete surface distress. The assessments included both 100 percent visual inspection and hammer sounding surveys. Areas of previous concrete repair and evidence of corrosion-related damage were observed on all five pier caps and all but one of the twelve columns. The damage conditions included cracking, delaminations, and spalling. Locations of concrete distress conditions were physically marked on the concrete surfaces and documented, and repairs were subsequently performed by MnDOT maintenance crews. The repairs consisted of conventional chipping and patching, or crack filling techniques, and were performed using a cement-based repair mortar [11]. Refer to Figure 2.5 for an example of the surface repairs.

Figure 2.5 Example concrete repairs performed in 1997 (west face of Pier 40WB shown)

Refer to Figure 2.6 through Figure 2.10 for wire frame illustrations of the distress and repair locations, identified with blue shading, on all of the elements included in the 1998 study. For future reference, notes are embedded within these and other illustrations included within this chapter to identify the locations where ECE treatments and different surface protection measures were ultimately installed. The ECE treatment and surface protection measures are discussed further in the Section 2.2.3.

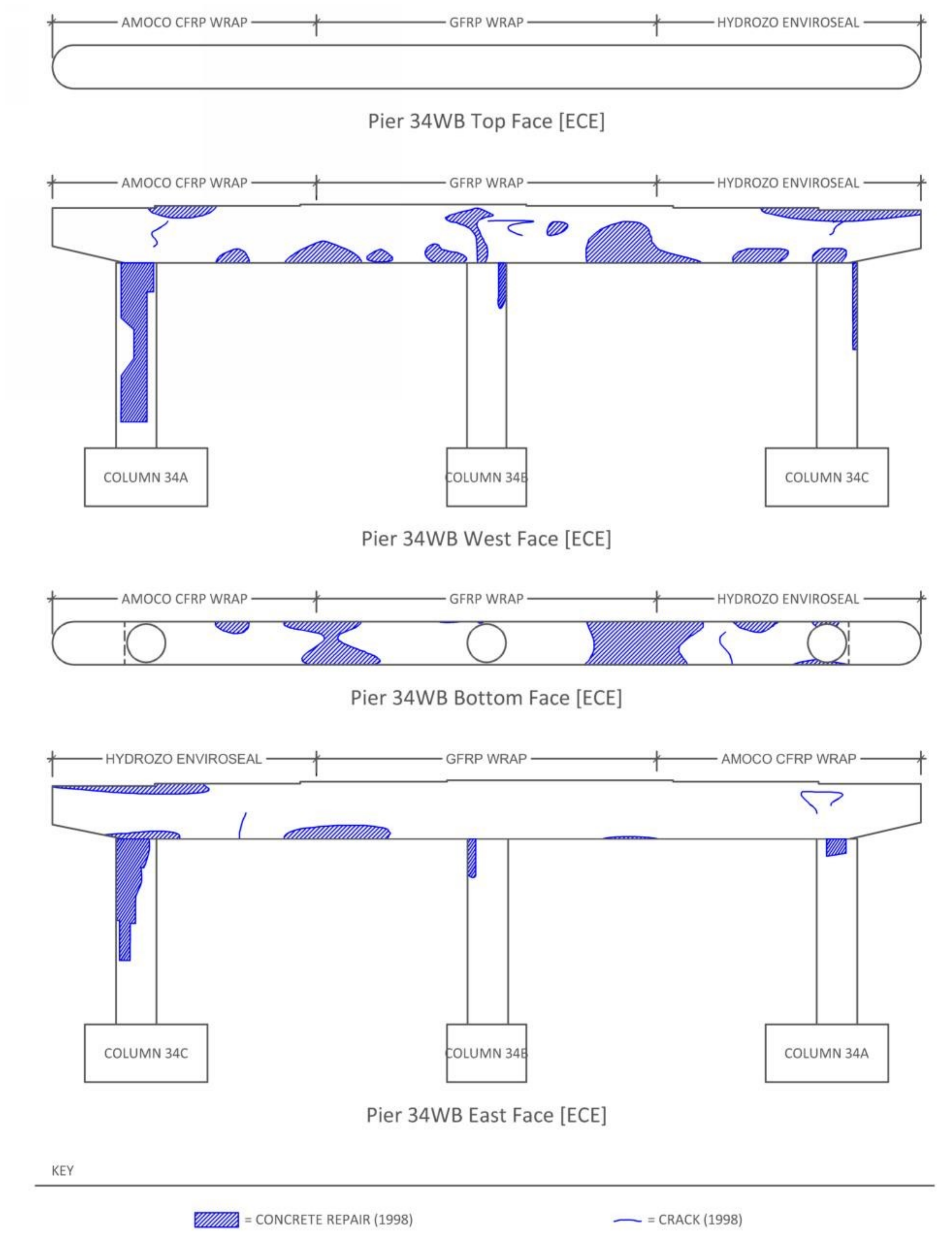


Figure 2.6 Pier 34WB - Areas of concrete distress and repair (blue shading) - 1998 study

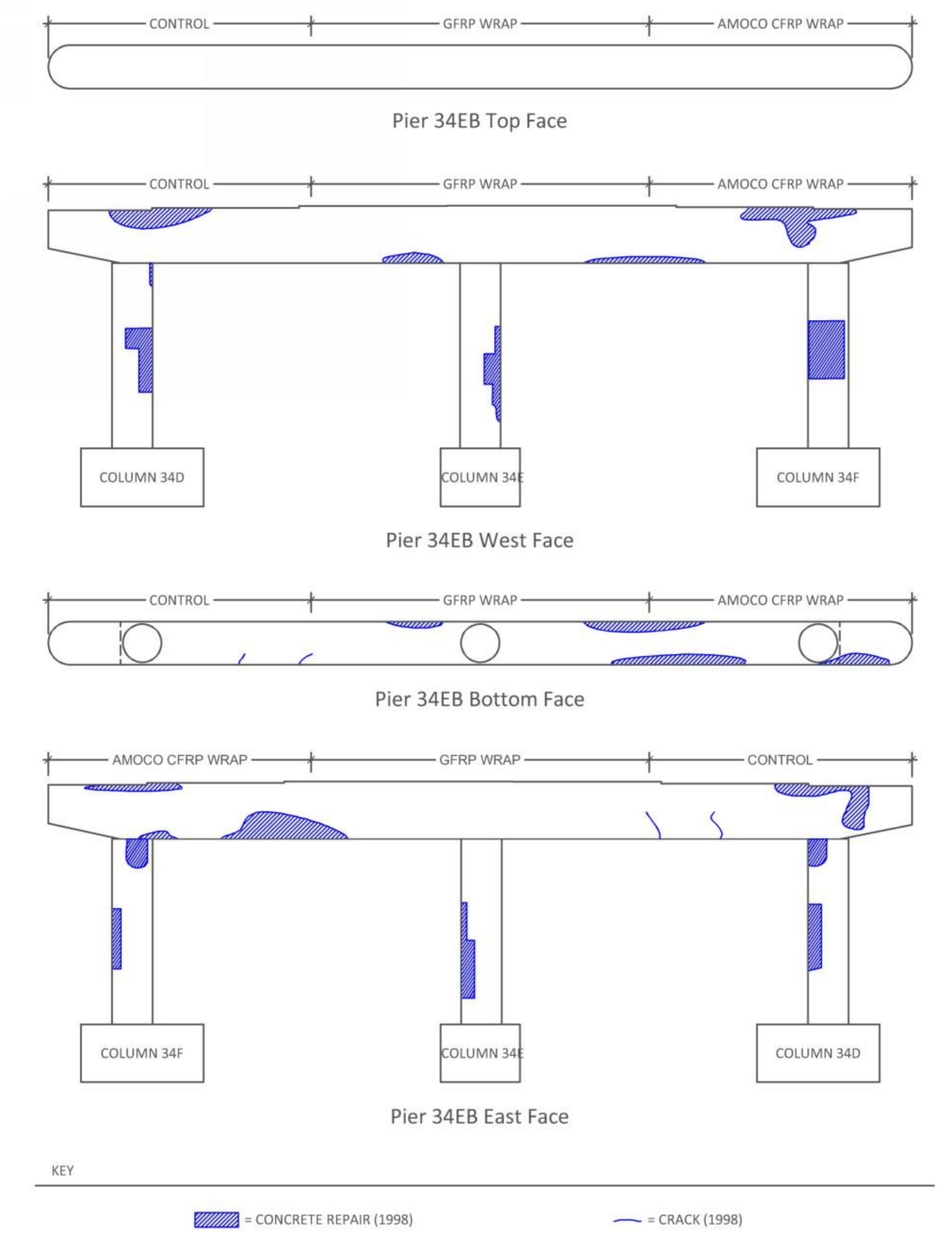


Figure 2.7 Pier 34EB - Areas of concrete distress and repair (blue shading) - 1998 study

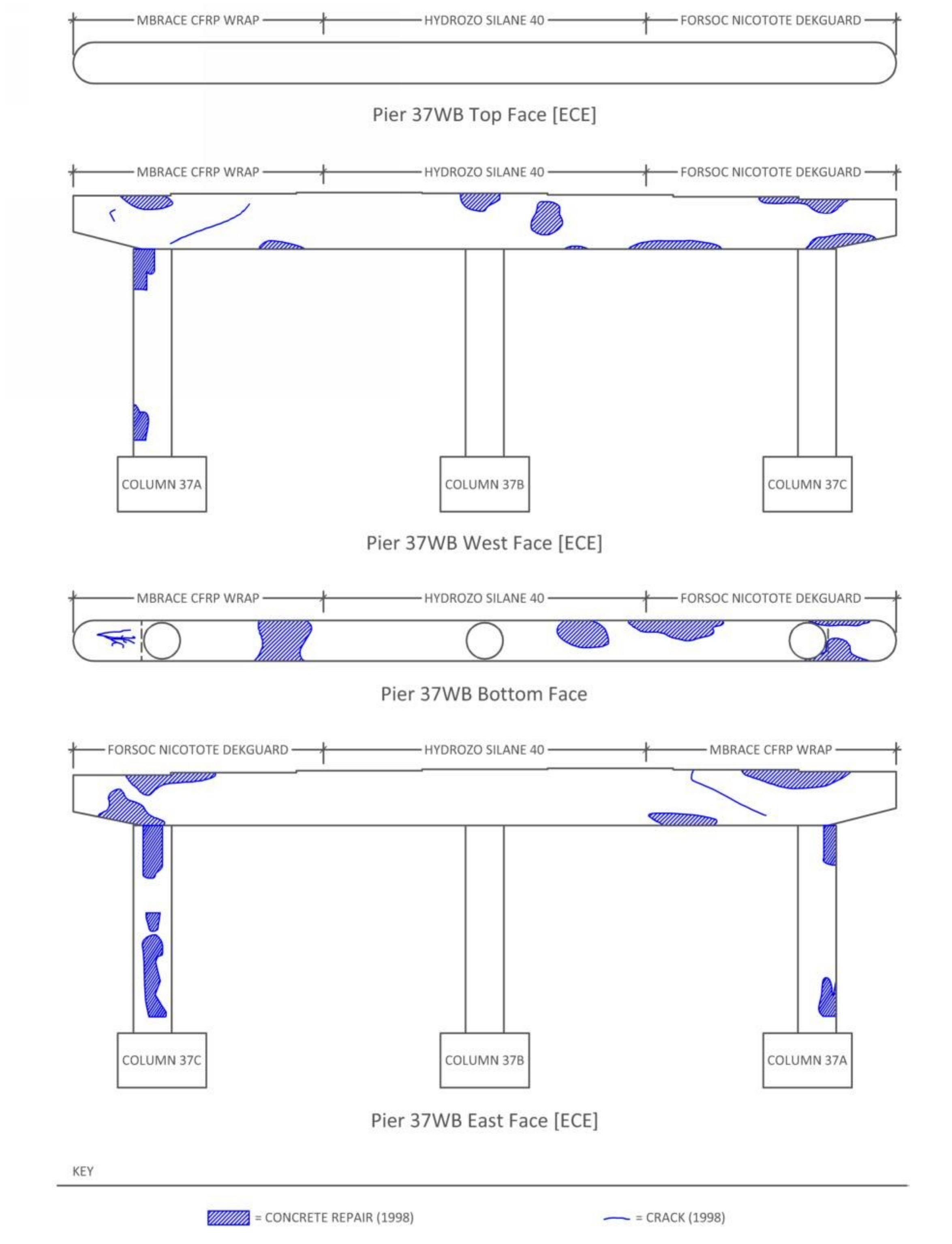


Figure 2.8 Pier 37WB - Areas of concrete distress and repair (blue shading) - 1998 study

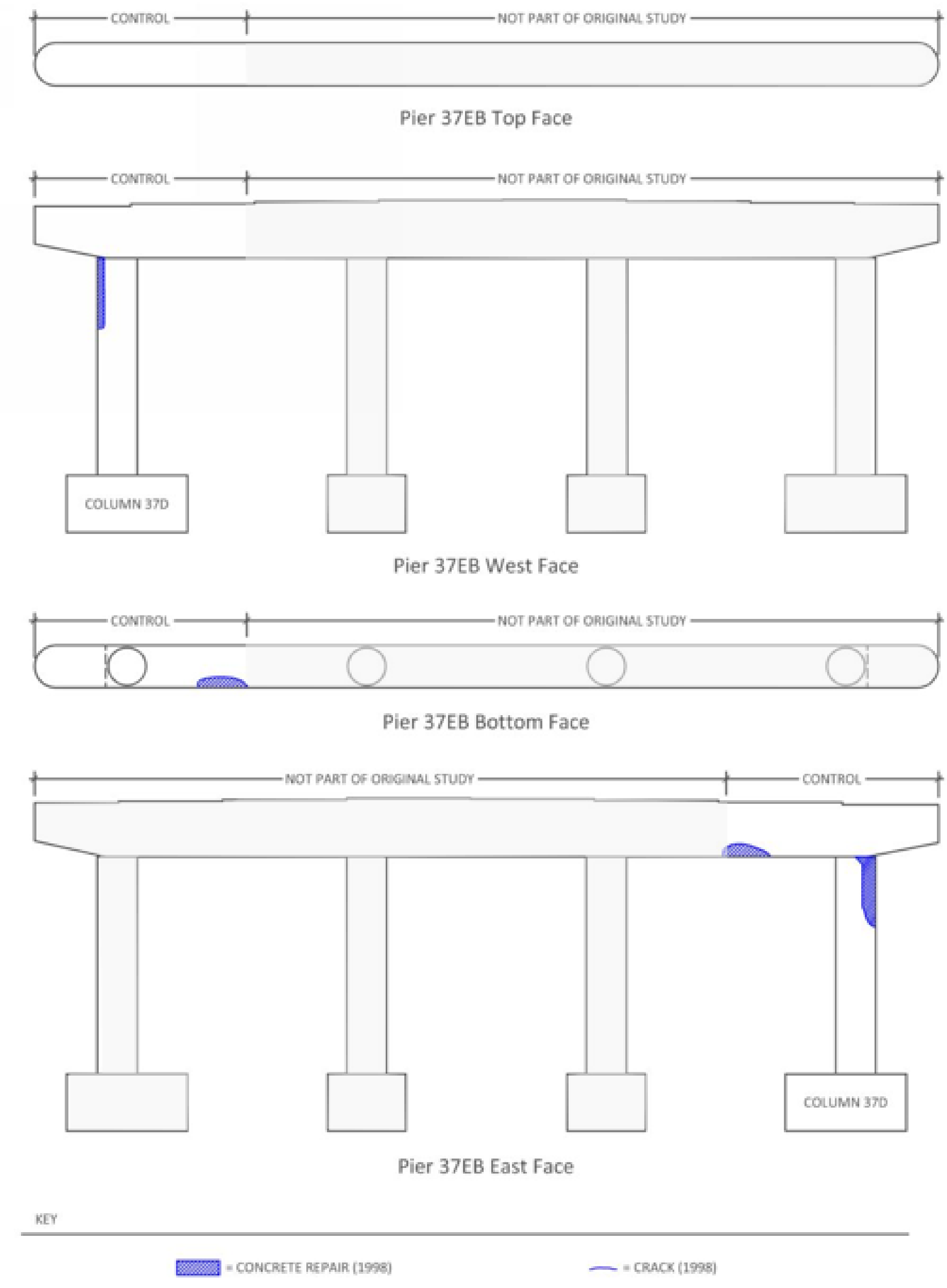


Figure 2.9 Pier 37EB - Areas of concrete distress and repair (blue shading) - 1998 study

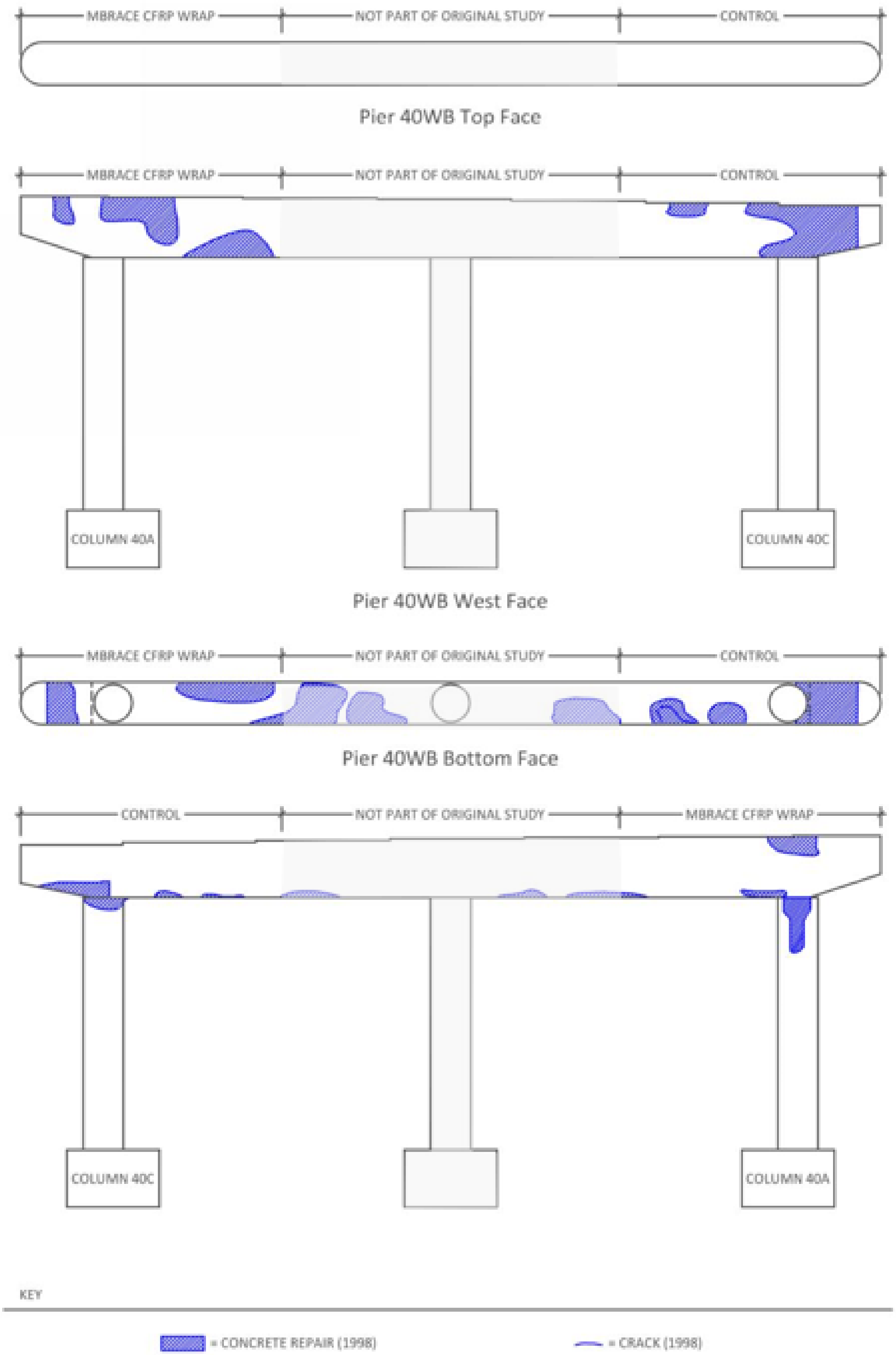


Figure 2.10 Pier 40WB - Areas of concrete distress and repair (blue shading) - 1998 study

Following the execution of concrete surface repairs, and before the various corrosion mitigation strategies were implemented, additional field investigation work was performed in October 1997 to supplement the condition survey findings. The work was intended to collect data which would establish baseline conditions related to corrosion activity and corrosion risk factors and facilitate future evaluations of performance during and following the completion of the study. The field investigation work included the drilling of concrete powder samples for laboratory testing of chloride concentration and performing half-cell potential testing.

Concrete powder samples were collected at a total of 68 different locations. Samples were typically obtained at four locations distributed along each elevation of each pier cap, and at three locations around the perimeter of each column. The sample locations were selected to be either immediately adjacent to areas where surface repairs had been recently performed, or some distance away from recent repair in areas of no visible corrosion-related distress. At each location, powder samples were drilled and collected into plastic bags at five different depth increments - 0-0.5 in. (0-1.25 cm), 0.5-1 in. (1.25-2.5 cm), 1-1.5 in. (2.5-3.75 cm), 1.5-2.5 in. (3.75-6.25 cm), and 2.5-3.5 in. (6.25-8.75 cm). The drill holes were then patched with a repair mortar. Each powder sample was tested by MnDOT to determine the acid-soluble chloride content in accordance with ASTM C1152 [12]. This set of samples is collectively referred to as *Pre-ECE Chloride Concentrations*. Refer to Appendix A for a complete tabulation of the pre-ECE chloride data.

Although not yet introduced, the tabulation presented in Appendix A also summarizes additional data sets associated with chloride sampling which was performed at the same locations after the ECE treatments were completed in 1998, and then again in 2018 after 20 years of service, as discussed further in Section 2.2.4 of this chapter and Chapter 3, respectively. As shown in Appendix A, almost 40 percent of the 336 pre-ECE powder samples which were tested possessed a chloride content in excess of 0.035 percent by weight of concrete (i.e., the assumed corrosion threshold). At most sampling locations, the chloride concentrations exceeded the threshold in the outer 1 inch nearest the surface of the elements, but decreased to levels below threshold at the depth of the reinforcing steel. Pier 34WB, however, exhibited more severe contamination including chloride levels in excess of the corrosion threshold at the depth of the reinforcing steel at several locations in the columns and pier cap. The elevated chloride levels were attributed to the positioning of this pier alongside a roadway, where increased exposure to splashing, plowed snow, or aspirated moisture laden with deicing salt would be expected.

In addition to chloride sampling, half-cell potential testing was performed in general accordance with ASTM C876 *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete* using a copper-copper sulfate electrode [13]. Readings were collected around the perimeter of the columns and across the side face surfaces of the pier caps on an approximate one-foot square grid. Readings were not collected on the top and bottom surfaces of the pier caps due to access difficulties. Upon completion, the half-cell potential data was utilized to generate contour plots which could be evaluated to identify areas of possible corrosion activity (numbers between -200 and -350 mV vs. CSE) or probable corrosion activity (numbers more negative than -350 mV vs. CSE). Refer to Figure 2.11.

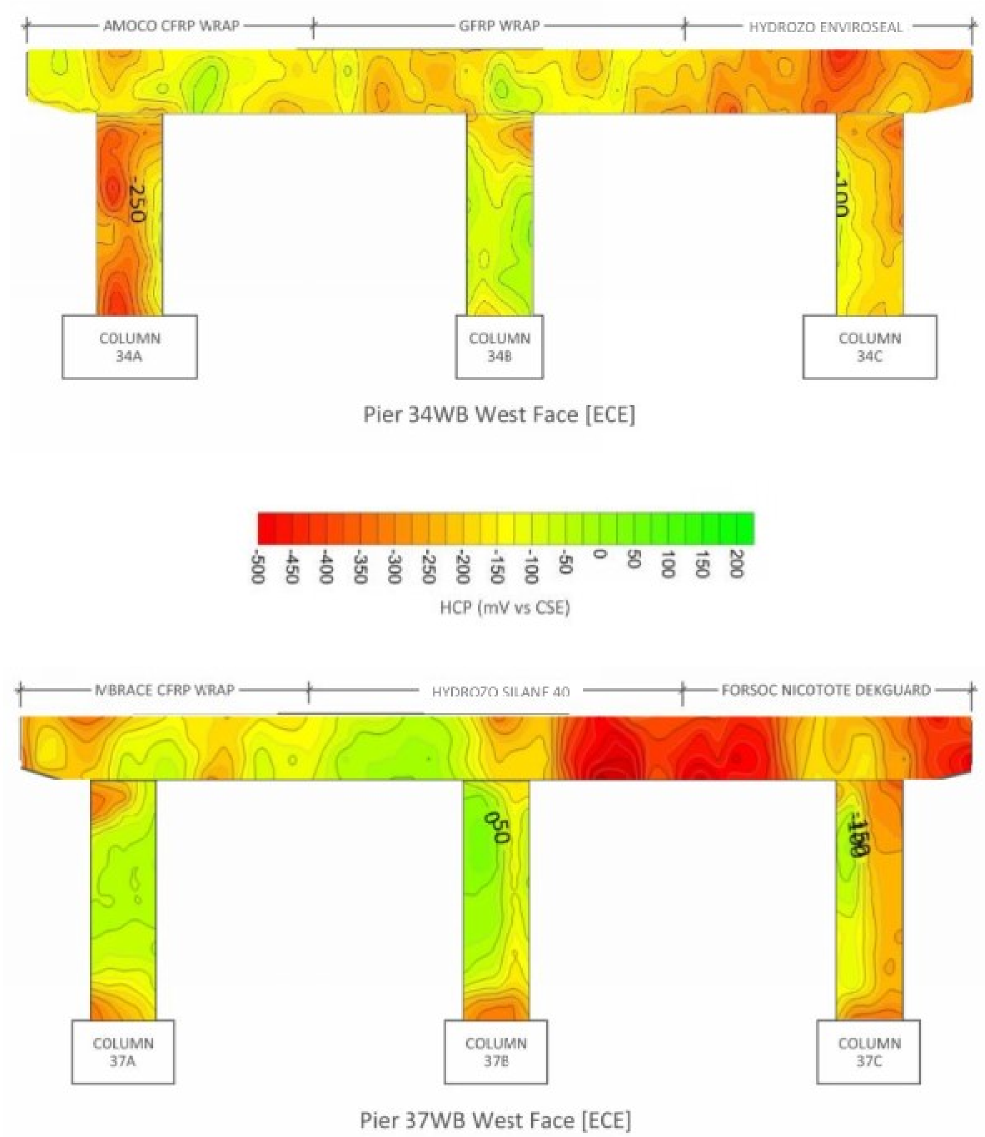


Figure 2.11 Pier 34WB and 37WB - Half-cell potential plots before ECE treatment

In summary, the 1998 condition survey confirmed possible or probable corrosion activity in several columns and almost all of the pier caps in and around areas of the 1997 concrete surface repairs.

2.2.3 ECE and FRP Installation

MnDOT contracted with Vector Construction, Ltd. (Vector) to implement the corrosion mitigation strategies which were to be evaluated as a part of the 1998 study. The work performed by Vector included the installation, operation and removal of an ECE treatment system, and the installation of six different surface protection systems. The surface protection systems were selected by MnDOT and each was intended to minimize or prevent water penetration into the concrete. The six systems included two different types of carbon fiber reinforced polymer (CFRP) wrap, one type of conventional fiberglass fabric (GFRP), and three different types of concrete sealer or coating systems:

- AMOCO CFRP Wrap - TYFO® Schedule 20 (AMOCO) fabric manufactured with THORNEL T250 (AMOCO) fiber and applied with TYFO® S Epoxy
- MBrace CFRP Wrap - MBrace Composite Strengthening System supplied by Master Builders Technology consisting of FORCA Tow unidirectional laminate sheets (CFRP) manufactured by Tonen Corporation and epoxy.
- GFRP Wrap - bidirectional fiberglass laminate fabric sheets distributed through Ashland Chemical Company, applied with the MBrace putty and epoxy system.
- Hydrozo Enviroseal, a water-based silane/siloxane blended water-repellant sealer
- Hydrozo Silane 40, a solvent-based, 40 percent silane penetrating sealer
- Fosroc Nictote Dekguard, a single component penetrating silane/siloxane primer and an aliphatic acrylate, solvent based protective coating.

The 1998 study team selected the locations where each of the various treatment installations would be performed by Vector. Each of the five piers included in the study were divided into sections consisting of one column and the tributary area of the pier cap located directly above. This exercise produced 12 sections which allowed for different treatment combinations to be installed and evaluated. The specification of where treatments were performed then considered several criteria:

- Piers receiving ECE treatment should be positioned near each other to simplify logistical considerations associated with the work, including the supply of electrical power.
- Interconnectivity of the reinforcing steel required that ECE treatments be performed on an entire pier, including the pier cap and all three columns below, and not just a section.
- Each type of surface protection would be installed on piers with ECE treatment and piers without ECE treatment.
- Different surface protection systems would be investigated together on each pier, but each system should be installed homogenously within each section (i.e., from the top surface of the pier cap to the base of the column at grade level).
- Several sections should be preserved as control specimens, receiving only patching and no ECE treatment or surface protection, to allow the performance of a traditional repair approach to be compared against the other corrosion mitigation strategies included in the study.

Table 2.1 and Figure 2.12 tabulate and illustrate, respectively, the locations which were selected for ECE treatment and the installation of the six different surface protection systems.

Table 2.1 Summary of Corrosion Mitigation Strategy Installation Locations

Location					
Pier					
34WB	North End	A	Y	FRP Wrap	AMOCO CFRP
	Middle	B		FRP Wrap	GFRP
	South End	C		Sealer	Hydrozo Enviroseal
34EB	North End	D	N	Control	
	Middle	E		FRP Wrap	GFRP
	South End	F		FRP Wrap	AMOCO CFRP
37WB	North End	A	Y	FRP Wrap	MBrace CFRP
	Middle	B		Sealer	Hydrozo Silane 40
	South End	C		Sealer	Fosroc Nicotote Dekguard
37EB	North End	D	N	Control	
40WB	North End	A	N	FRP Wrap	MBrace CFRP
	South End	C	N	Control	

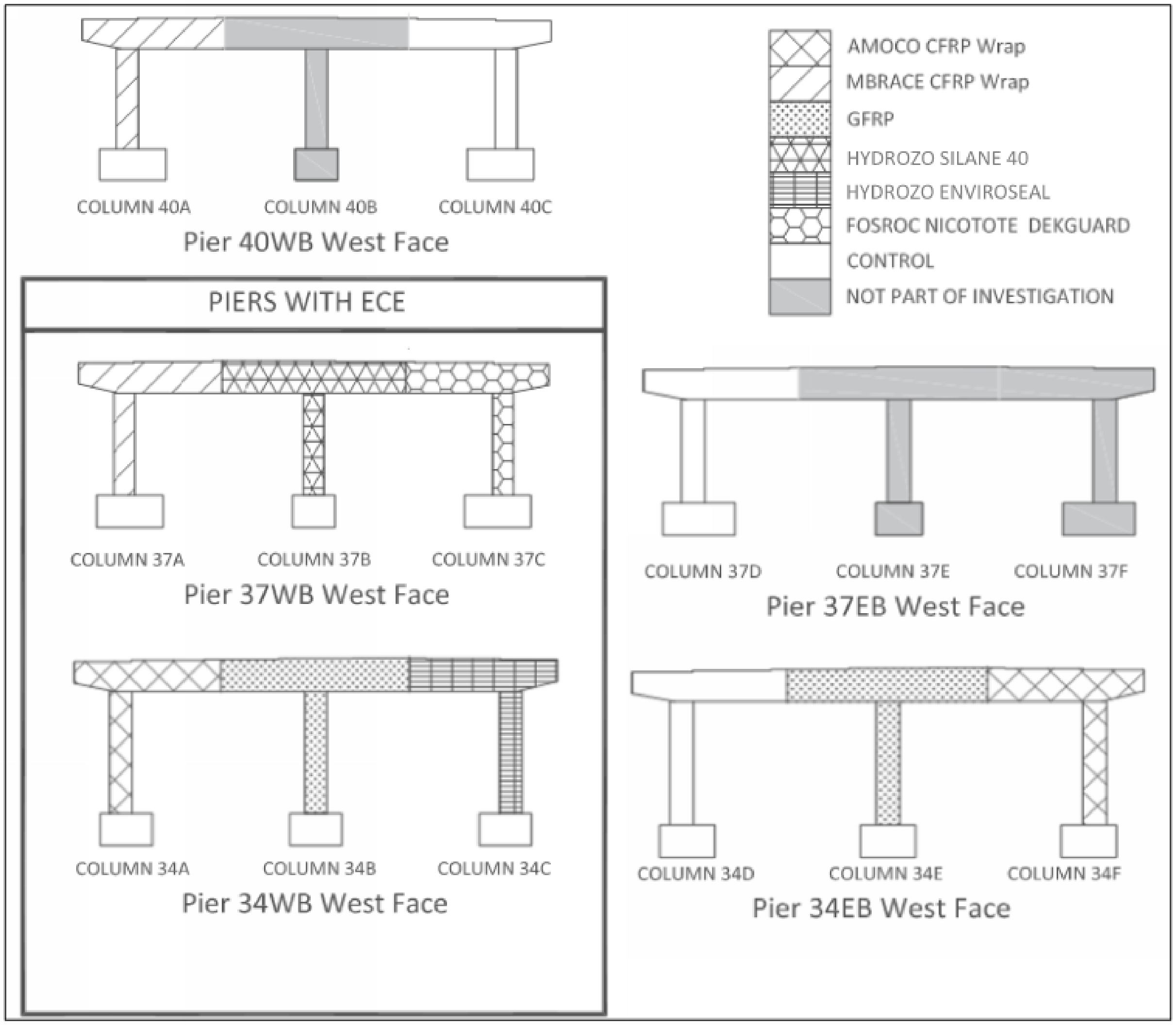


Figure 2.12 Summary of Corrosion Mitigation Strategy Installation Locations

As shown above, Pier 34WB and Pier 37WB were selected to receive ECE treatment. The ECE system installed by Vector was identified by the brand name NORCURE.™ Mild steel welded wire mesh (4 inches by 4 inches by 6 gauge) was fastened on the exterior of the piers to serve as the anode, and was electrically connected to the embedded reinforcement. A minimum of one connection was made between the mesh and reinforcement every 500 ft^2 (50 m^2) of surface area, and electrical continuity was checked at each location to verify connectivity of the reinforcement. The mesh was secured to thin wood battens which had been mounted on the surface of the concrete to provide a slight standoff. See Figure 2.13. Once installed, the mesh was clipped to create a number of equally sized sub-zones across the surface of the piers which were electrically isolated from each other. Each subzone covered approximately 100 to 150 ft^2 (10 to 15 m^2). Electrical leads ran from each subzone to a junction box, with the junction boxes then connected to one of four 40V DC rectifiers that powered the system. One rectifier was dedicated to each pier cap (34WB and 37WB), and one rectifier was dedicated to each group of columns (e.g., 34A, 34B and 34C). At each pier cap, the wiring was configured to negatively charge the reinforcement and positively charge the anode mesh once the treatment was activated [1].

Figure 2.13 Pier 37WB - ECE installation in progress, prior to application of cellulose fiber

Once electrical connections were established, the steel mesh was encapsulated in Fosroc NCT 2000 cellulose fiber. The cellulose fiber was applied wet and was intended to serve as a conductive media, or electrolyte, which facilitated the treatment and distributed the electrical current across the concrete surfaces. Refer to Figure 2.14. Drip hoses were connected to a central water supply and installed along the tops of the pier caps to continuously wet the cellulose during the treatment. Finally, the piers were wrapped in plastic sheeting to control moisture loss and evaporation.

Installation of the steel mesh anode and its electrical connections to the pier reinforcement was performed by Vector in September 1997. However, the onset of cold weather conditions delayed the completion of the installation work, and operation of the system, to the following spring. Given the delay in activation, supplemental chloride sampling work was performed in spring 1998 to determine if any significant changes had occurred over the winter months. Concrete powder samples were again collected, but the sampling was limited to spot verification at nine (of 72) representative locations and three (of 5) depth increments. Samples were drilled several feet away from the initial locations and were similarly tested in accordance with ASTM C1152-90 procedures [12]. No significant changes were identified in comparison to the Pre-ECE Chloride Concentrations at the locations sampled.

The ECE treatment system was energized on Pier 34WB and Pier 37WB beginning on April 16 and 17, 1998, respectively. The rectifiers were operated in a constant voltage mode, set with a maximum DC output of 40 V or 1A/m^2 (100 mA/ft^2) of surface area. Total current flow and voltage outputs, and subzone readings, were recorded with a data logger at each rectifier and were monitored throughout the treatment. Current flow is influenced by concrete resistivity, and concrete resistivity is influenced by the chloride content of the concrete. Decreases in current flow that were recorded while the system voltage remained steady were attributed to the concrete resistivity increasing, as the chloride content between the reinforcing steel and anode was being reduced. After 62 days of operation, the current flow no longer exhibited a steady decline and instead appeared to plateau. Vector collected concrete powder samples from spot locations on both piers for chloride content testing and verified that chloride reductions of approximately 50 percent had been achieved. This will be discussed further in Section 2.2.4. The ECE treatments were then stopped on June 16, 1998 and Vector removed the plastic sheeting, cellulose fiber, steel mesh, wood battens and electrical support systems [11].

Figure 2.14 Pier 34WB - ECE installation in progress, during application of cellulose fiber

Following ECE treatment, the surfaces of Piers 34WB and 37WB were wet, mottled with clumps of cellulose fiber, and rust-stained from corrosion of the anode mesh. A two-month waiting period was established to allow the concrete to dry out. Vector then performed grit-blasting to remove the cellulose remnants and rust-staining, and to prepare the concrete surfaces of these and other sections. Installation of the six surface protection systems was performed by Vector in late August 1998.

The installation procedures for the AMOCO CFRP, MBrace CFRP and GRFP products were generally similar and consisted of the application of a primer, one coat of epoxy resin, the FRP material, a saturant, and then a finish coating which provided protection against ultraviolet (UV) exposure. See Figure 2.15 for photographs of the applied CFRP. The MBrace and GFRP systems also included a putty layer which followed the primer and was intended to create a smoother surface upon which the resin and FRP material was applied. FRP material was installed with longitudinal fibers oriented circumferentially on the columns, and vertically on the pier caps. The material was pressed into the resin coat using hand pressure, and then a roller, to release entrapped air. At the pier caps, the FRP sheets were cut to fit around the precast girder bearing seats, stretched down the face of the pier cap, and then wrapped onto the bottom surface of the pier caps approximately 4 inches to create a drip edge. No FRP material was installed on the bottom surface between the two 4-inch drip edge extensions. Similarly, the FRP wrap terminated approximately 1 foot above grade near the base of the columns. These open areas were desired to avoid a full encapsulation which may promote trapping of moisture within the section. Adjacent FRP sheets were butted and sealant was installed at edge terminations of the FRP material at the tops of the pier caps and columns.

Figure 2.15 Pier 37WB - MBrace CFRP installation before application of UV coating.

During the FRP installation work, Vector prepared several mockup samples of each of the three FRP systems. The systems were applied using the same general procedure as the study elements except the materials were applied on both concrete test slabs and clear polycarbonate sheets. The samples were collected by the University of Minnesota to facilitate laboratory testing of water diffusion properties and peel strengths. The testing programs established that the MBrace and GFRP systems offered high peel strength and could be considered impermeable. In contrast, the AMOCO system exhibited a lower peel strength and allowed some water to migrate through certain samples at cracks or seams between woven fibers [11].

Vector also performed field bond strength testing of the FRP materials after the systems had cured. One test was performed at the column and at the pier cap within each section where FRP was installed. The test results reported by Vector are presented in Table 2.2. The average bond strength exceeded 450 psi, well above the desired target value of 200 psi, and no test failures occurred within the FRP systems.

The application of the three concrete sealer products - Hydrozo Enviroseal, Hydrozo Silane 40 and Fosroc Nicotote Dekguard - was performed by Vector following the completion of the FRP work. Each of the sealer systems was applied in general accordance with the manufacturer installation instructions. The limits of application matched the FRP wrap systems, including the 4-inch drip edge extensions on the bottom pier surfaces and terminating the treatments approximately 1 foot above grade at the base of the columns.

Table 2.2 Results of FRP Bond Strength Testing Performed by Vector

Location	Element	FRP Wrap	Pull Load (lbs)	Dolly Area (sq. in.)	Bond Strength (psi)	Failure Location
Pier 40WB	Column A	MBrace CFRP	1300	2.25	578	Dolly paste
	Pier Cap		1100		489	Dolly paste
Pier 37WB	Column A		1300		578	Dolly paste
	Pier Cap		1300		578	Dolly paste
Pier 34WB	Column B	GFRP	1220		542	Concrete
	Pier Cap		1110		493	Concrete
Pier 34EB	Column E		1260		560	Concrete
	Pier Cap		1130		502	Concrete
Pier 34WB	Column A	AMOCO CFRP	900		400	Concrete
	Pier Cap		950		422	Concrete
Pier 34EB	Column F		1000		444	Concrete
	Pier Cap		550		244	Dolly paste

2.2.4 Post-ECE Testing

Post-ECE chloride sampling work was performed in August 1998 during the two-month drying period which followed the completion of ECE treatments. This sampling was performed prior to surface protection system installations. The sampling protocol matched the pre-ECE sampling protocol performed in October 1997, before ECE treatment. Concrete powder samples were again collected at 5 different depth increments - 0-0.5 in. (0-1.25 cm), 0.5-1 in. (1.25-2.5 cm), 1-1.5 in. (2.5-3.75 cm), 1.5-2.5 in. (3.75-6.25 cm), and 2.5-3.5 in. (6.25-8.75 cm) - from a total of 72 locations on the pier caps and columns which were included in the study. The locations of the August 1998 samples were approximately two inches away from the October 1997 samples to avoid drilling into the repair mortar which had been used to fill the initial drill holes. Each powder sample was tested to determine the acid-soluble chloride content in accordance with ASTM C1152 [12]. This set of samples was collectively referred to as *Post-ECE Chloride Concentrations*. Refer to Appendix A for a complete tabulation of the post-ECE chloride data and comparison to the pre-ECE chloride levels.

Between October 1997 and August 1998, no significant chloride level increases or decreases were identified in sites not receiving ECE treatment [11]. Slight differences in chloride content were consistently measured but were judged to be representative of the variability inherent with the testing procedure. Identical results would not be expected with powder samples drilled from adjacent locations within an inhomogeneous material (concrete) that has been subjected to non-uniform chloride exposure over the course of 20 years. The total percentage of samples with chloride concentrations in excess of the corrosion threshold was 13 percent (26 of 195), and only one location possessed a chloride content over the corrosion threshold at the depth of reinforcing steel (40C-1).

Significant reductions in chloride content were observed at Pier 34WB and Pier 37WB which received ECE treatment [11]. Within Appendix A, changes in chloride contents between the samples collected before and after ECE treatment are calculated and highlighted by red (increase) or green (decrease) backgrounds. Plots of pre-ECE vs. post-ECE chloride data obtained at Pier 34WB and Pier 37WB are also included as Figure 2.16 and Figure 2.17. Of note:

- At Pier 34WB, the number of samples which contained chloride concentrations in excess of the corrosion threshold was reduced from 54 percent (35 of 65) to 8 percent (7 of 85) as a result of ECE treatment. The number of sample locations with chloride concentrations at the depth of the reinforcing steel in excess of the corrosion threshold was reduced from three to zero, although two locations (34C-3 and 34N-W2) were only slightly below (0.033 percent by mass of concrete).
- At Pier 37WB, the initial chloride levels were much lower than Pier 34WB. Nonetheless, the number of samples which contained chloride concentrations in excess of the corrosion threshold was reduced from 16 percent (14 of 85) to 2 percent (2 of 85) as a result of ECE treatment. The number of sample locations with chloride concentrations at the depth of the reinforcing steel in excess of the corrosion threshold was reduced from three to zero.
- Considering all samples, the average reduction in chloride content which occurred at each sample depth as a result of ECE treatment was approximately 45 percent.

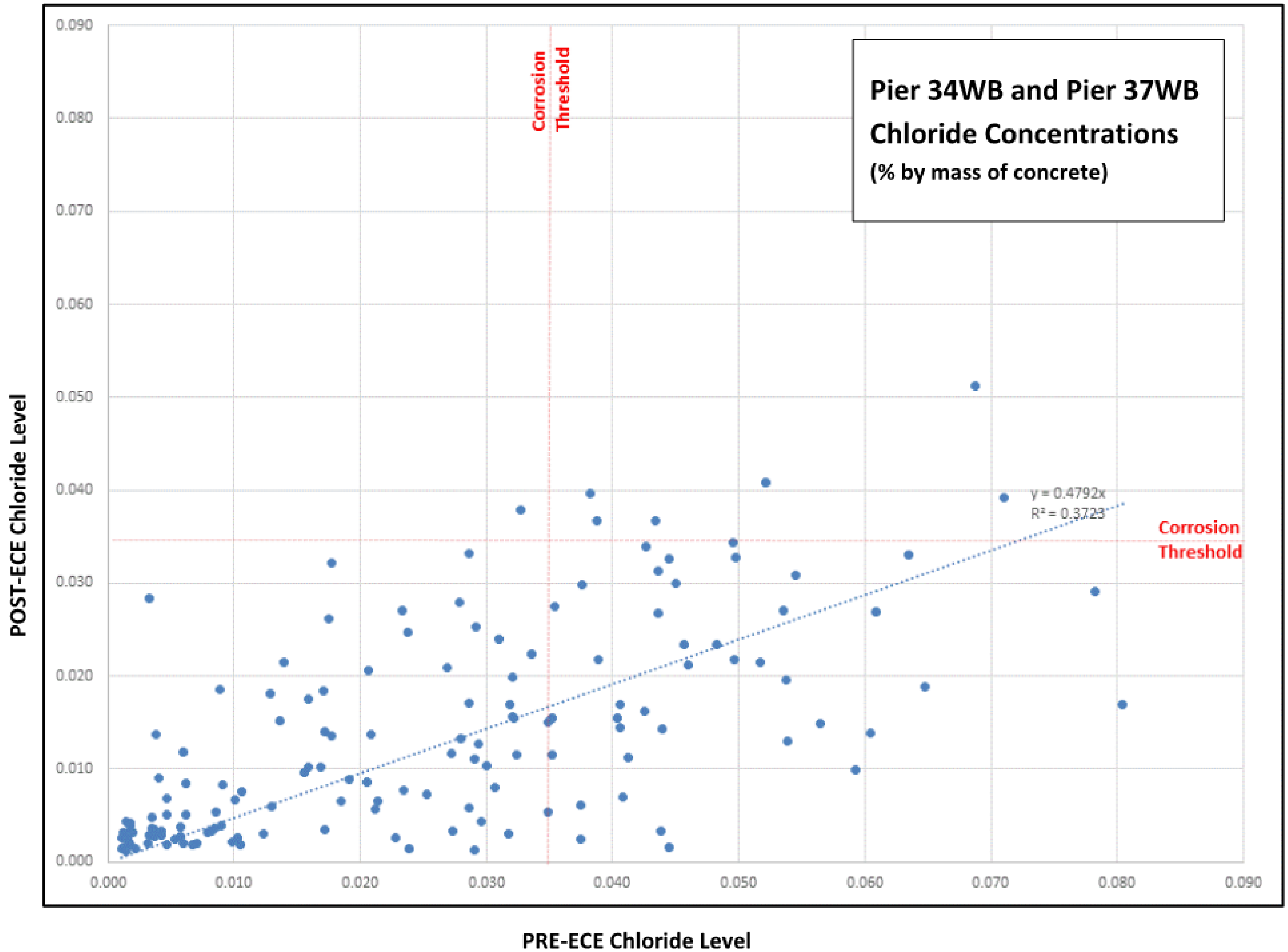


Figure 2.16 Scatter plot of pre-ECE (x axis) vs. post-ECE (y-axis) chloride levels

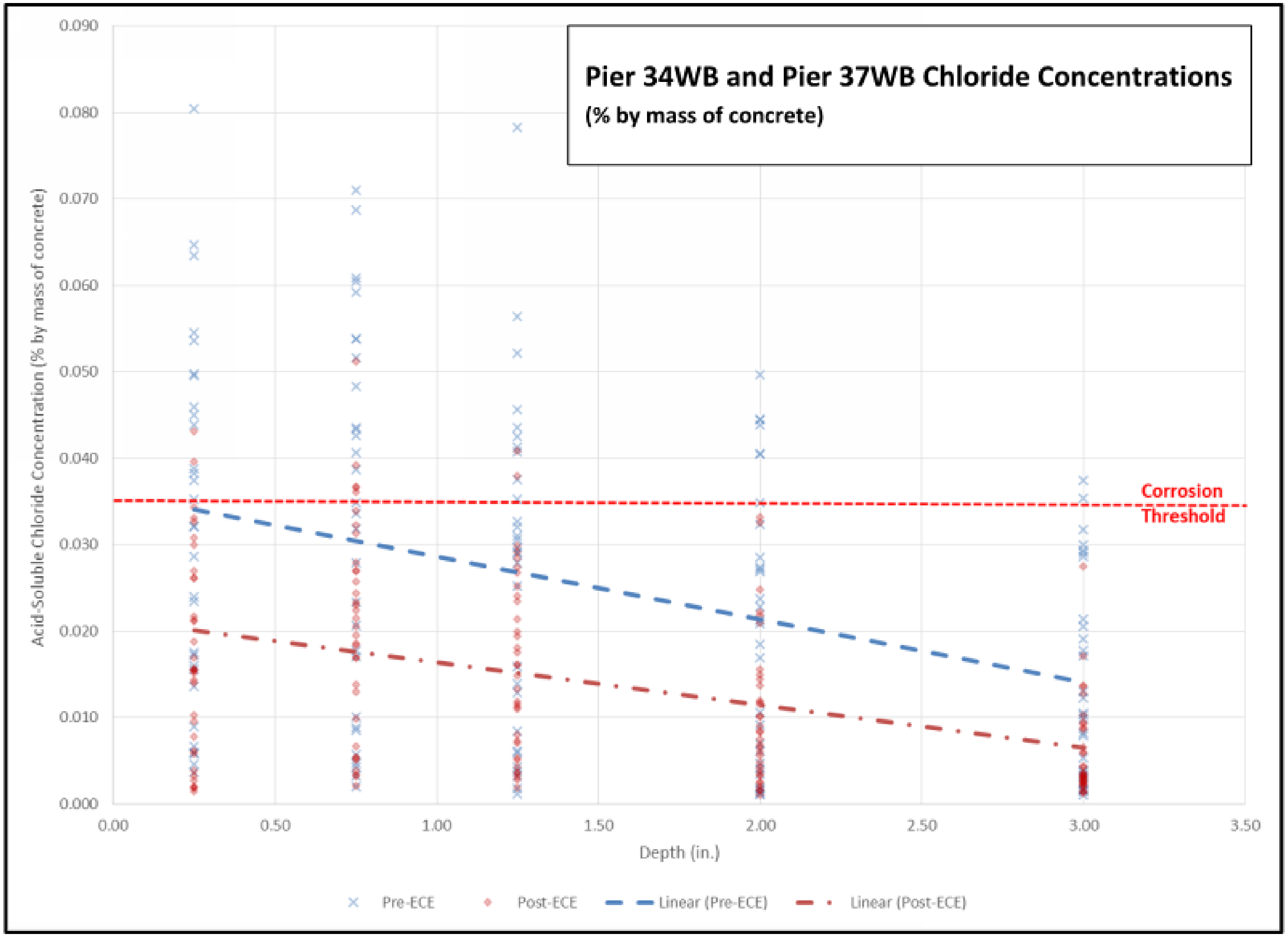


Figure 2.17 Comparison of pre-ECE vs. post-ECE chloride levels by horizon

In general, the greatest reductions in chloride content which occurred as a result of ECE treatment were sample depths within 1 inch of the concrete surface which had exhibited significant contamination [11]. These results were expected because the greater concentration of chlorides, and the proximity to the sacrificial anode, would facilitate a large driving force for extraction. In contrast, less significant reductions, or even some slight increases, were observed at locations of low chloride concentration or deeper chloride penetration (i.e., behind the reinforcing steel) where a much smaller driving force would be generated during treatment. The effectiveness of the ECE treatment also varied somewhat by location, even for sample locations in close proximity to each other (e.g., 34A-2 vs. 34A-3 and 34C-2 vs. 34C-3) or at similar depths (e.g., 37N-E1 vs. 37N-W2). The variability was presumed to be influenced by the proximity of reinforcing steel to the sample location. ECE would be most effective at removing chloride ions which were near reinforcing steel, and less effective at removing chlorides in regions further from reinforcing steel, since the current path runs through the reinforcing steel.

In addition to the chloride sampling, follow-up half-cell potential testing was performed by Vector on the sections of Pier 34WB and Pier 37WB. The testing was limited to the sections which did not receive FRP wrap because there was access to the concrete surface. This testing was performed on November 26, 1998 and was intended to verify re-passivation of the reinforcing steel had occurred as a result of ECE treatment. All but 3 of the 215 half-cell potential readings collected by Vector were more positive than -200 mV, representative of a low risk (less than 10 percent probability) that active corrosion was occurring [11].

In summary, field and laboratory testing demonstrated that ECE treatments had significantly reduced the level of chloride contamination and the corrosion potentials which existed in Piers 34WB and 37WB. However, nine locations in Pier 34WB and two locations in Pier 37WB still possessed chloride concentrations in excess of the corrosion threshold following treatment. These elevated chloride levels were all located within 1-1/2 inches of the concrete surface. Although available research indicated that it may remain for several years, the longevity of the increased passivation of the reinforcing steel was unknown [3,4]. The combination of these considerations created some concern that, over time, residual chlorides may migrate back to the depth of the reinforcing steel and re-initiate corrosion activity. Continued monitoring was recommended to evaluate long-term effectiveness.

2.2.5 Instrumentation

As a component of the 1998 study, the University of Minnesota installed instrumentation in all twelve columns and five pier caps to facilitate periodic monitoring of corrosion conditions. Instrumentation was desired to identify latent corrosion activity, particularly at sections where FRP wraps had been installed and the surface of the concrete could no longer be visually inspected or accessed for half-cell potential testing, and the effectiveness of sounding techniques was questionable. While the impermeable nature of the FRP wraps was expected to prevent the ingress of new moisture and chlorides, concern remained that existing moisture and chlorides could become trapped and may create a contained corrosive environment. Although conventional concrete assessment techniques remained viable at sections without FRP, instrumentation was placed at all elements included in the study to allow conditions to be monitored at each location similarly.

The instrumentation which was installed included embeddable half-cell electrodes, relative humidity sleeves, and resistivity-based corrosion probes. The embeddable half-cell electrodes consisted of silver/silver-chloride (Ag/AgCl) electrodes which were manufactured by ELGAARD®. The relative humidity sleeves were plastic plugs with caps which could be monitored with a Protimeter® Concrete Master II humidity sensing probe. The resistivity-based corrosion probes were developed by the University of Minnesota. See Figures 2.18 and 2.19.

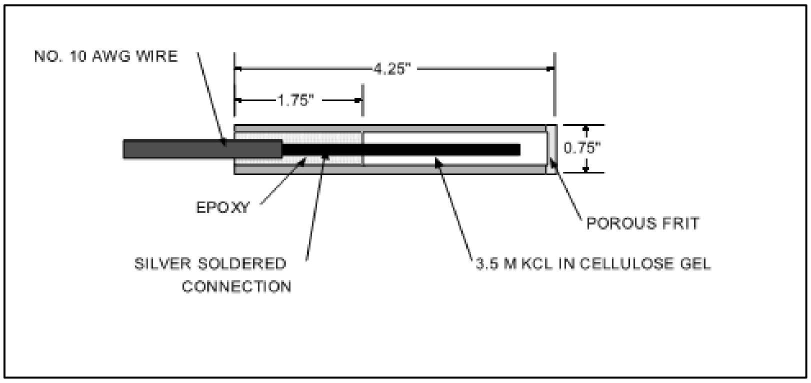


Figure 2.18 Schematic of embeddable Ag/Ag-Cl electrodes [11].

The resistivity-based corrosion probes consisted of a 1 inch (25.4 mm) long loop of iron wire measuring 0.02 in. (0.5 mm) or 0.01 in. (0.25 mm) in diameter which were identified as “large” and “small” probe types, respectively. For either size, the ends of the iron wire were soldered to lead wire and then covered with silicone sealant. The probes were designed as on/off indicators of corrosion activity which could be monitored with a multimeter. The resistance of the wire, initially less than 2 Ohms, would increase to infinity once sufficient corrosion had occurred to consume the wire cross-section. When exposed to the same conditions, the small probes were demonstrated to fail approximately twice as fast as the large probes, as would be expected given half the cross-sectional area.

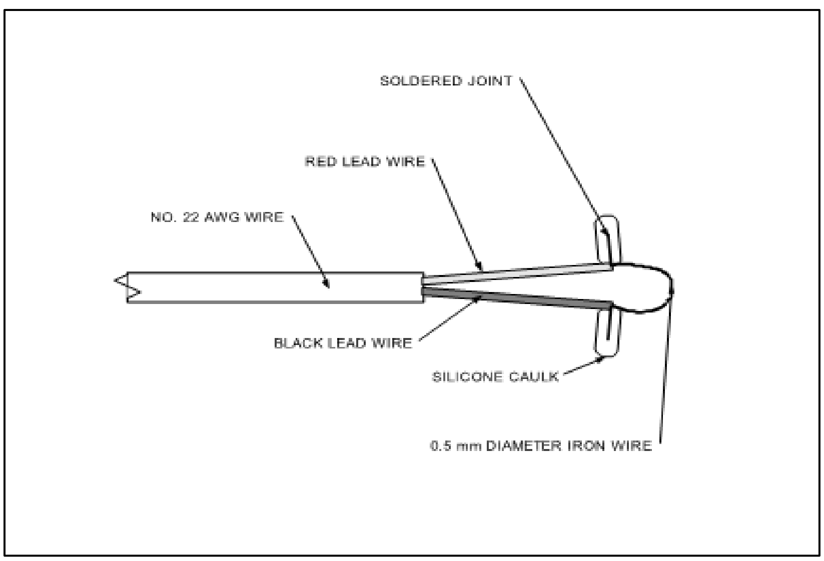


Figure 2.19 Schematic of a large resistivity-based corrosion probe [11].

Instrumentation was installed in December 1998 at 50 of the 72 locations where chloride samples had been obtained after the ECE treatment phase. At each instrument location, one embeddable half-cell and two resistivity probes were installed. One small and one large resistivity probe were installed at 30 locations, and two small probes were installed at the other 20 locations. Humidity sleeves were also installed but were positioned slightly away from the other probes to avoid congestion.

All of the instruments were placed in holes drilled into the concrete surfaces, including through the FRP wraps or concrete sealers which had been applied by Vector as necessary. The embeddable half-cells and resistivity probes were set near the depth of the reinforcing steel (approximately 2 inches) and then the holes were filled with a standard cement grout and covered with silicone sealant at the concrete surface. Refer to an example location shown in Figure 2.20.

Figure 2.20 View of the instrumentation installed at Column 34F-3.

Once installed, lead wires from the groups of instruments were labelled, bundled and run through plastic conduit to locked metal cabinets which were mounted on the side faces of the pier caps. The metal cabinets served as data collection hubs.

2.3 POST-1998 SERVICE HISTORY

2.3.1 Inspection Findings and Repairs

Following implementation of the 1998 study, Bridge Inspection Reports continued to be prepared by MnDOT personnel to summarize the conditions that were observed and documented by bridge inspection teams, including element ratings, data and distress. Almost all of the strip seal joint in the entire bridge deck were rated as Condition State 1, defined as “little to no deterioration” and “no leakage," throughout the time period between 1998 and 2017. Below the bridge deck, no new distress or any significant changes or concerns were reported regarding the condition of the pier caps and columns which were included in the 1998 study. Instead, for these elements, the reports included a standard note "*[1997] Experimental Chloride Extraction Project on Piers #34, #37 & #40*." However, at some point around 2007, fire damage occurred to the FRP systems and instrumentation which had been installed at the east face of Pier 34WB above Column 34B. The fire was believed to have been caused by vagrants. The fire resulted in debonding and partial melting of the GFRP wrap and melting of the lead wires for the embedded half-cells and resistivity probes installed in that area.

Local substructure patching repairs and deck expansion joint replacement work was performed by MnDOT in 2004 and 2008. The work in 2008 included localized patching, joint replacement and reglanding of existing joints. At Piers 34 and 37, the expansion joint was only scheduled for reglanding while Pier 40 was scheduled for replacement. Reglanded joints do not offer the durability or performance of a fully reglanded joint [15]. MnDOT reported that a change order occurred, and the expansion joint was fully replaced at Piers 34, 37 and 40 on both the westbound and eastbound structures.

Away from the study area, the bridge deck was removed and replaced at the west end of the bridge from Spans 1 to 12 of the westbound structure and Spans 1 to 18 of the eastbound structure in 2007/2008. Most recently, the bridge deck wear course was milled and replaced at the east end of the bridge in 2017.

2.3.2 Review of Instrumentation Data (1999 to 2007)

Periodic site visits were performed by University of Minnesota personnel in 1999, and then MnDOT personnel from approximately 1999 to 2009, to manually collect data from the installed instrumentation. The site visits were generally conducted every 2 to 3 months and the data which was collected reflected single measurements at that moment in time [14]. Because the embedded half-cells were Ag/Ag-Cl electrodes, collected potential readings required correction by -96mV to enable direct comparison to potentials which had been corrected before and after ECE treatment using a copper sulfate electrode [13]. All collected readings were compiled in a spreadsheet, and the combined data from each instrument location was plotted.

Figures 2.21 and 2.22 present example plots of collected data. Tables 2.3 and 2.4 summarize the complete instrumentation results separated by the structures which received ECE treatment, and those that did not, respectively. As discussed previously, abrupt and significant spikes in resistance were assumed to be representative of corrosion failure of the small or large resistivity probes. Embedded half-cell readings which were more negative than -350 mV were considered evidence of probable corrosion activity. For reference, the highest chloride concentration which was measured in the post-ECE sampling, and the associated depth of that sample, is identified for each instrument location with Table 2.3 and 2.4. Refer to Appendix B for the collected instrumentation data.

Data indicative of probable corrosion activity was collected during the monitoring period (1999 to 2007) from either the resistivity probes or the embedded half-cells, or both, at 44 percent of the 50 locations where instruments were installed. However, at many locations, inconsistencies were apparent in the data collected by the different instrument types. These inconsistencies included corrosion failure of large resistivity probes before small resistivity probes (e.g., 34EB-W2), corrosion failures of both resistivity probes despite passive half-cell potential readings (e.g., 34C-1), and highly negative half-cell potentials with no corrosion failure of either of the installed resistivity probes (e.g., 37C-1).

Additional analysis of the collected instrumentation data is presented in Chapter 4, following presentation of the results of the field investigation work and laboratory analyses which were performed as a component of the 2018 study. As will be discussed, the instrumentation data generally lacked consistency with the findings of the 2018 study.

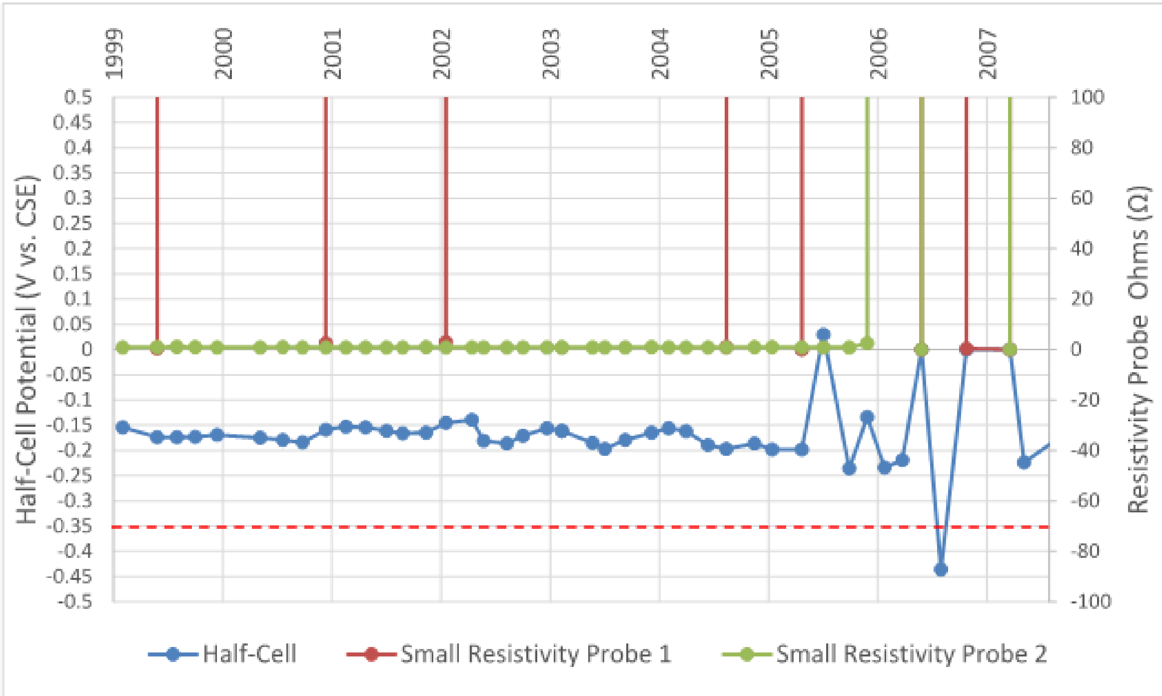


Figure 2.21 Plot of instrumentation data - Column 34C-3

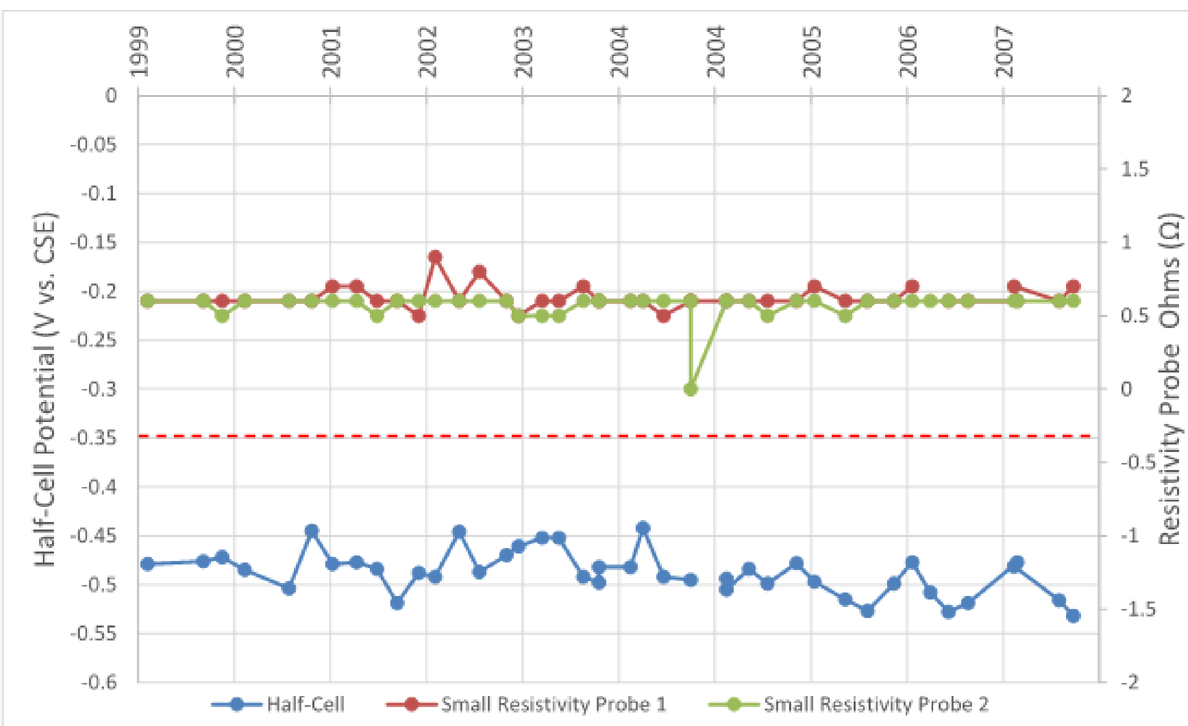


Figure 2.22 Plot of instrumentation data - Column 37D-3

Table 2.3 Summary of Instrumentation Data for Piers 34WB and 37WB (ECE)

Structure	Pier 34WB													Pier 37WB									
Element	34A			34B			34C			Pier Cap				37A		37B		37C		Pier Cap			
Sensor Location	1	2	3	1	2	3	1	2	3	W1	W2	E1	E2	1	2	1	2	1	2	W1	W2	E1	E2
Highest CL- Content (POST-ECE August 1998)	0.027	0.023	0.039	0.013	0.013	0.023	0.035	0.010	0.051	0.035	0.038	0.022	0.030	0.028	0.019	0.004	0.027	0.032	0.003	0.022	0.040	0.031	0.028
Depth Range (in.)	0-0.5	0.5-1	0.5-1	0-0.5	0.5-1	1-1.5	0-0.5	0.5-1	0.5-1	0.5-1	1-1.5	0-0.5	0-0.5	1-1.5	0.5-1	1.5-2.5	0.5-1	0.5-1	All	0.5-1	0-0.5	0.5-1	0.5-1
Half-Cell																							
Probable Corrosion Activity?	N	Y	N	N	N	N	N	N	Y	N	N	N	N	N	N	N	N	Y	N	N	N	N	N
Year		2005							2007									1999					
Small Resistivity Probe 1																							
Failure?	Y	Y	Y	N	N	N	Y	N	Y	N	N	N	N	Y	N	N	N	N	N	N	Y	N	N
Year	2005	2003	1999				1999		1999					2007							1999		
Small Resistivity Probe 2																							
Failure?	Y		N		N		Y	N	Y	N	Y			Y	Y	N	N	N	N				
Year	2001						2006		2006		2007			1999	2007								
Large Resistivity Probe																							
Failure?		Y		N		N						N	N							N	N	N	N
Year		2004																					

Table 2.4 Summary of Instrumentation Data for Piers 34EB, 37EB and 40WB (Non-ECE)

Structure	Pier 34EB												Pier 37EB					Pier 40WB									
Element	34D			34E		34F			Pier Cap				37D			Pier Cap		40A			40C	Pier Cap					
Sensor Location	1	2	3	1	2	1	2	3	W1	W2	E1	E2	1	2	3	W1	E1	1	2	3	1	W1	W2	W3	E1	E2	E3
Highest CL- Content (August 1998)	0.035	0.010	0.035	0.008	0.045	0.025	0.056	0.053	0.031	0.045	0.035	0.038	0.009	0.051	0.029	0.027	0.024	0.028	0.030	0.039	0.048	0.043	0.037	0.024	0.044	0.033	0.035
Depth Range (in.)	0.5-1	0-0.5	0.5-1	0-0.5	0-0.5	0-0.5	0-0.5	0-0.5	0-0.5	0.5-1	0.5-1	0-0.5	0-0.5	0.5-1	0.5-1	0-0.5	0-0.5	0-0.5	0-0.5	0-0.5	1.5-2.5	0.5-1	0.5-1	0.5-1	0.5-1	1-1.5	0.5-1
Half-Cell																											
Probable Corrosion Activity?	N	N	N	N	N	N	N	N	N	N	N	N	Y	Y	Y	N	N	N	N	N	N	Y	Y	N	N	N	Y
Year													1999	1999	1999							1999	2003				2007
Small Resistivity Probe 1																											
Failure?	N	N	N	Y	N	Y	Y	Y	N	Y	N	Y	N	N	N	N	N	N	N	Y	N	N	Y	N	N	N	N
Year				2002		2005	2005	2005		2009		2004								1999			2005				
Small Resistivity Probe 2																											
Failure?	N	N	N	N										N	N												
Year																											
Large Resistivity Probe																											
Failure?					N	Y	Y	Y	N	Y	N	Y	N			N	N	N	N	N	N	N	Y	N	N	N	N
Year						2004	2006	2005		1999		2004											1999				

CHAPTER 3: 2018 STUDY

3.1 OVERVIEW

MnDOT retained Collins Engineers, Inc. in 2017 to perform a special inspection of Bridge No. 27831 and its four adjoined ramp bridges. As a component of that project, the authors were retained to perform a targeted corrosion investigation of the pier cap and column elements which had been included in the 1998 study. The corrosion investigation is termed herein as the 2018 study. The objective of the 2018 study was to evaluate the performance of the corrosion mitigation strategies which had been implemented at Pier 34WB, Pier 34EB, Pier 37WB, Pier 37EB and Pier 40WB after 20 years of service.

The 2018 study included a combination of field work and laboratory analysis tasks which were generally similar to the tasks which had been performed in the 1998 study. Consistency between studies was desired to facilitate direct comparisons between current conditions and those that existed at the onset and completion of the 1998 study, including locations of distress, corrosion activity and chloride concentrations. Inspections were performed by Collins at all twelve column and pier cap sections, with the results provided to the authors for review and verification. The authors performed non-destructive evaluation (NDE) work including concrete cover surveys, half-cell potential testing, FRP bond testing, and concrete resistivity measurement. The authors also selected locations for core samples to be extracted by MnDOT personnel for laboratory testing of carbonation and acid-soluble chloride content. This chapter describes the procedures and presents the findings of these tasks.

3.2 FIELD INVESTIGATION

3.2.1 Visual Inspection and Delamination Survey

Collins Engineers completed 100 percent visual inspection and hammer sounding surveys of the substructure of Bridge No. 27831 in fall 2017, including the twelve columns and five pier caps which were included in the 1998 study. Visual inspections identified areas of obvious cracking, spalling or exposed reinforcing steel, and hammer sounding surveys located hollow sounding areas which are typically indicative of delaminations associated with corrosion of reinforcement. Piers 34WB, Pier 34EB, Pier 37WB and Pier 40WB included areas of FRP wrap installation. The FRP-covered areas were visually inspected and hammer sounded to attempt to identify underlying concrete distress. However, the FRP deadened the hammer impact and muffled the resultant sounds, rendering the sounding of the covered areas somewhat inconclusive. The locations and limits of all distress conditions identified by Collins were physically marked on the structure surface using white paint, photographed, and documented on field sheets by Collins Engineers. An example of a typical distress marking is shown in Figure 3.1. The authors reviewed the provided documentation and verified the distress locations during field work. Some additional areas of delaminations were identified in areas which received FRP wrap, and at the end of the pier cap of Pier 37WB which received Fosroc Nicotote Dekguard.

Figure 3.1 Pier 34WB - Example of distress identified on the west face of the pier cap

Refer to Figures 3.2 through 3.21 for a compilation of paired elevation views and wire frame illustrations of the distress conditions identified at each pier. Locations of 1998 repairs and 2018 distress are both depicted in the illustrations using blue and red shading, respectively. The surface treatments which were installed as part of the 1998 study are identified above the pier cap illustrations, but apply to the entire section including the columns directly below.

Concrete distress redeveloped at some locations of 1998 surface repairs, including at least one area on each pier cap. However, the total area of distress in 2018 was 45 percent less than the total area of distress repair which was performed in 1998, including very little damage at the columns. Of note:

- Limited concrete distress was observed at Pier 34WB, which had received ECE treatment. The distress was isolated to the south end of the pier cap and Column C, where a concrete sealer was applied. Water staining and wetness was present on the pier cap in this area. Contrary to the findings of the visual inspections performed through 2017 which indicated the joints were in good condition, daylight was visible through the strip seal joint above this pier. See Figure 3.22.
- Only one small delamination was present at Pier 34EB, within the control (untreated) region at the north end of the pier cap. The concrete surface was discolored, consistent with water staining, near the distress. Just as found on Pier 34WB, daylight was visible through the strip seal deck joint directly above.
- Pier 37WB received ECE treatment but exhibited comparable levels of distress to that observed at the time of the 1998 study. All of the distress was present in the middle and south end regions of the pier cap and Column 37C, where concrete sealers had been applied.
- Delaminations were detected through the FRP wrap at three locations at Pier 40WB. Wetness and water staining was apparent on the concrete and FRP surfaces near these areas. Daylight was visible through the end of the deck joint directly above the north end of the pier cap, and a deck drain and downspout were located at the south end. See Figure 3.23.
- The majority of GFRP and AMOCO CFRP materials installed on the east face of Pier 34WB were blistered, discolored and debonded as a result of past fire damage. See Figure 3.24.

Figure 3.2 Pier 34WB - West Face

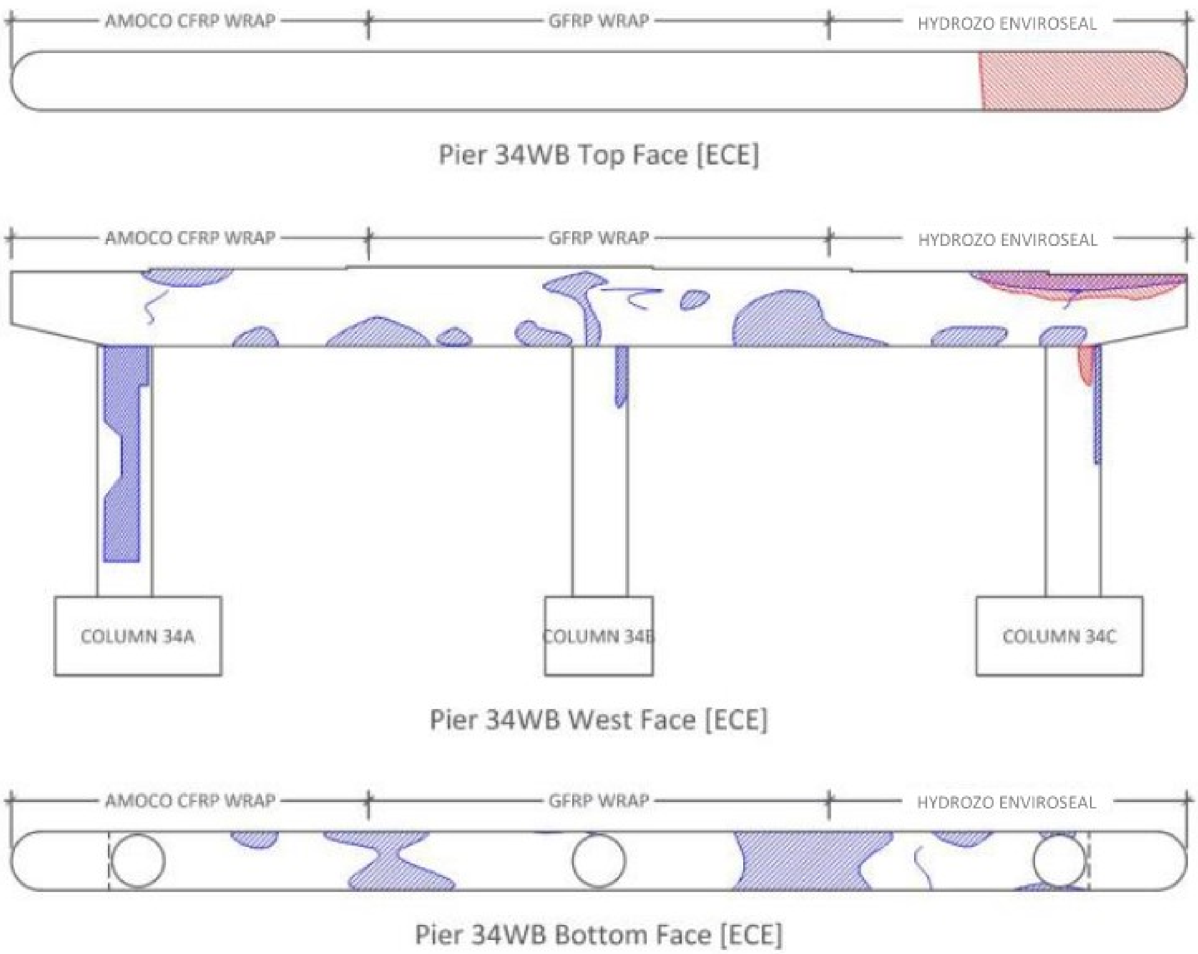


Figure 3.3 Pier 34WB - West Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.4 Pier 34WB - East Face

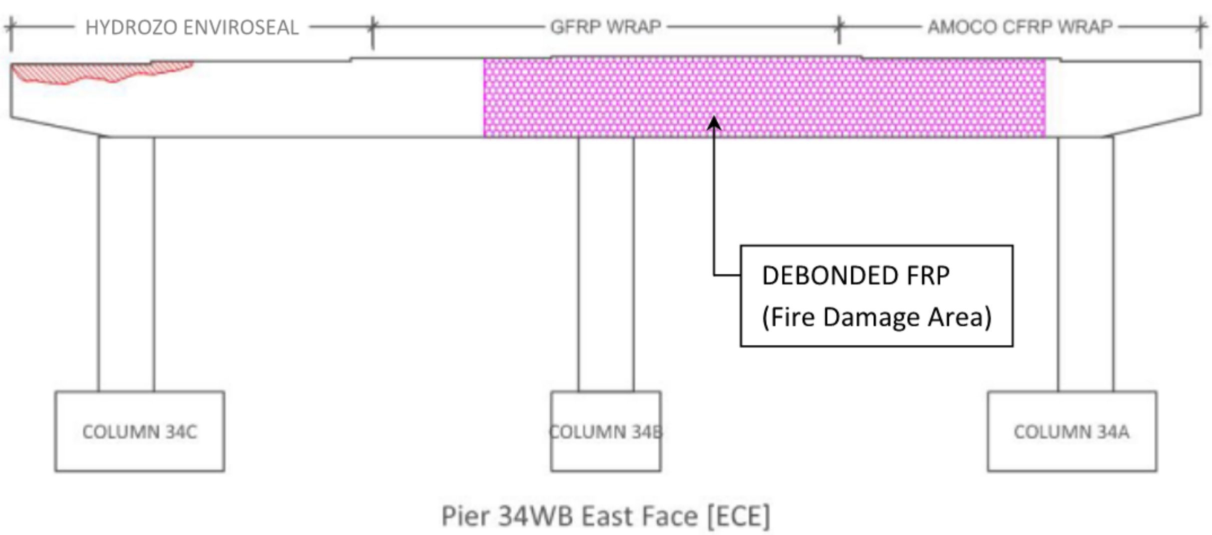


Figure 3.5 Pier 34WB - East Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.6 Pier 34EB - West Face

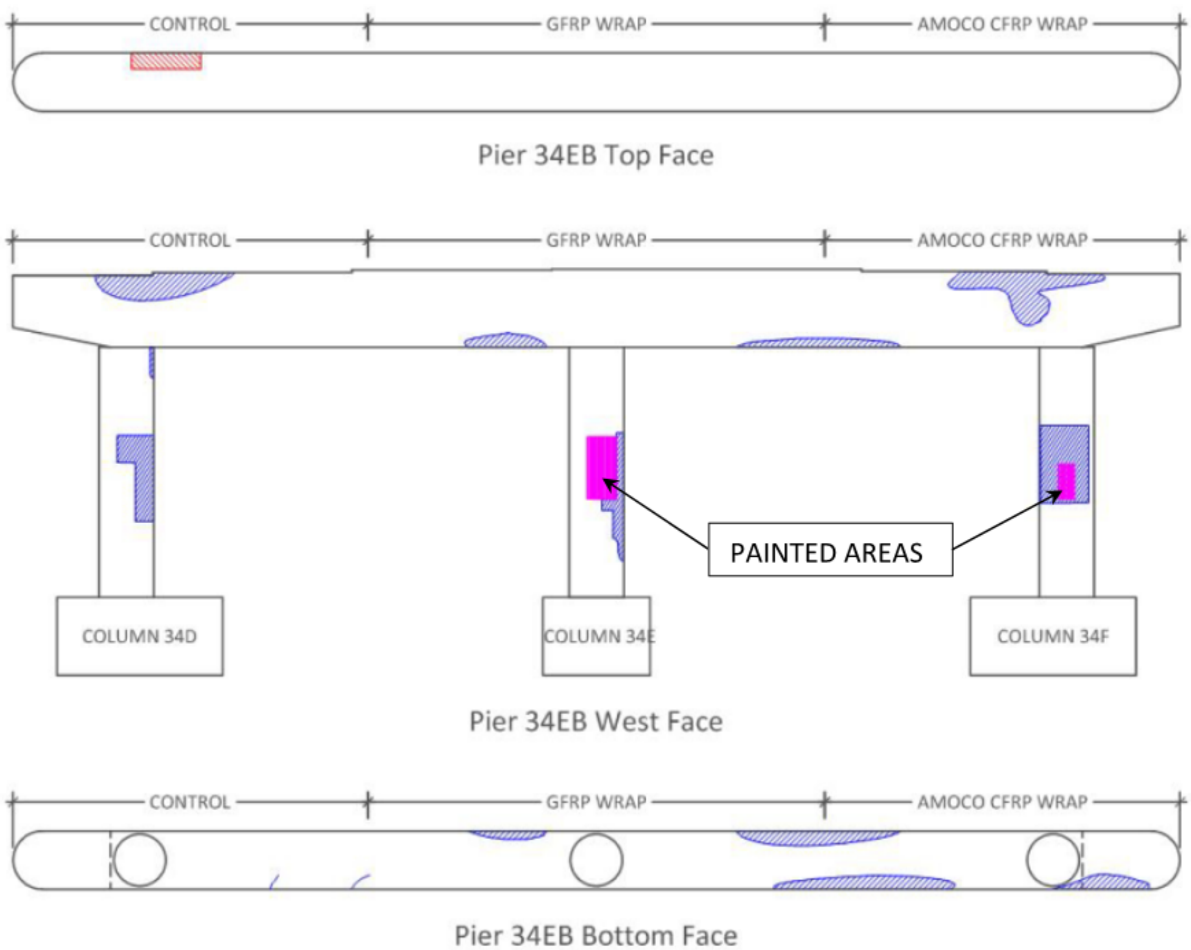


Figure 3.7 Pier 34EB - West Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.8 Pier 34EB - East Face

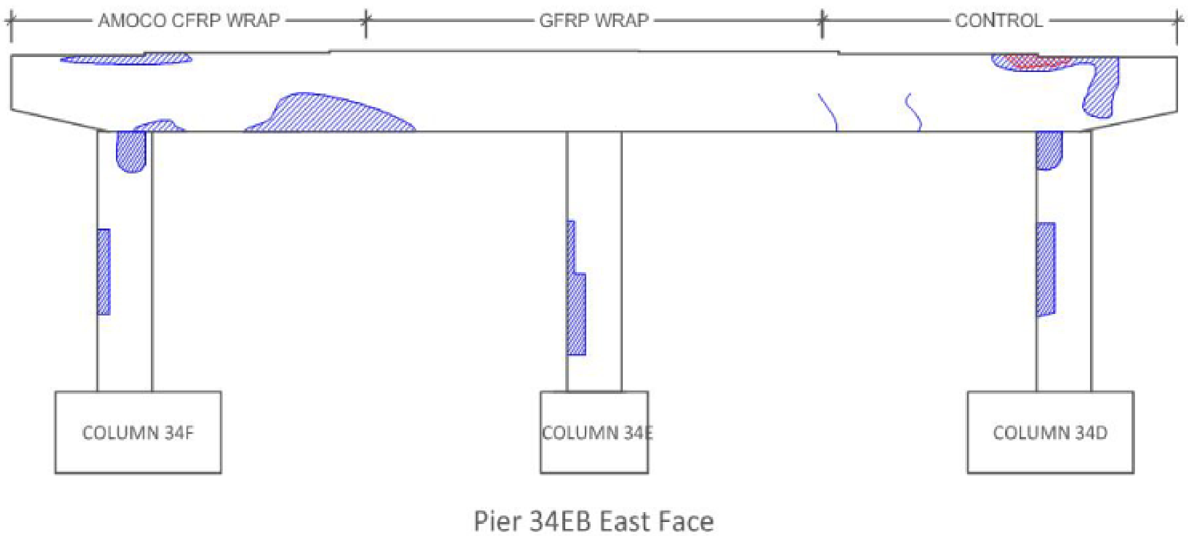


Figure 3.9 Pier 34EB - East Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.10 Pier 37WB - West Face

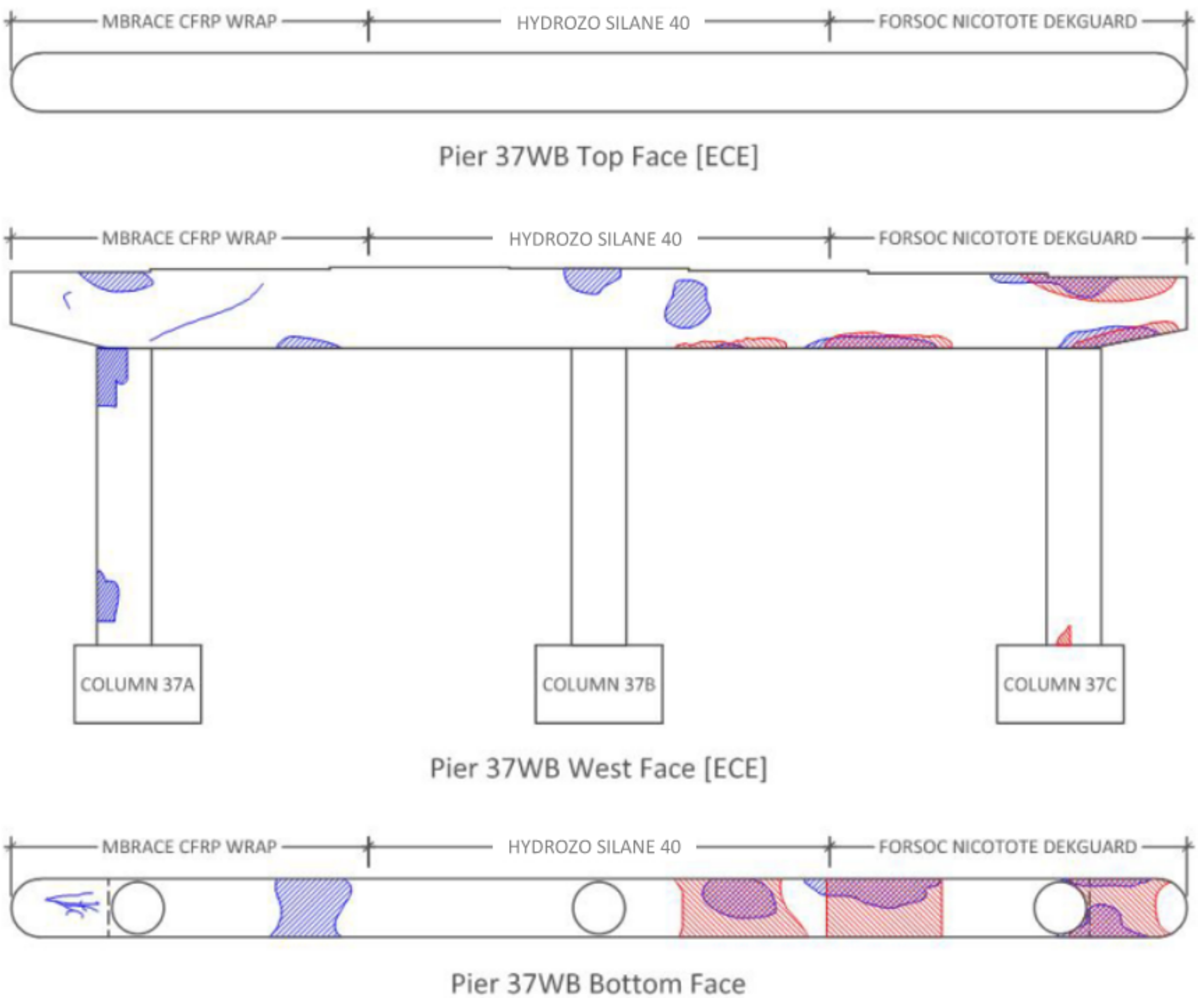


Figure 3.11 Pier 37WB - West Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.12 Pier 37WB - East Face

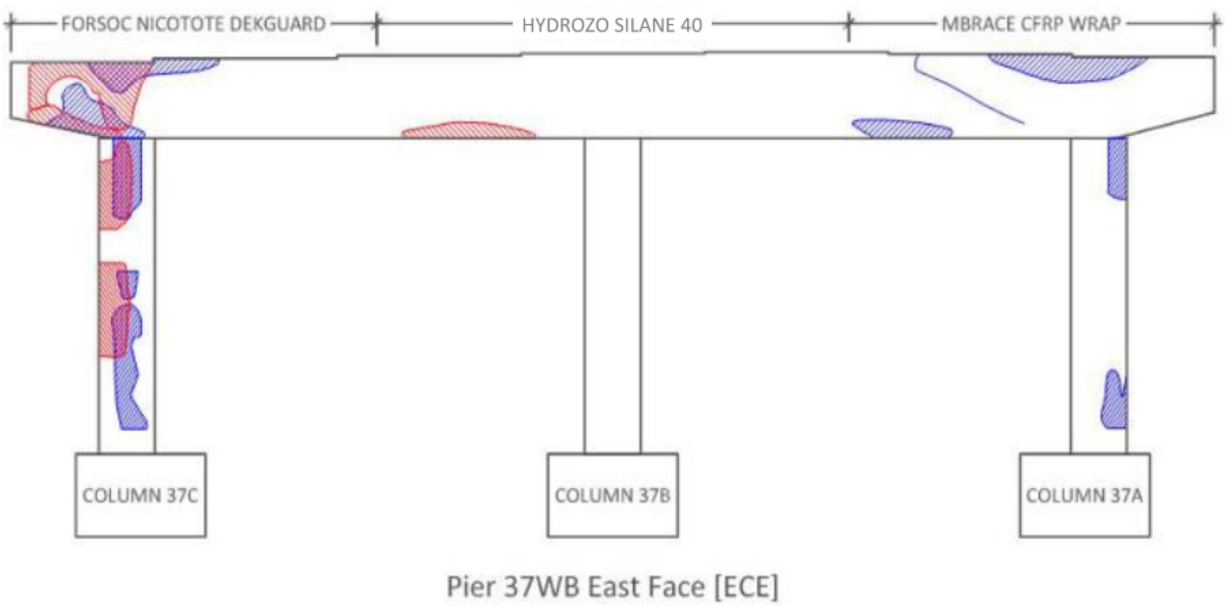


Figure 3.13 Pier 37WB - East Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.14 Pier 37EB - West Face

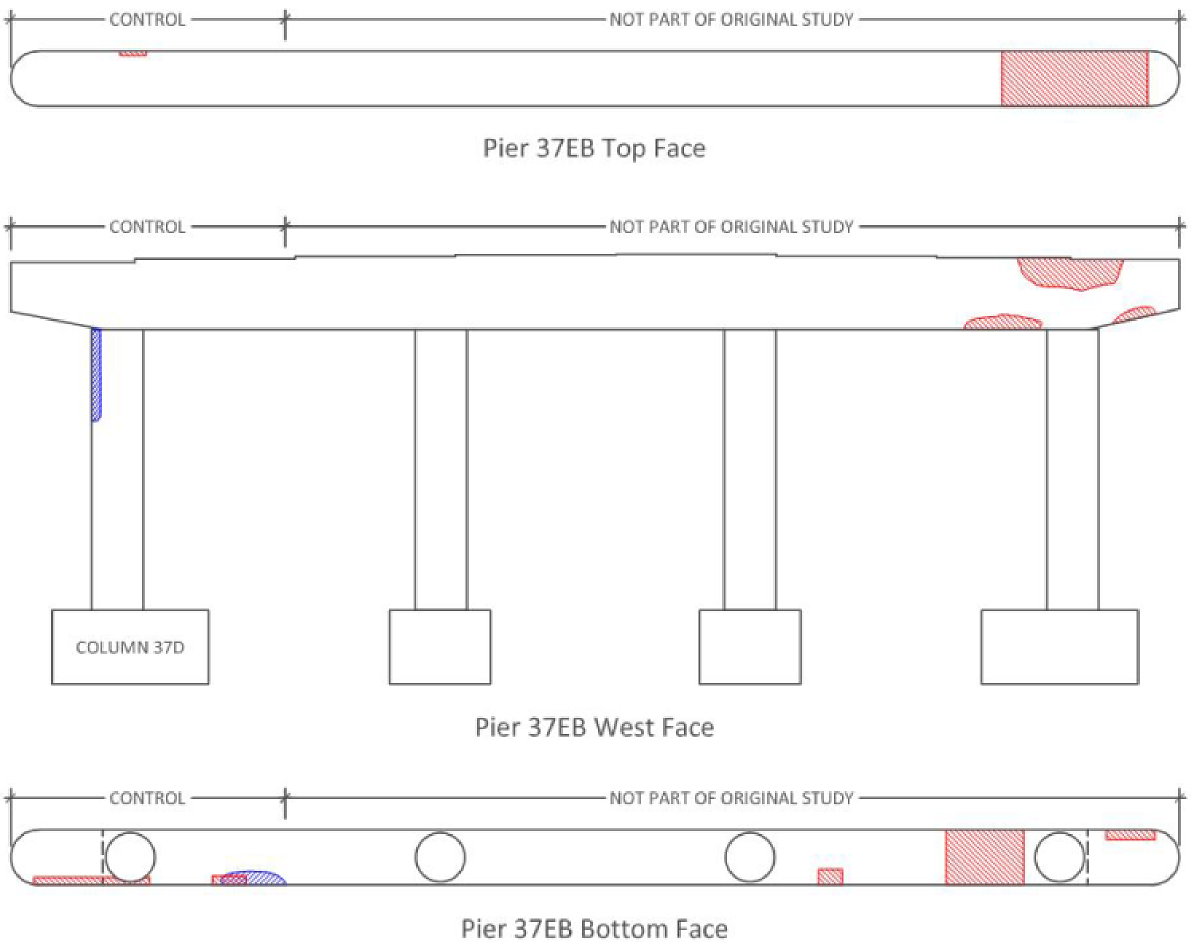


Figure 3.15 Pier 37EB - West Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.16 Pier 37EB - East Face

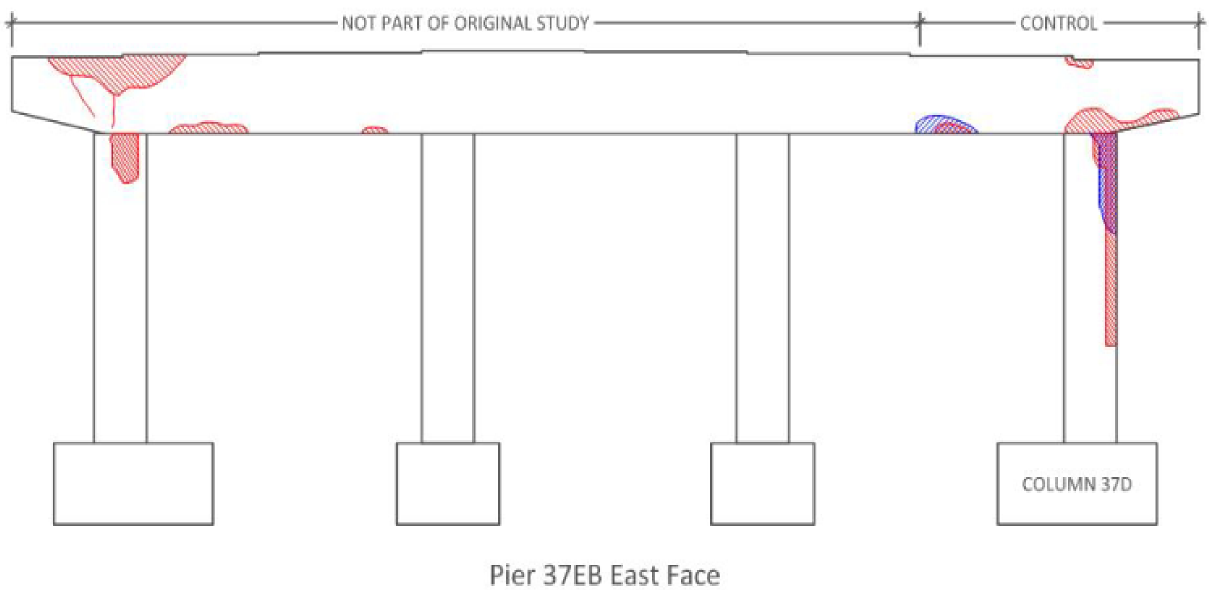


Figure 3.17 Pier 37EB - East Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.18 Pier 40WB - West Face

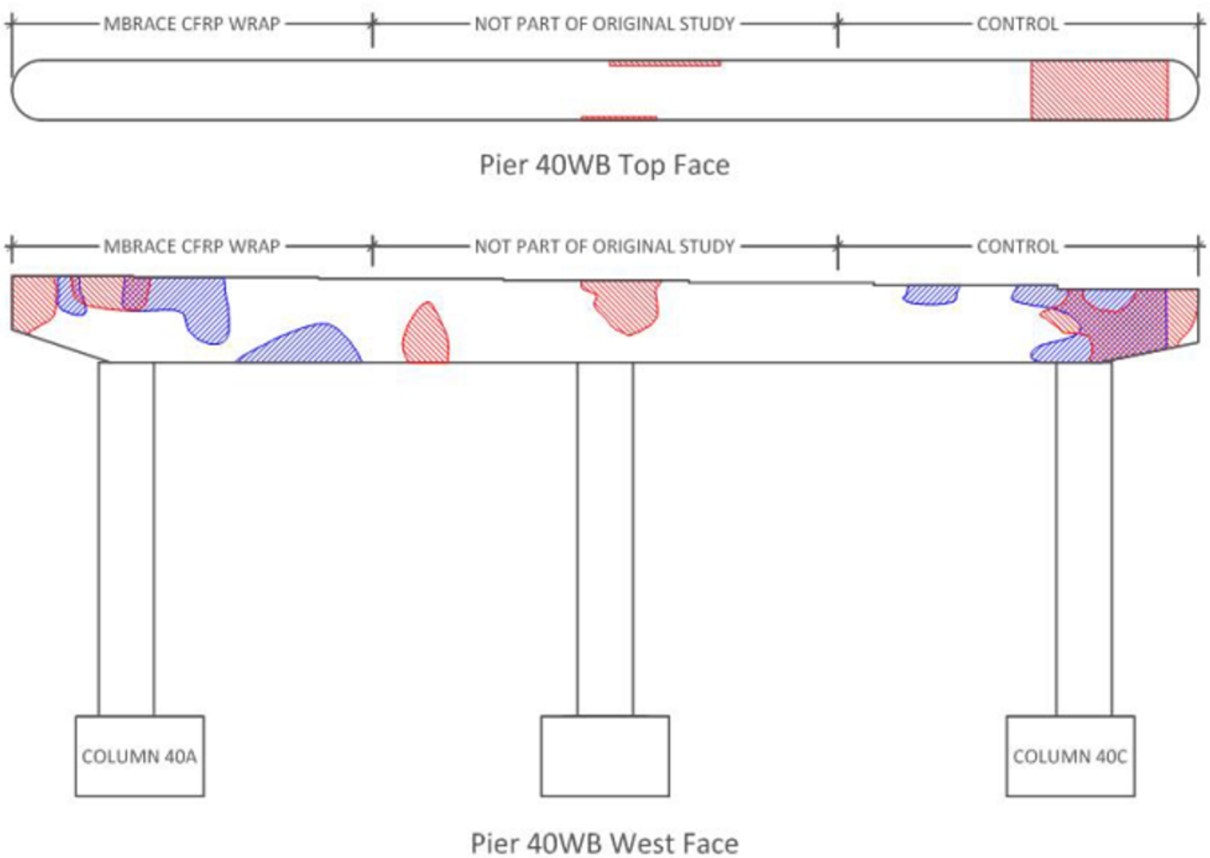


Figure 3.19 Pier 40WB - West Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

Figure 3.20 Pier 40WB - East Face

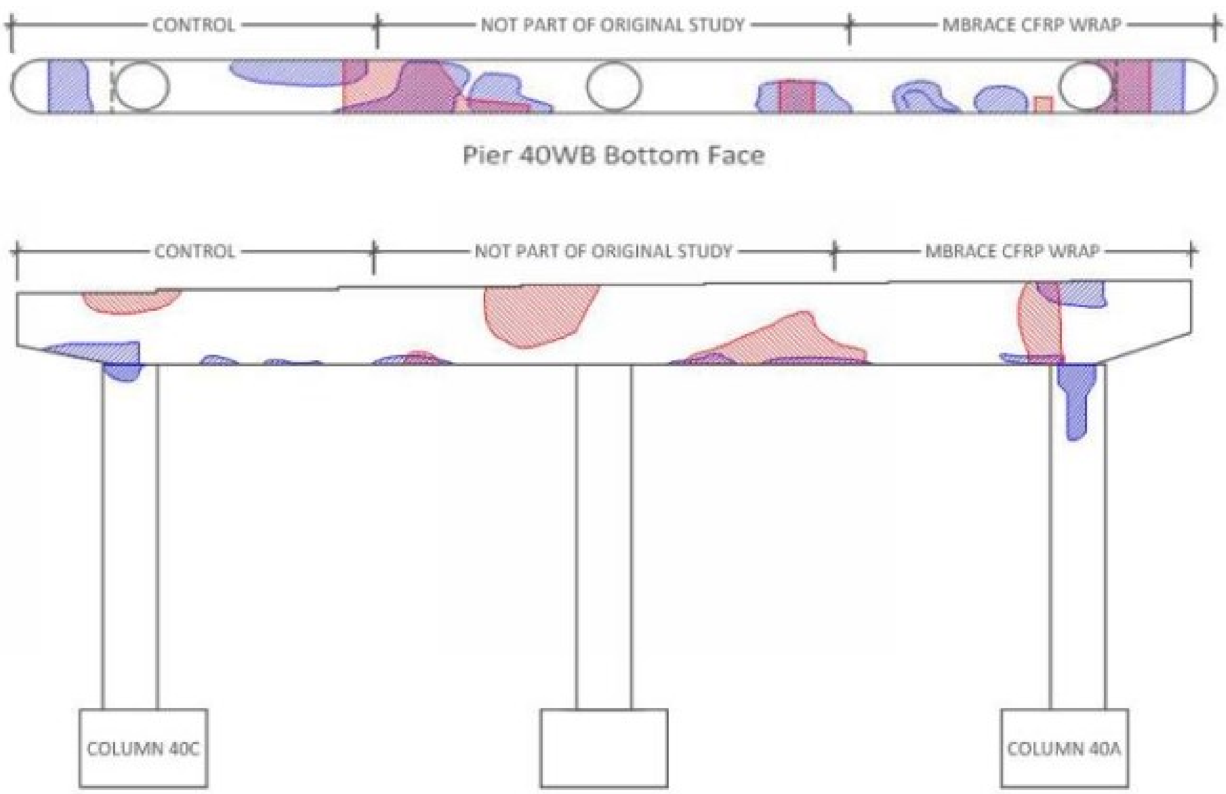


Figure 3.21 Pier 40WB - East Face - Overlay of 1998 repair areas (blue) and 2018 distress areas (red)

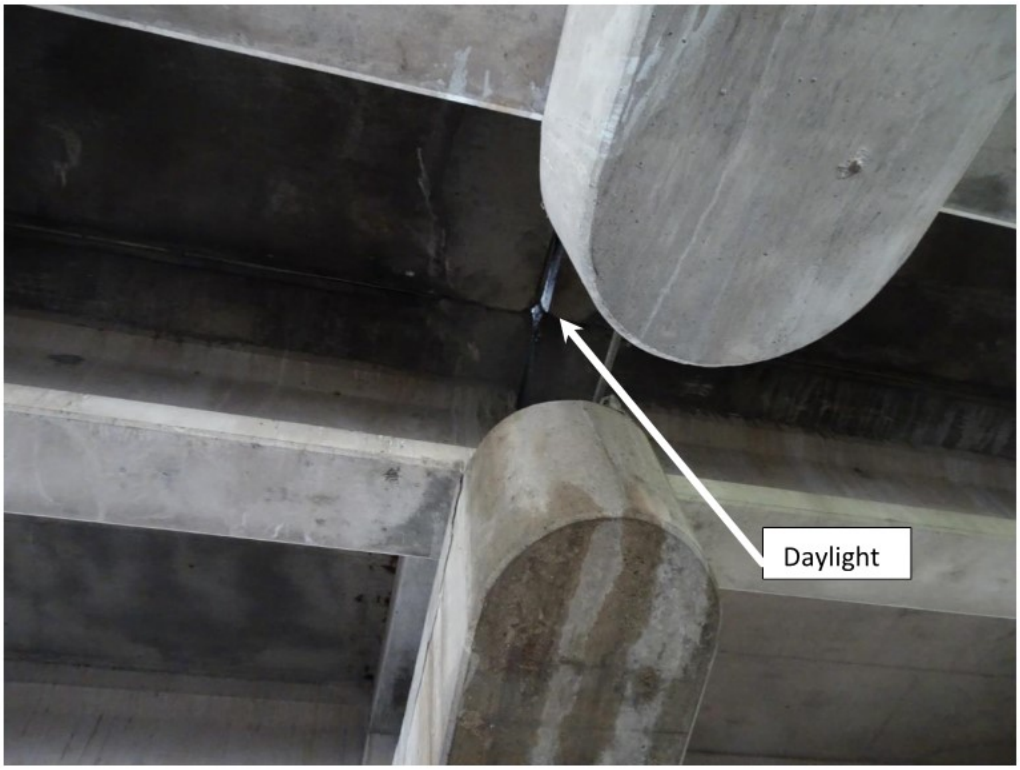


Figure 3.22 Pier 34WB - Daylight at deck joint and moisture staining below on end of pier cap.

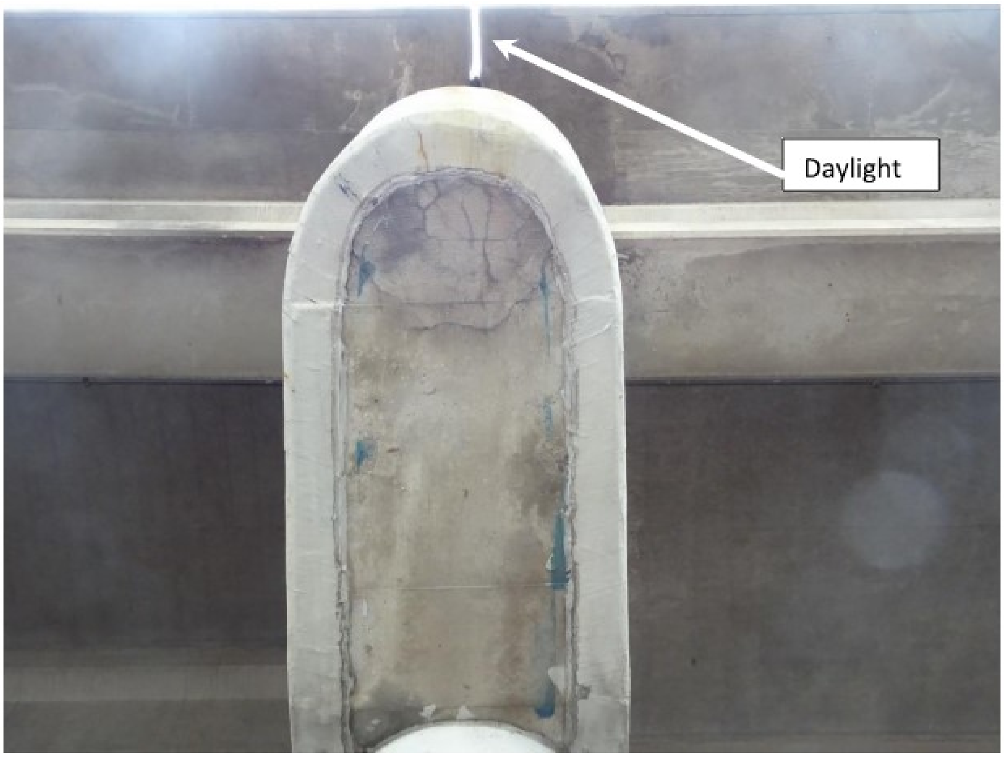


Figure 3.23 Pier 40WB - Daylight visible at a deck joint above the pier cap

Figure 3.24 Pier 34EB - Fire damage to FRP applied on east face

3.2.2 Concrete Cover Survey

Ground penetrating radar (GPR) was utilized to non-destructively assess the spacing and general concrete cover depth over the reinforcing steel at representative locations of the pier caps and columns. Concrete cover depth is useful in evaluating the risk of corrosion from chlorides and carbonation. At select locations, physical measurements of concrete cover were obtained to calibrate the GPR scans. This process consisted of drilling a small diameter hole into the concrete to the surface of the reinforcing bar and directly measuring the depth of the reinforcing bar. GPR was unsuccessful at detecting reinforcement behind areas wrapped in CFRP when scanning was performed in the direction perpendicular to the primary CFRP fiber direction; this was because the CFRP interfered with the radar signal. All collected data was analyzed to determine minimum, maximum, and average concrete cover depth over steel reinforcement in the surveyed areas. The standard deviations and coefficients of variation (COV) for the data were also calculated, to understand the variation or dispersion of the data. Refer to Table 3.1 for a summary of the results obtained.

The minimum design concrete cover over steel reinforcement for the columns and pier caps is shown as 2 inches in the original bridge plans. The average concrete cover measured in surveyed areas was approximately 2-1/2 inches, with limited dispersion in the data. However, some anomalies were noted. The concrete cover was as low as 0.8 inches for some reinforcement in Column 37C, and as high as 4.6 inches for some reinforcement in Column 34C.

Table 3.1 Summary of Concrete Cover Data

Pier	Element	Reinforcement Type	Avg. Nominal Spacing (in.)	Concrete Cover				
				Min. (in.)	Max. (in.)	Avg. (in.)	Std. Dev. (in.)	COV
34W	Column 34B	Vertical	N/A	1.6	3.8	2.7	0.7	27%
	Column 34B	Spiral	6	1.3	3.3	2.4	0.6	25%
	Column 34C	Spiral	6	1.3	4.6	2.9	1.0	35%
	Pier Cap - West Face	Transverse/Stirrup	N/A	1.9	2.2	2.1	0.1	4%
	Pier Cap - East Face	Transverse/Stirrup	N/A	2.3	3.4	3.0	0.3	11%
34E	Column 34D	Vertical	N/A	1.5	3.0	2.2	0.5	23%
	Column 34D	Spiral	6	1.2	2.9	2.1	0.5	25%
	Column 34E	Vertical	N/A	1.8	3.5	2.4	0.6	26%
	Column 34E	Spiral	6	1.3	3.2	2.2	0.5	21%
	Pier Cap- West Face	Transverse/Stirrup	N/A	2.3	2.8	2.5	0.1	4%
	Pier Cap - East Face	Transverse/Stirrup	N/A	1.9	3.3	2.7	0.3	12%
37W	Column 37A	Vertical	N/A	1.8	3.6	2.6	0.6	24%
	Column 37B	Vertical	N/A	1.4	3.2	2.3	0.7	28%
	Column 37B	Spiral	6	1.2	3.2	2.3	0.6	26%
	Column 37C	Vertical	N/A	0.9	4.0	2.4	1.1	44%
	Column 37C	Spiral	6	0.8	4.0	1.9	1.1	55%
	Pier Cap - West Face	Transverse/Stirrup	N/A	1.8	2.9	2.2	0.3	11%
	Pier Cap - East Face	Transverse/Stirrup	N/A	2.3	3.1	2.7	0.2	6%
37E	Column 37D	Vertical	N/A	1.9	2.4	2.2	0.2	8%
	Column 37D	Spiral	6	1.3	2.5	1.9	0.4	19%
40W	Column 40A	Vertical	N/A	1.7	2.6	2.1	0.3	15%
	Column 40C	Vertical	N/A	2.4	2.7	2.5	0.1	4%
	Column 40C	Spiral	6	1.6	2.6	2.1	0.3	14%
	Pier Cap - West Face	Transverse/Stirrup	N/A	1.7	2.5	2.1	0.3	12%

3.2.3 Half-Cell Potential Survey

Half-cell potential testing, also termed corrosion potential testing, was performed in general accordance with ASTM C876 to evaluate the corrosion activity of the reinforcing steel at Pier 34WB, Pier 34EB, Pier 37WB, and Pier 40WB [13]. This technique involves obtaining potential (voltage) readings between a portable reference electrode and the reinforcing steel embedded within the concrete element of interest using a high impedance voltmeter. The test requires an electrical connection to the reinforcement, and the reference electrode is placed on the concrete surface to obtain readings.

Half-cell potential testing cannot measure the extent of previous corrosion, or the present rate of corrosion, but the test is useful in assessing corrosion risk and identifying areas with probable active corrosion on a macro scale. Highly negative potential readings generally correlate to a high probability that active corrosion is occurring, as shown in Table 3.2. However, dramatic changes in measured half-cell potentials over a short distance are often a better indicator of corrosion activity than absolute potential values. Areas of more negative potential surrounded by areas of dramatically less negative (or even positive) potentials suggest the areas of more negative potential are anodic and likely undergoing corrosion. Distress conditions such as cracking and delaminations often result from corrosion activity, and thus usually coincide with more negative potential readings. However, anodic (corroding) regions that have not yet caused distress can also be identified by this technique. As such, corrosion potential testing can be used to identify regions of a structure that are both actively corroding and those that may see corrosion activity in the near future.

Table 3.2 Half-cell potential interpretation guidance (ASTM C876)

HCP vs CSE	Corrosion Activity
> -200 mV	> 90% probability that no corrosion is occurring (i.e., low risk)
-200 mV to -350 mV	Uncertain (i.e., moderate risk)
< -350 mV	> 90% probability that corrosion is occurring (i.e., high risk)

A prerequisite to performing corrosion potential testing on a grid over a large area is to first establish that the reinforcing steel in the structure is electrically continuous. Electrical continuity testing was performed prior to the survey by measuring the electrical resistance (ohms) between the steel reinforcement in different columns of the same pier using a multimeter. Access and connection to the steel reinforcement was made by drilling a 5/8-inch diameter hole into the concrete until the steel reinforcement was encountered and then attaching a wire lead to the steel. Resistances of one ohm (1.0 Ω) or less were measured for each of the piers, indicating satisfactory electrical continuity for collecting corrosion potential measurements from a single connection to the reinforcing steel at each pier.

Potentials were measured using a standard copper-copper sulfate reference electrode (CSE), with appropriate corrections applied for temperature effects. Measurements were obtained on a two foot spacing at the columns, and on a one foot vertical by two foot horizontal grid on the pier caps. In areas wrapped with FRP, a 5/8-inch diameter drill was used to pierce the FRP and allow for direct contact of the reference electrode to the concrete surface. The obtained corrosion potential measurement data is presented in Appendix C. Contour plots were generated from the measured corrosion potentials, expressed in millivolts (mV), and are shown in Figures 3.25, 3.27, 3.29 and 3.31. Images of the pier faces are included below contour plots for reference and are annotated with delamination survey findings from the 1998 and 2018 studies. Refer to Figures 3.26, 3.28, 3.30 and 3.32. Of note:

- Highly negative potentials indicative of high corrosion risk areas were measured at Pier 37WB, both at the south end of the pier cap and Column 37C, and at various locations along the pier cap of Pier 40WB. Concrete distress was present in all of these areas.
- Half-cell potentials between -200 and -350 mV, indicative of moderate corrosion risk, were measured at various locations in the pier caps of Pier 34WB and Pier 40WB, and at the tops of Columns 34C, 40A and 40C. Concrete distress was present in these areas.
- In general, the locations of highly negative or moderate corrosion potential were aligned with the locations where the visual survey identified areas of visible wetness or water staining below open strip seal deck joint conditions.

Half-cell potentials which were indicative of moderate corrosion risk were also measured throughout the south end of Pier 34EB, both in the pier cap and over the height of Column 34F, behind the AMOCO CFRP which had been installed as part of the 1998 study. The potentials which were measured in this section were relatively uniform. However, no distress was identified by sounding through the FRP wrap at any location within this section. Similar uniformity was observed in the half-cell potential data which was obtained at the north end section of Pier 34WB, where AMOCO CFRP was also installed, although all of those readings were indicative of low corrosion risk. It was not clear to the authors if the corrosion potential data which was obtained for areas treated with AMOCO CFRP was accurate or was instead influenced by limited oxygen availability or other factors associated with that particular FRP system.

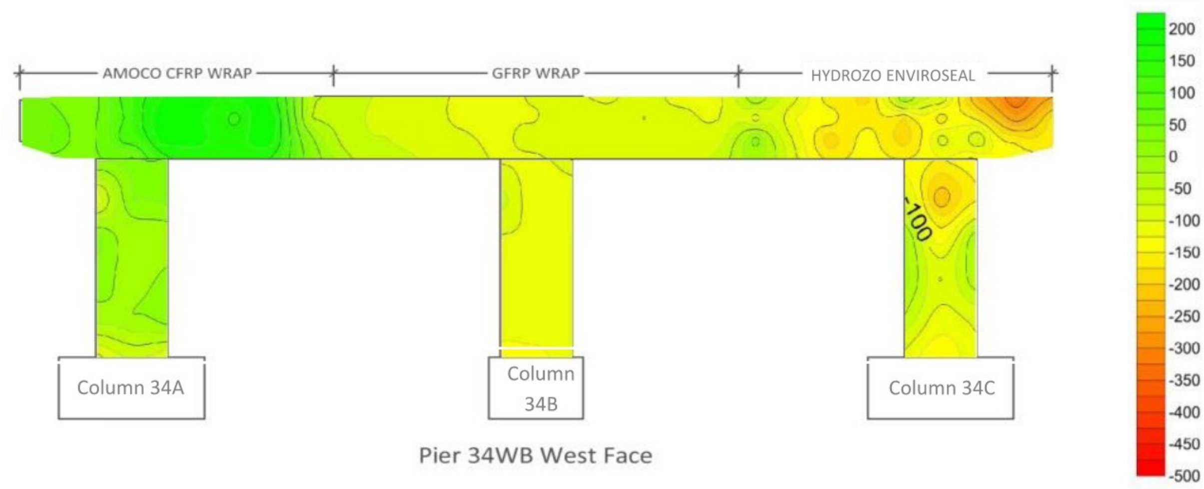


Figure 3.25 Pier 34WB [ECE] - Results of half-cell potential testing at west face

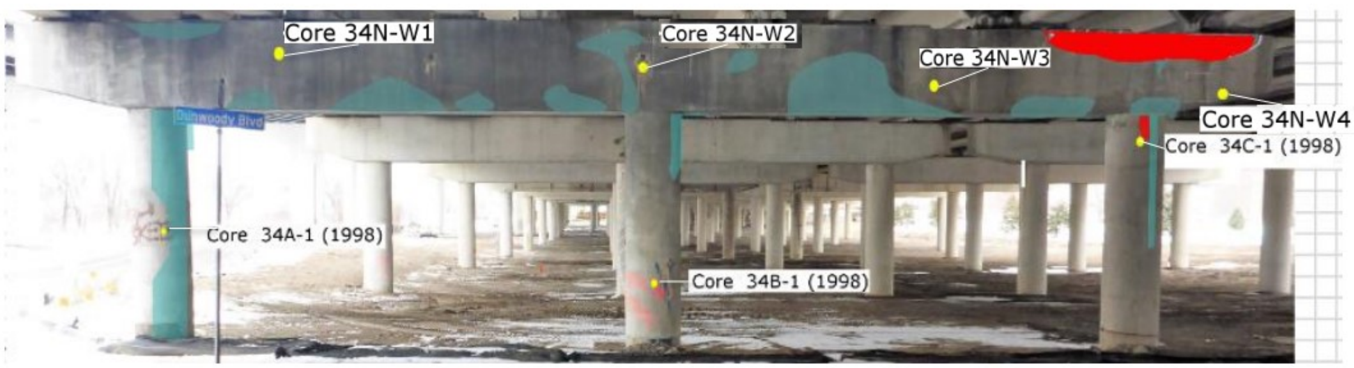


Figure 3.26 Pier 34WB [ECE] - West face - Delaminations in 2018 (red) and 1998 repair areas (blue)

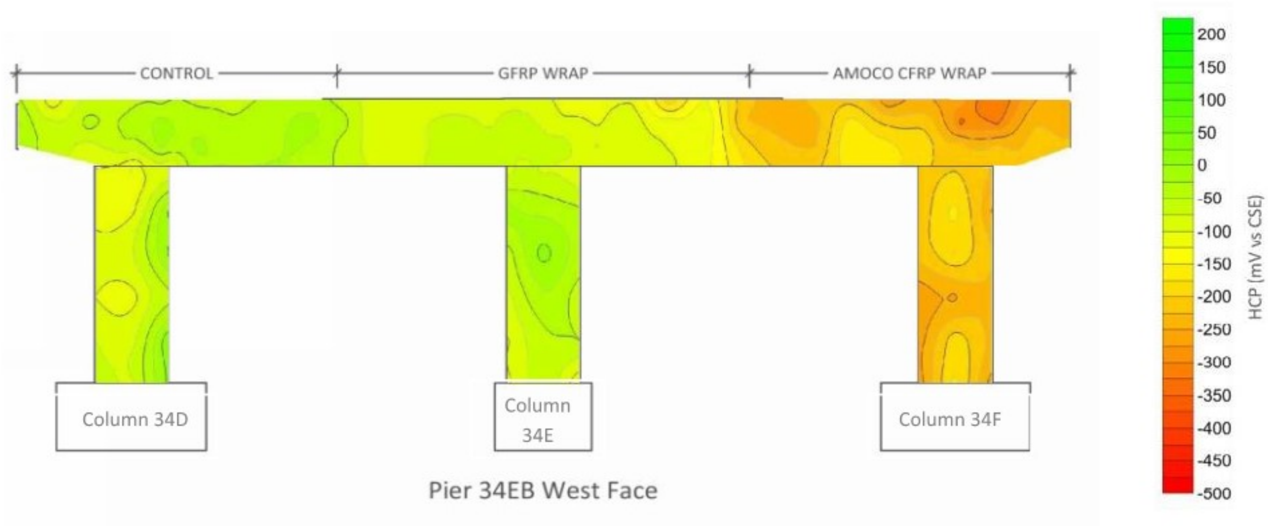


Figure 3.27 Pier 34EB - Results of half-cell potential testing at west face

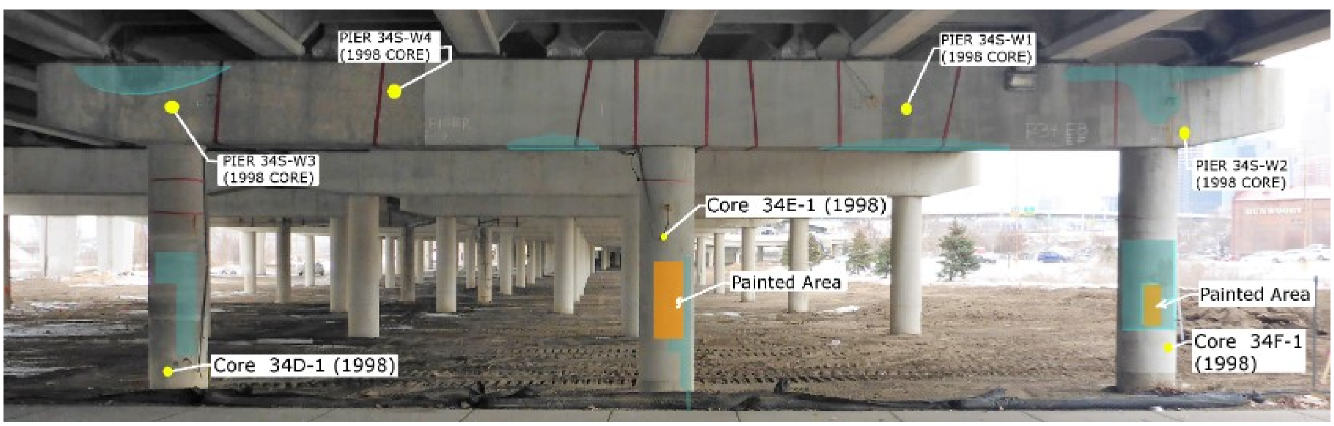


Figure 3.28 Pier 34EB - West face - Delaminations in 2018 (red) and 1998 repair areas (blue)

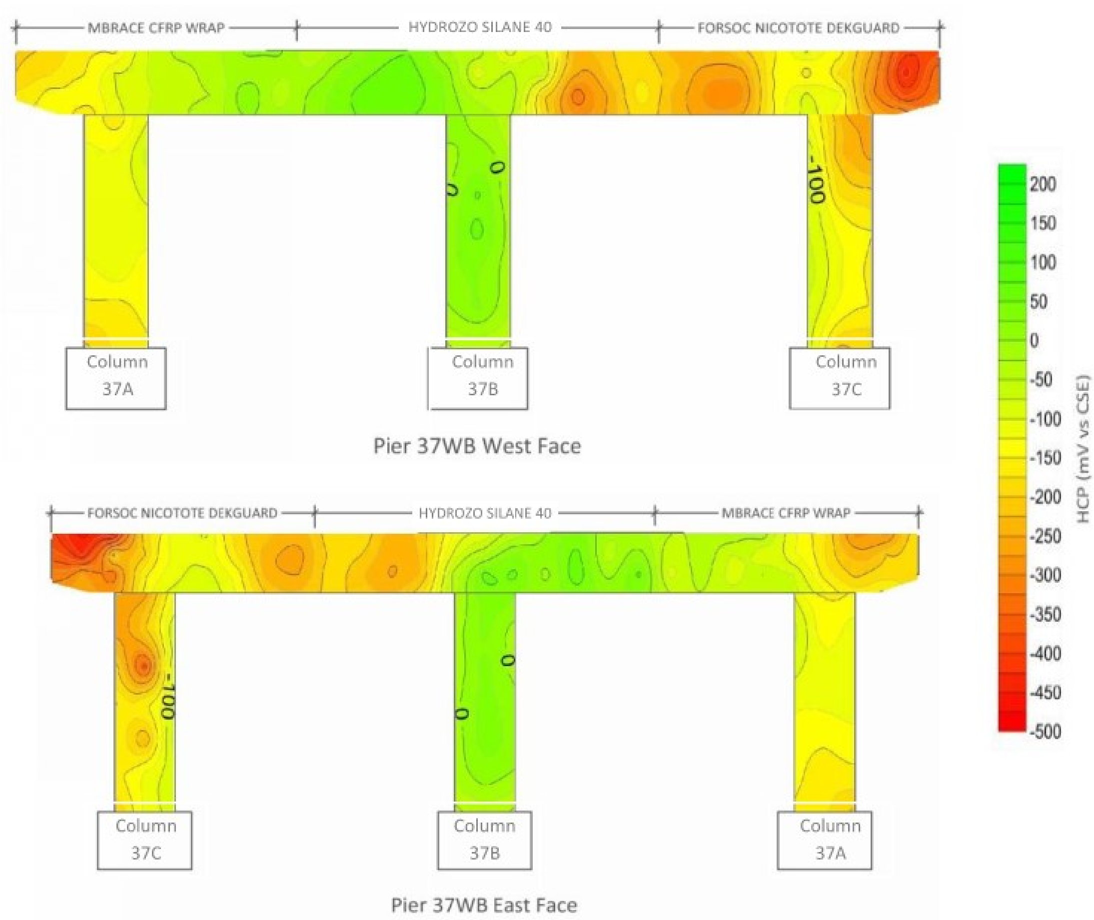


Figure 3.29 Pier 37WB [ECE] - Results of half-cell potential testing at west and east faces

Figure 3.30 Pier 37WB [ECE] - East face - Delaminations in 2018 (red) and 1998 repair areas (blue)

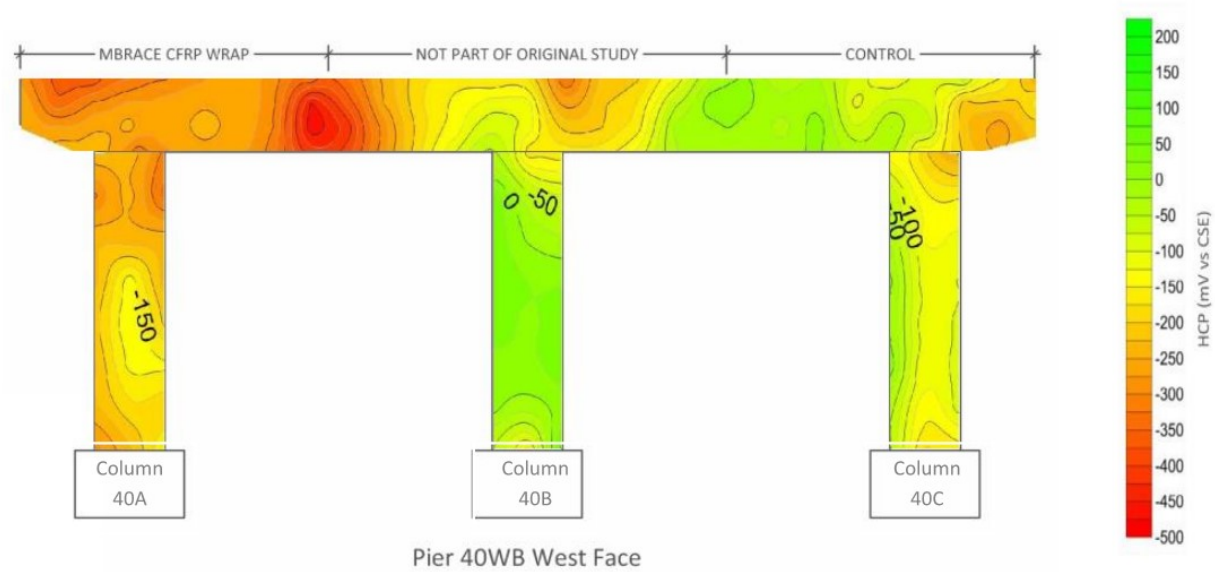


Figure 3.31 Pier 40WB - Results of half-cell potential testing

Figure 3.32 Pier 40WB - West face - Delaminations in 2018 (red) and 1998 repair areas (blue)

3.2.4 Concrete Resistivity Measurements

Surface measurements of concrete resistivity were collected using the Proceq Resipod Concrete Resistivity Meter, a four-pin Wenner-type probe. These measurements are of interest because the rate at which corrosion can occur in a concrete structure is inversely proportional to the resistivity of the concrete. High concrete resistivity restricts the flow of current along the ionic current path of a corrosion mechanism, while low concrete resistivity can facilitate higher current flow and higher corrosion rates. However, resistivity decreases as the concrete moisture content increases, but increases where the concrete carbonates. These near-surface conditions can potentially affect the readings obtained with a four-pin Wenner type probe and give resistance measurements which may be different than that at the depth of the reinforcement, where the corrosion reactions are occurring. Carbonation measurements are presented and discussed in Section 3.3.1.

Measurements of concrete resistivity were obtained adjacent to core sampling locations on Pier 34WB, Pier 34EB, Pier 37WB, and Pier 40WB where the concrete surface was exposed. The measurement locations are shown in Figure 3.35 to Figure 3.39. Multiple measurements were collected within each test location. The average measurements obtained are summarized in Table 3.3.

Table 3.3 Summary of Concrete Resistivity Measurements

Pier	Element	Treatment	Location	Test Location ID	Average Concrete Resistivity (kΩ-cm)
34WB	Pier Cap	AMOCO CFRP	North End	1734N-W1	136
	Pier Cap	Sealer	South End	1734N-W4	185
	Colum 34C	Sealer	Near Base	N/A	437
34EB	Pier Cap	Control	North End	1734S-W3	149
	Pier Cap	AMOCO CFRP	South End	1734N-W2	127
	Column 34D	Control	Near Base	N/A	316
37WB	Pier Cap	Sealer	Middle	37N-E4	500
	Pier Cap	Sealer	South End	1737N-E2	423
	Column 37B	Sealer	Mid-height	37B-3	415
	Column 37C	Sealer	Mid-height	N/A	203
40WB	Pier Cap	Control	South End	1740N-W4	327
	Column 40C	Control	Near Base	N/A	411

The averages of all of the concrete resistivity measurements which were obtained from all tested locations ranged from 127 kΩ-cm to 500 kΩ-cm. Readings greater than 100 kΩ-cm are indicative of a very high concrete resistivity which would be expected to significantly limit the rate of any corrosion reactions [1].

3.2.5 FRP Bond Testing

Pull-off testing of FRP wrap systems was performed to evaluate the quality of the FRP bond to the substrate concrete after 20 years of service. Two pull-off tests were performed for each of the three different types of FRP wraps which were installed as part of the 1998 study. Testing was performed in general accordance with ASTM D7522 *Standard Test Method for Pull-Off Strength for FRP Laminate Systems Bonded to Concrete Substrate* [15]. A diamond bit hole saw with a 2-inch inside diameter was used to score through the FRP and into the concrete substrate, to create isolated test regions of known area. The scored test regions were lightly sanded and cleaned to improve bonding of the loading fixture. A 2-inch diameter bond pull-off loading dolly was then attached to the prepared test region using a fast setting epoxy. The epoxy was allowed to cure a minimum of two hours before performing the pull-off test. A Proceq DYNA Z16 Pull-off Tester was used to perform the tests.

Testing was performed at a column and pier cap for each system. This included two locations on Pier 34WB, two locations on Pier 34EB, and two locations on Pier 37WB. These locations are shown in Figure 3.35 to Figure 3.39. No pull-off testing was performed at the east face of Pier 34WB where the installed FRP wrap was fully delaminated as a result of a past fire. Pull-off test results are summarized in Table 3.4.

Table 3.4 Summary of FRP Bond Pull-off Testing

Location	Element	Test Location ID	FRP Type	Maximum Pull-off Load (lbs)	Maximum Pull-off Stress (psi)	Failure Plane
34WB	Column 34A	34WB-P1	AMOCO CFRP	610	> 195	Test fixture adhesive
	Column 34B	34WB-P2	GFRP	2171	> 694	Test fixture adhesive
34EB	Pier Cap	34EB-P1	GFRP	1590	> 508	Concrete
	Pier Cap	34EB-P2	AMOCO CFRP	364	> 116	Test fixture adhesive
37WB	Pier Cap	37WB-P1	MBrace CFRP	411	> 131	Test fixture adhesive
	Column 37A	37WB-P2	MBrace CFRP	452	> 144	Test fixture adhesive

Five of six pull-off tests ended with failure of the epoxy bonding adhesive which was used to attach the loading fixture, and one test performed in an area of GFRP ended with cohesive failure in the concrete substrate. See Figure 3.33. None of the tests produced failure at the FRP/concrete interface. The ultimate bond strength of the FRP wrap to the concrete was greater than the maximum pull-off stress achieved during testing. These results suggest that, at the six test sites, 20 years of exposure in a freeze-thaw environment has not resulted in a weak bond, as might be represented by bond strengths of less than 100 psi between the FRP wrap to the concrete substrate.

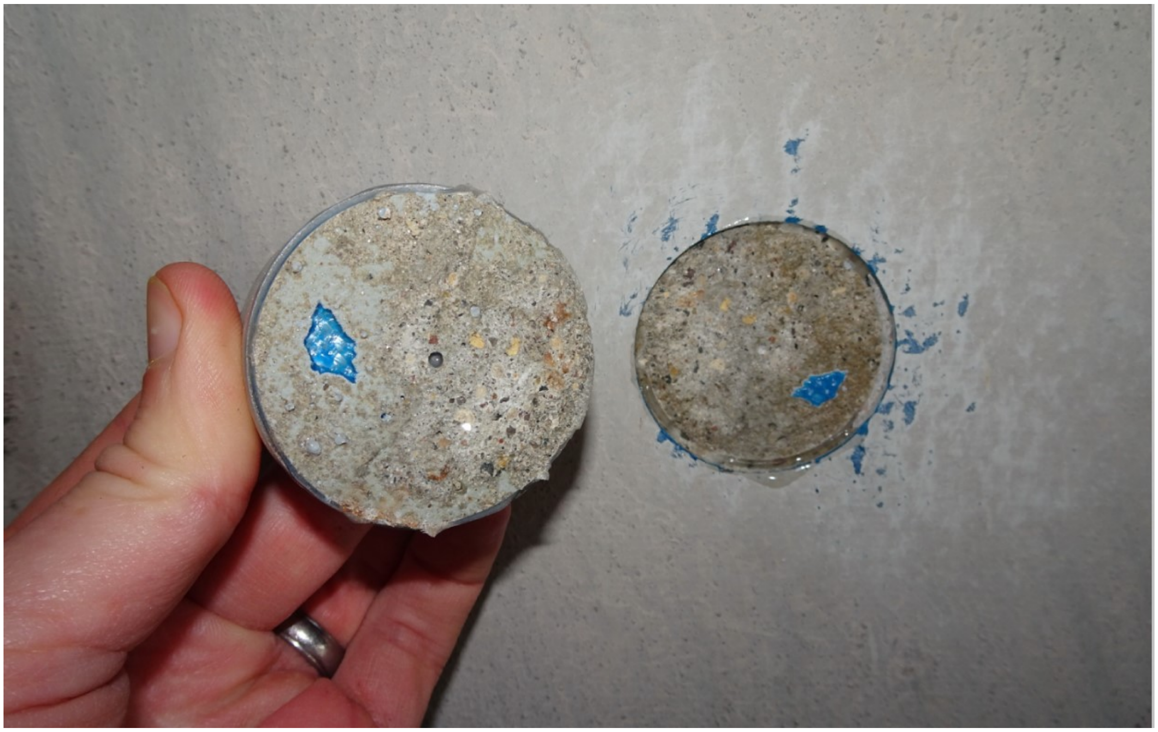


Figure 3.33 Pull-off testing of FRP wrap - cohesive failure in the concrete substrate

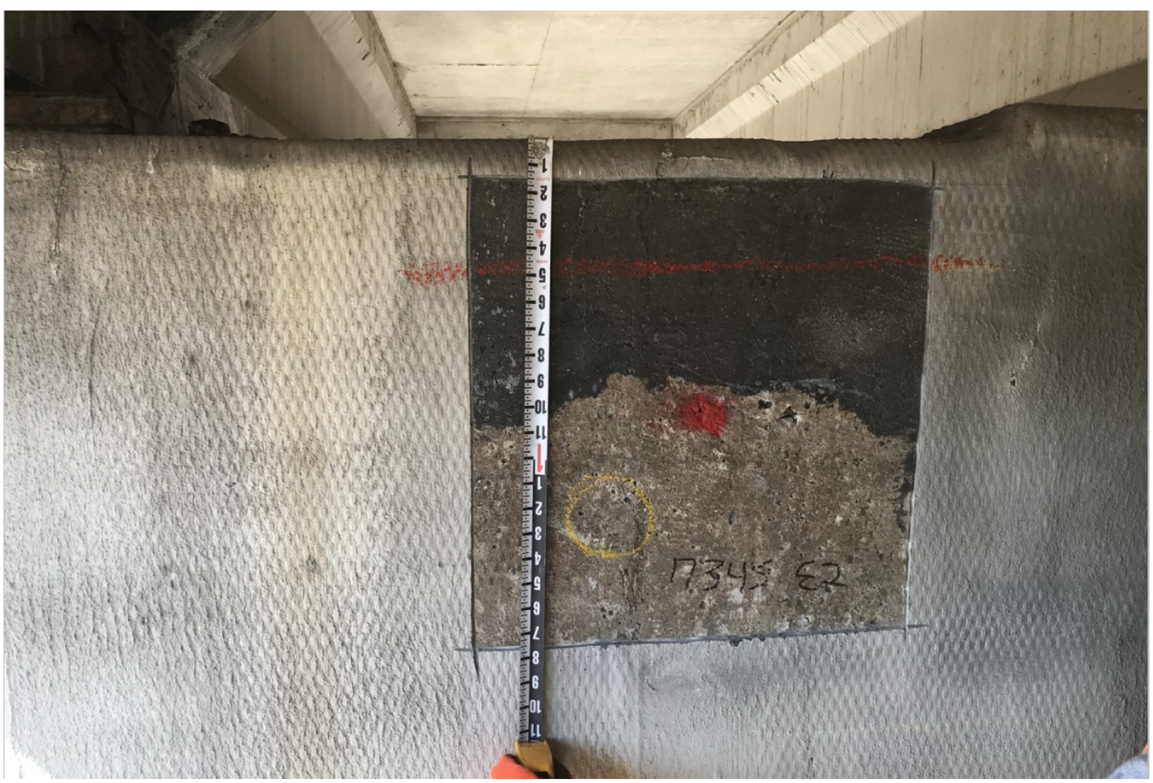


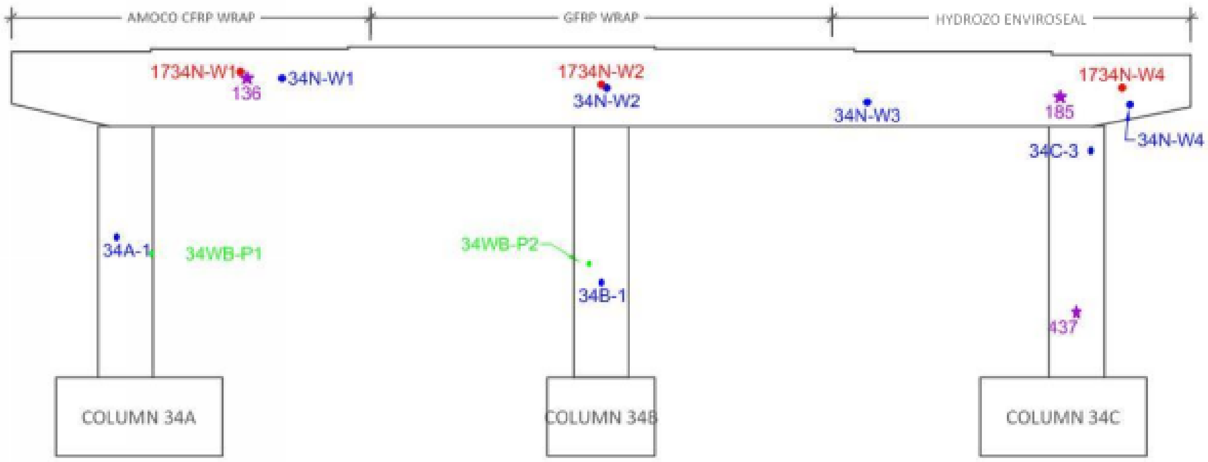
Figure 3.34 Pier 34EB - Example CFRP removal to facilitate locating of core sample

3.2.6 Concrete Core Sampling

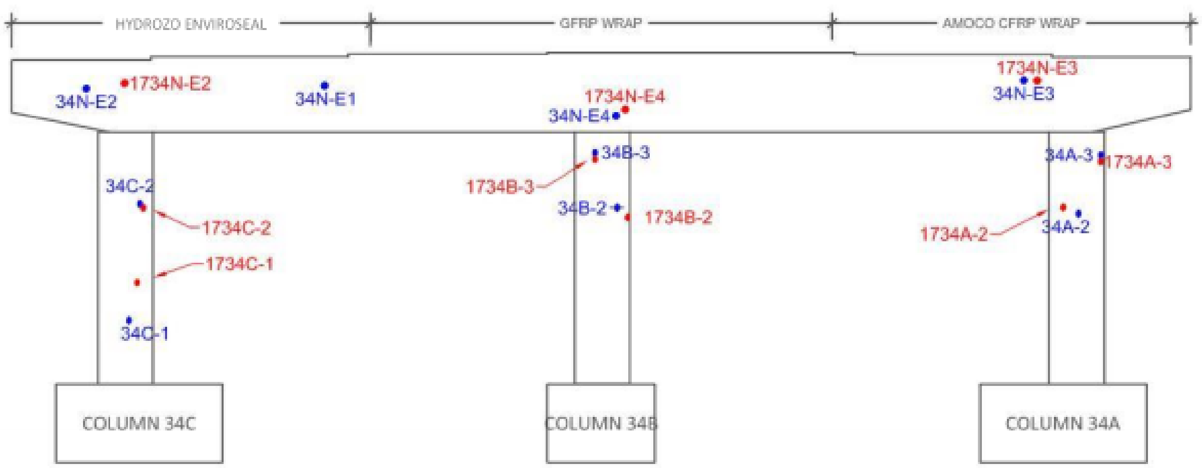
Locations were identified for forty-six (46) concrete core samples to be drilled for laboratory testing of chloride concentration and carbonation depths. The core locations were positioned as near as could be determined to the locations where chloride sampling was performed in the pre-ECE and post-ECE phases of the 1998 study. GPR was utilized to locate steel reinforcement in proximity to intended coring locations, and the final core placement was selected to avoid cutting bars with the coring bit.

As previously noted, GPR was unsuccessful at detecting reinforcement behind areas wrapped with AMOCO and MBrace CFRP. MnDOT personnel instead removed 18 inch by 18 inch square areas of these wraps at locations of intended coring to allow access to the concrete surface for the GPR unit. See Figure 3.34.

The concrete core locations of the 1998 study and the 2018 study are depicted in Figure 3.35 to Figure 3.39. Four-inch diameter concrete cores were extracted by MnDOT personnel at all identified locations. All core samples were obtained by drilling horizontally into the pier caps and columns and the work was performed in general accordance with ASTM C42 [16]. The concrete core samples were then provided to the authors and were shipped to the WJE Janney Technical Center (JTC) laboratory in Northbrook, Illinois for laboratory testing of chloride content and carbonation depth. The procedures and results of the testing are discussed in Section 3.3.



Pier 34WB West Face [ECE]



Pier 34WB East Face [ECE]

KEY

- = CHLORIDE POWDER SAMPLE LOCATION (1998 STUDY)
- = CONCRETE CORE SAMPLE LOCATION (2018 STUDY)
- = 2018 FRP BOND PULL-OFF TEST LOCATION
- ★ = 2018 CONCRETE RESISTIVITY TEST LOCATION (### = AVERAGE RESISTIVITY IN KΩ-CM)

Figure 3.35 Pier 34WB - Core sampling and testing locations

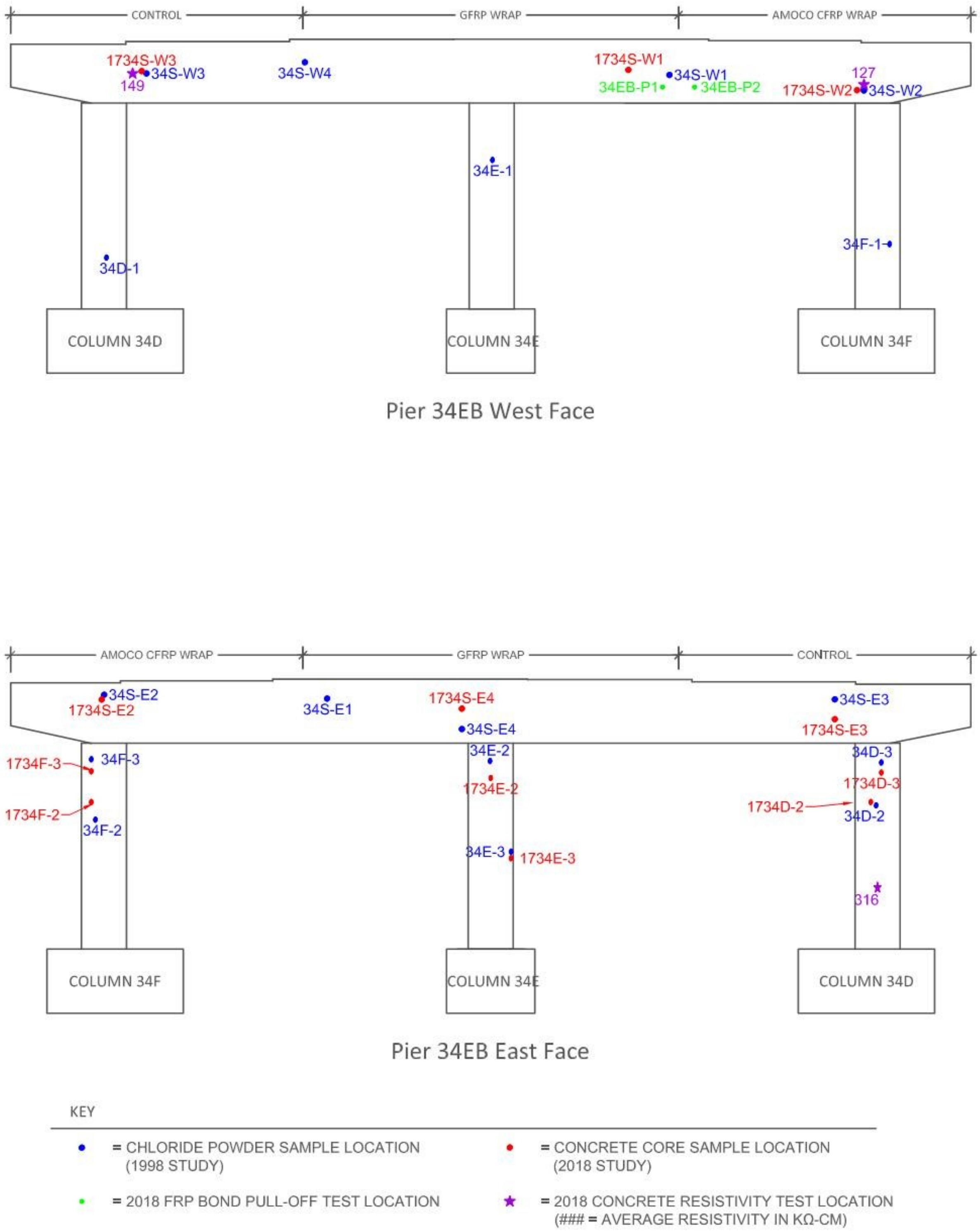


Figure 3.36 Pier 34EB - Core sampling and testing locations

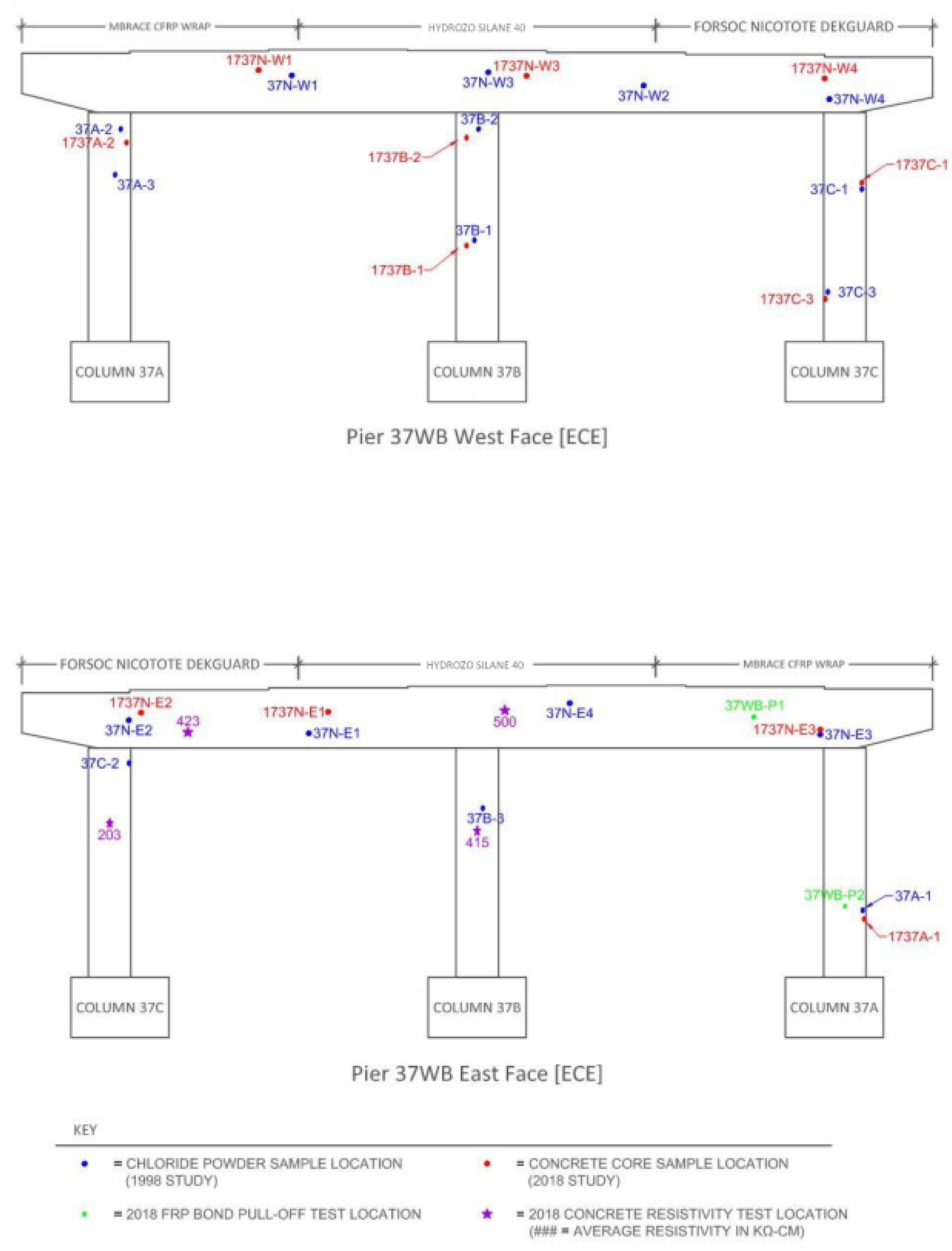


Figure 3.37 Pier 37WB - Core sampling and testing locations

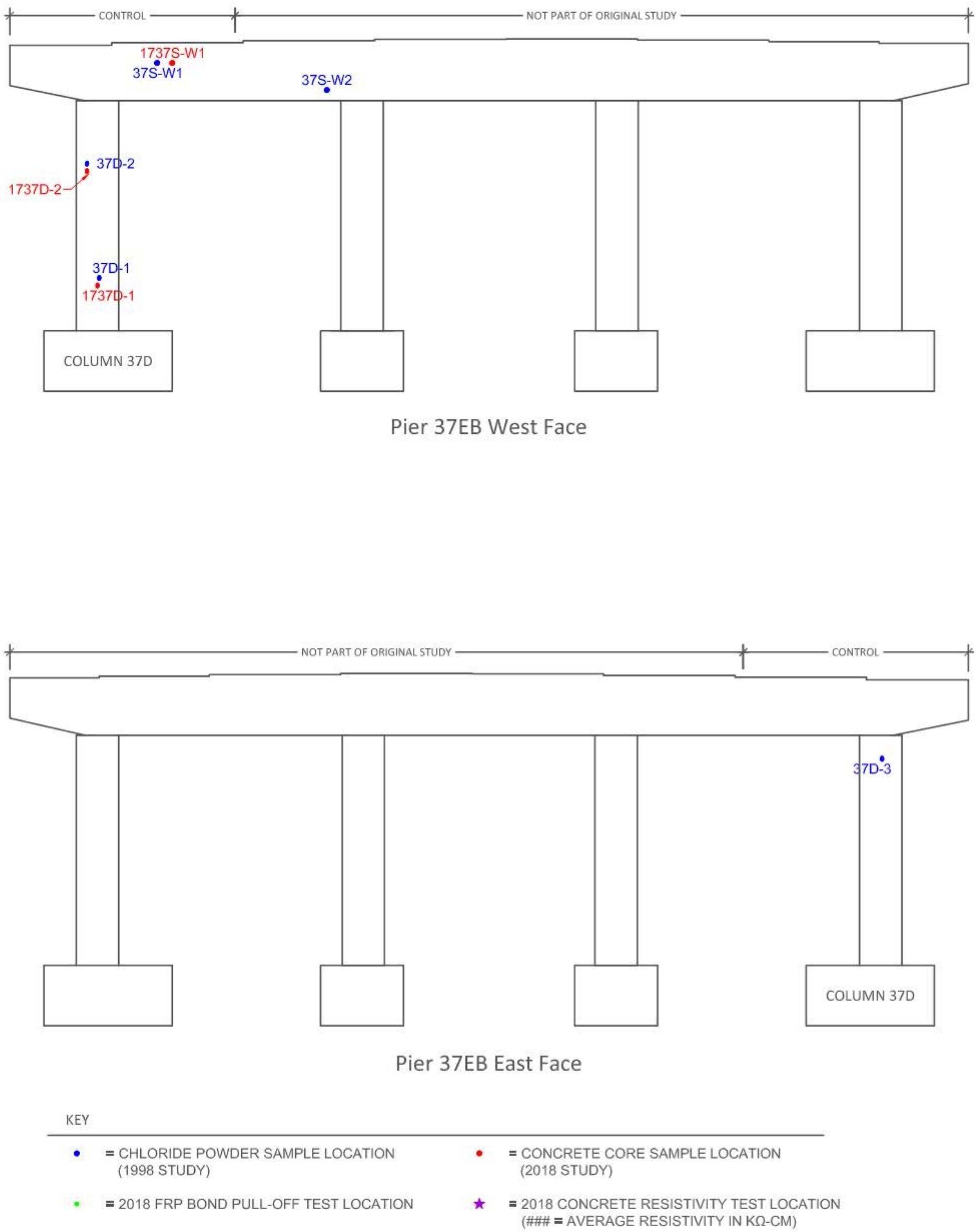
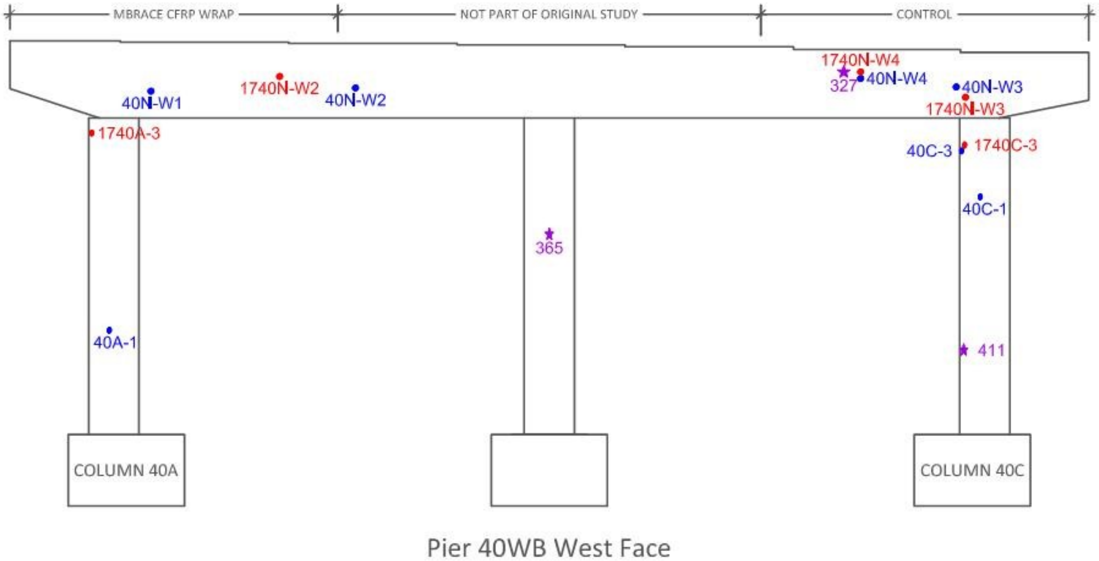


Figure 3.38 Pier 37EB - Core sampling and testing locations



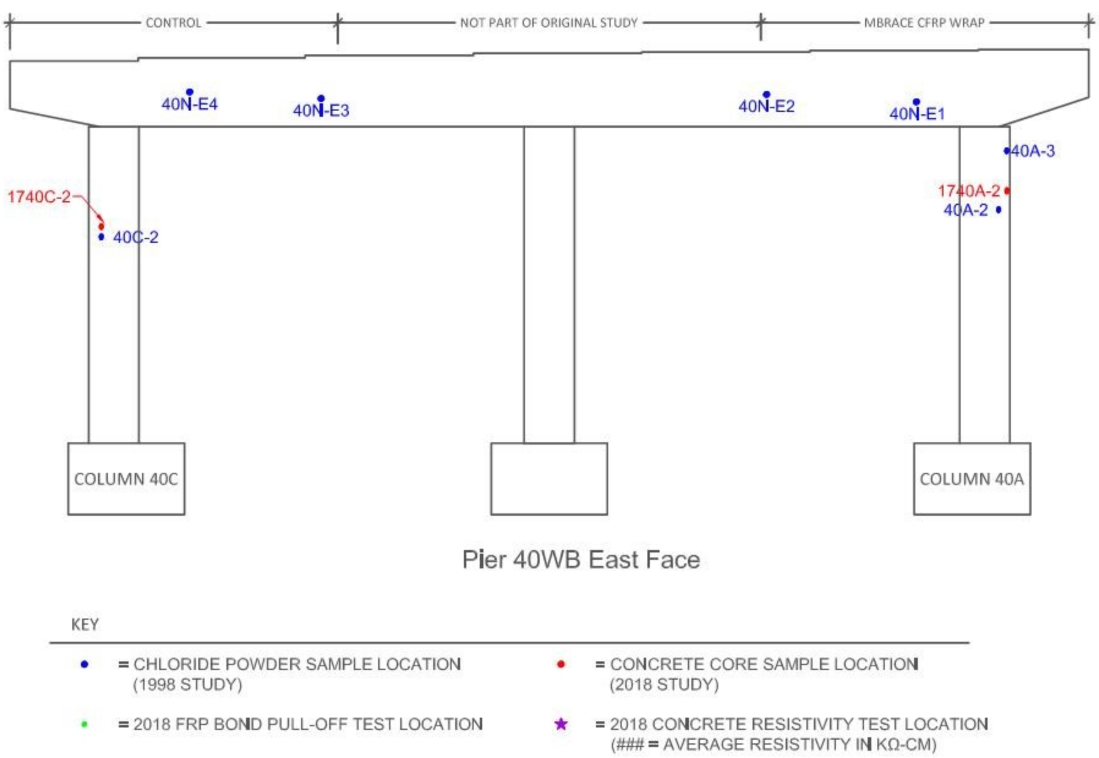


Figure 3.39 Pier 40WB - Core sampling and testing locations

3.3 LABORATORY STUDIES

3.3.1 Carbonation Testing

All forty six (46) cores were evaluated for depth of carbonation by applying a 1% phenolphthalein indicator solution on freshly broken sections. The broken sections were obtained by cutting longitudinally along the side edge of the core, and then splitting off a thin section with a chisel. The phenolphthalein solution was then immediately applied. Carbonated concrete (pH less than about 9) will remain the same color while non-carbonated concrete (pH greater than about 9) will turn magenta. The depth of carbonation is then measured as the thickness of the non-magenta concrete layer. Depths of carbonation, measured from the exterior face of each core, are summarized in Table 3.5. In all cores, the depth of carbonation was much less than the specified 2-inch minimum cover to the reinforcing steel.

Table 3.5 Summary of Carbonation Testing Results

Pier	Element	Sample ID	Carbonation Depth (in.)	Pier	Element	Sample ID	Carbonation Depth (in.)
34WB	Column A	1734A-2	0.05	34EB	Column D	1734D-2	0.25
		1734A-3	0.17			1734D-3	0.24
	Column B	1734B-2	0.00		Column E	1734E-2	0.25
		1734B-3	0.04			1734E-3	0.27
	Column C	1734C-1	0.15		Column F	1734F-1	0.03
		1734C-2	0.28			1734F-2	0.11
	Pier Cap	1734N-W1	0.00		Pier Cap	1734S-W1	0.00
		1734N-W2	0.00			1734S-W2	0.03
		1734N-W4	0.12			1734S-W3	0.13
		1734N-E2	0.02			1734S-E2	0.07
		1734N-E3	0.07			1734S-E3	0.14
		1734N-E4	0.05			1734S-E4	0.00
37WB	Column A	1737A-1	0.03	37EB	Column D	1737D-1	0.23
		1737A-2	0.82			1737D-2	0.25
	Column B	1737B-1	0.55	40WB	Column A	1740A-2	0.14
		1737B-2	0.63			1740A-3	0.84
	Column C	1737C-1	0.32		Column C	1740C-2	0.51
		1737C-3	0.04			1740C-3	0.74
	Pier Cap	1737N-W1	0.25		Pier Cap	1734S-W2	0.02
		1737N-W3	0.19			1734S-W3	0.19
		1737N-W4	0.35			1734S-W4	0.15
		1737N-E1	0.08				
		1737N-E2	0.08				
		1737N-E3	0.00				

3.3.2 Chloride Content Testing

Acid-soluble chloride content was evaluated for all forty-six (46) core samples in general accordance with the procedures described in ASTM C1152 *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete* [12]. Acid-soluble testing was performed in lieu of water-soluble testing for a consistent bases of comparison with the chloride data obtained as part of the 1998 study. Acid-soluble testing provides total chloride content, not just the water-soluble concentration. Each core was tested at five depth increments by cutting precise slices from the cores which were then separately pulverized into fine powder samples for testing. The five depth increments which were sliced match the sampling depths tested in the 1998 study - 0-0.5 in. (0-1.25 cm), 0.5-1 in. (1.25-2.5 cm), 1-1.5 in. (2.5-3.75 cm), 1.5-2.5 in. (3.75-6.25 cm), and 2.5-3.5 in. (6.25-8.75 cm) - measured from the exterior faces of the concrete elements.

The chloride content test results are summarized in Appendix A, alongside the results of pre-ECE and post-ECE sampling performed as part of the 1998 study. Changes in chloride contents between the samples collected after ECE treatment and those collected as part of the 2018 study are calculated and highlighted by red (increase) or green (decrease) backgrounds. Note that some locations that were tested in the 1998 study were not included in the 2018 study due to budgetary considerations. These locations are identified with not-applicable (N/A) entries for both the chloride level and percent change.

Significant increases in chloride content compared to the post-ECE levels were identified at almost all depths of almost all forty-six core sampling locations. The chloride concentrations in all five piers were high, and typically exceeded the corrosion threshold (0.035 percent by weight of concrete), particularly in the pier caps. The increases occurred regardless of whether or not ECE treatment was performed as part of the 1998 study, and regardless of which type of surface protection was installed, if any. Additional analyses of these findings is presented in Chapter 4.

CHAPTER 4: ANALYSIS AND DISCUSSION

4.1 CONCRETE DISTRESS AND CORROSION ACTIVITY: 20-YEAR REVIEW

4.1.1 Overview

The 1998 study included five piers that were approximately 30 years old. At that time, concrete distress conditions had developed in all five pier caps and eleven of the twelve columns. Field and laboratory testing identified probable corrosion activity and chloride levels near or in excess of the corrosion threshold around almost all areas of distress. In total, approximately 485 square feet of concrete repair was performed at the distress locations. This value was derived by the authors by scaling the published graphical representations of repair locations from 1998. Using this information, “distress ratios” were calculated for each treatment study section, which consists of the treated portion of the pier cap and the associated column. The distress ratio is defined as the area of surface distress divided by the total surface area of the study section. Distress ratios at the time of the 1998 study ranged from 5 percent (such as at the north end of Pier 37EB) up to 21 percent (at the south end of Pier 34EB).

The 2018 study evaluated the condition of the five piers after 20 additional years of service. Distress was identified by visual inspection and hammer sounding, and half-cell potential testing was performed at four of five piers. Significantly less concrete distress was identified than was present at the onset of the 1998 study. The total quantity of surface distress was approximately 220 square feet, a reduction of over 55 percent, and was almost entirely at or adjacent to locations of 1998 patch repairs. While corrosion activity recurred in some areas, the majority of the patch repairs performed well. Five different treatment sections exhibited no new distress in either the pier cap or the column. The highest 2018 distress ratio was 18 percent and occurred at the south end of Pier 40WB, which was a control section. However, areas of potential corrosion activity were identified in all four piers where half-cell potential testing was performed. These areas were generally more localized, and exhibited less negative corrosion potential, than was identified in the 1998 study, but exhibited good correlation with the observed locations of damage.

4.1.2 Findings

Comparison of total distress conditions at the piers would suggest similar overall levels of durability whether ECE treatment was performed or not. Pier 34WB and Pier 37WB exhibited approximately 255 square feet of distress (distress ratio of 12 percent) before ECE treatments were applied. The 2018 study identified approximately 110 square feet of total distress at these two piers (distress ratio of 5 percent). Pier 34EB, Pier 37EB and Pier 40WB did not receive ECE treatment. The six sections of these piers included in the 1998 study exhibited approximately 230 square feet of distress (distress ratio of 11 percent) at the onset of the 1998 study. The 2018 study identified approximately 110 square feet of total distress (distress ratio of 5 percent), the same quantity and ratio as the ECE-treated piers. However, distinctions in performance were apparent within this data for the different sections and different types of surface protection which were installed. See Figure 4.1.

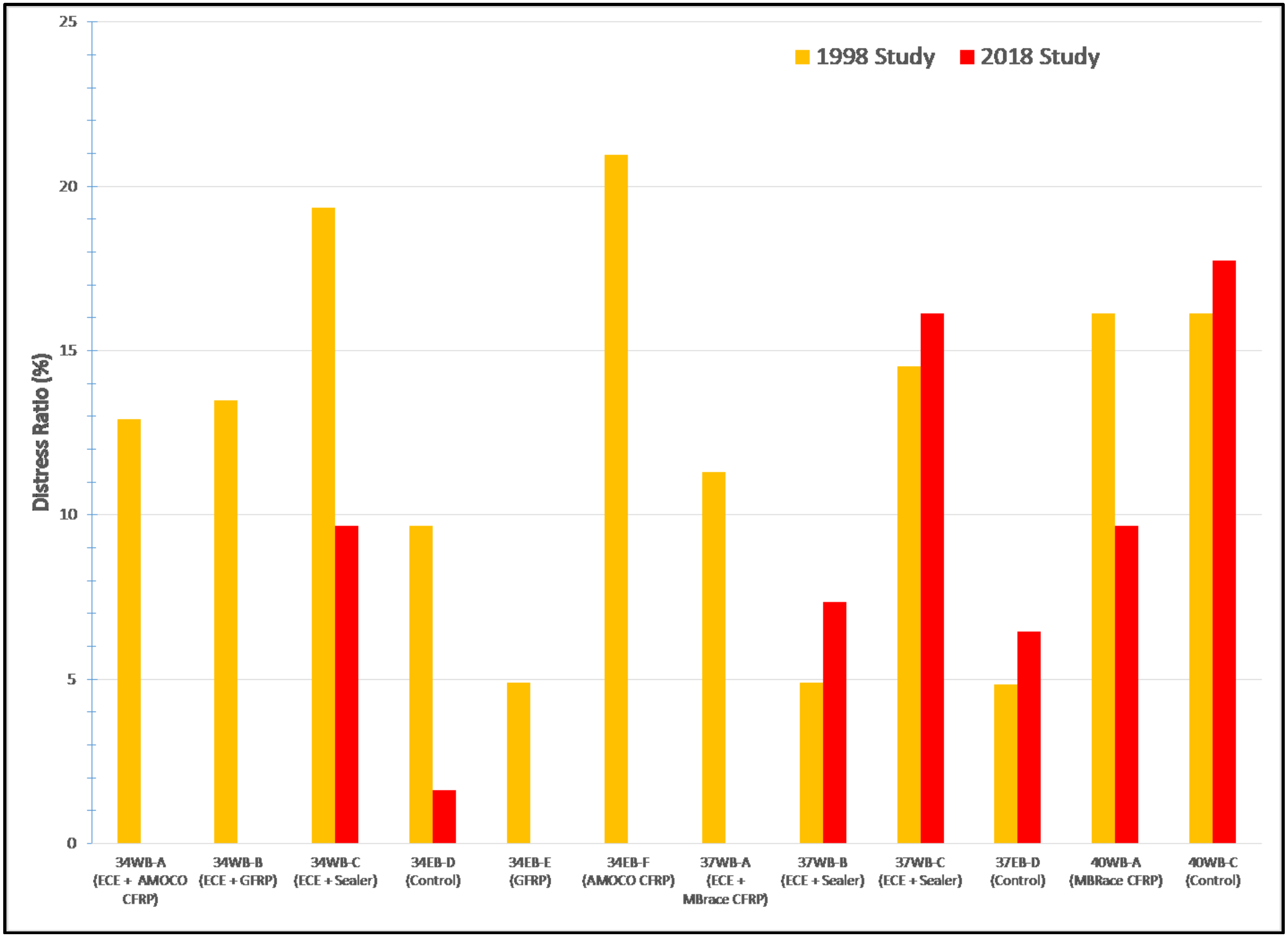


Figure 4.1 Comparison of distress ratio by section - 1998 vs 2018

No concrete distress and no areas of probable corrosion activity (half-cell potential more negative than -350mV) were identified in any of the three sections which received ECE treatment and were subsequently covered with any of the three different FRP wrap products - AMOCO CFRP, GFRP or MBrace CFRP. This included the north end and middle of Pier 34WB and the north end of Pier 37WB. Approximately 130 square feet of distress (distress ratio of 13 percent) was repaired as part of the 1998 study in these sections. No distress recurred and no new distress developed after 20 additional years of service. These findings were even more noteworthy considering the fire damage sustained in 2007 which resulted in the blistering and debonding of the majority of the FRP wrap material installed on the east face of the 34WB pier cap. However, ten half-cell potential measurements which are indicative of possible corrosion activity (between -200 mV and -350 mV) were obtained at the north end of Pier 37WB, where MBrace CFRP was installed. Eight of these readings were concentrated at the north end of the pier cap, and two readings were located at the base of Column 37A.

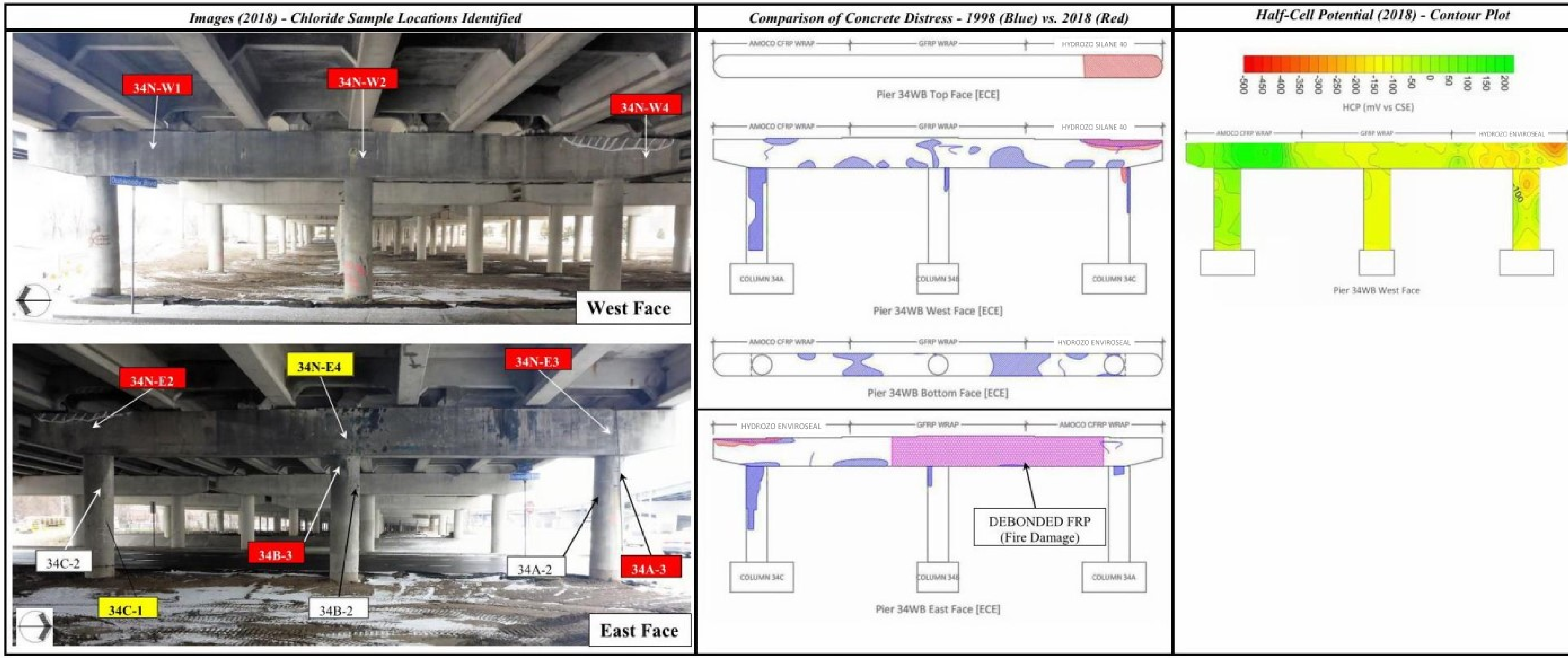
The south end of Pier 34WB, and the middle and south end of Pier 37WB, also received ECE treatment as part of the 1998 study. However, different concrete sealers were applied instead of FRP wraps. The total quantity of distress identified in these three coated sections as part of the 2018 study (110 square feet) was only slightly less than that which was present in 1998 (120 square feet), and the distress ratios at the middle and south end sections of Pier 37WB actually *increased*. Areas of possible or probable corrosion activity were identified in the pier caps in all three sections, as well as the top of Column 37C at Pier 37WB. The locations of distress and likely corrosion activity were generally consistent with the locations of 1998 corrosion damage repair.

Six pier sections did *not* receive ECE treatment as part of the 1998 study. FRP wraps were installed at the middle and south end of Pier 34EB, and the north end of Pier 40WB. At the other three sections - the north ends of Pier 34EB and Pier 37EB, and the south end of Pier 40WB - no surface protection was installed. Somewhat mixed performance was observed within these six areas.

At the time of the 1998 study, approximately 85 square feet of distress was repaired at the middle and south end sections of Pier 34EB before GFRP and AMOCO CFRP wraps, respectively, were installed. No distress was identified in either section in the 2018 study. No obvious evidence of significant or long-standing joint leakage problems were noted at these two sections during the field inspection. Half-cell potential testing may have identified possible corrosion activity at Pier 34EB behind the AMOCO CFRP, although the moderate risk corrosion potentials which were measured were relatively uniform and may instead reflect low oxygen availability at the reinforcing. Unlike those two sections, however, corrosion damage did redevelop behind the MBrace CFRP at the north end of Pier 40WB. Sounding performed by Collins Engineers identified approximately 30 square feet of unsound concrete behind the FRP. Corrosion potentials indicative of probable corrosion activity were measured in the vicinity of the areas of unsound concrete. The most negative half-cell potentials were concentrated near the edge termination of the FRP wrap, adjacent to the middle section of the pier cap which was not included in the 1998 study and received no surface treatments. Evidence of water leakage through the deck joint was observed in this area, including visible wetness on the surface of the FRP and pier cap, and daylight was visible through the north end of the joint directly above the nose of the pier cap.

Somewhat variable performance was also observed at the three control sections. Only a small region of distress (5 square feet) redeveloped at the north end of Pier 34EB. Approximately 30 square feet of distress had been present in this section at the time of the 1998 study. The recurrent distress was positioned along the top edge of the pier cap, in an area where evidence of strip seal joint leakage (wetness) was observed during the field survey. Despite the distress and exposure, half-cell potentials in the pier cap and column were passive. In contrast, the total quantity of distress identified in the 2018 study at the north end of Pier 37EB (20 square feet) and south end of Pier 40WB (55 square feet) represented increases of approximately 15 percent in comparison to the distress levels which existed in 1998. The distress at Pier 37EB includes new areas which were not present in 1998. Half-cell potential testing identified possible corrosion activity at the south end section of Pier 40WB in the area of visible distress. Although half-cell potential testing was not performed at Pier 37EB, the distress conditions are consistent with corrosion activity.

Refer to Figures 4.2 to 4.6 for summary graphics of each of the five pier caps. These graphics include images of each pier face obtained by Collins Engineers in 2017, alongside wire frame illustrations of the concrete distress. Concrete repair locations associated with the 1998 study are shown in blue. Areas of unsound concrete identified in the 2018 hammer sounding survey are shown in red. Contour plots of half-cell potentials measured in the 2018 study, if applicable, are also presented at the far right. Below the graphics, two tables present chloride data and distress ratios from the 1998 study and 2018 study. The upper table tabulates the chloride concentrations measured at the depth of the reinforcing steel. The chloride concentrations include pre-ECE data from 1997, post-ECE data from 1998 and 2018 data. Values in excess of the corrosion threshold are highlighted in red. The approximate locations of the chloride sampling are identified on the pier face images. Red or yellow designations represent locations with chloride concentration above threshold (0.035 percent by weight of concrete) or near threshold (0.03 to 0.0349 percent by weight of concrete), respectively, at the depth of the steel. The lower table presents the 1998 and 2018 distress ratios which were calculated by the authors for each treatment study section, with the applicable treatments received by that section noted for reference.



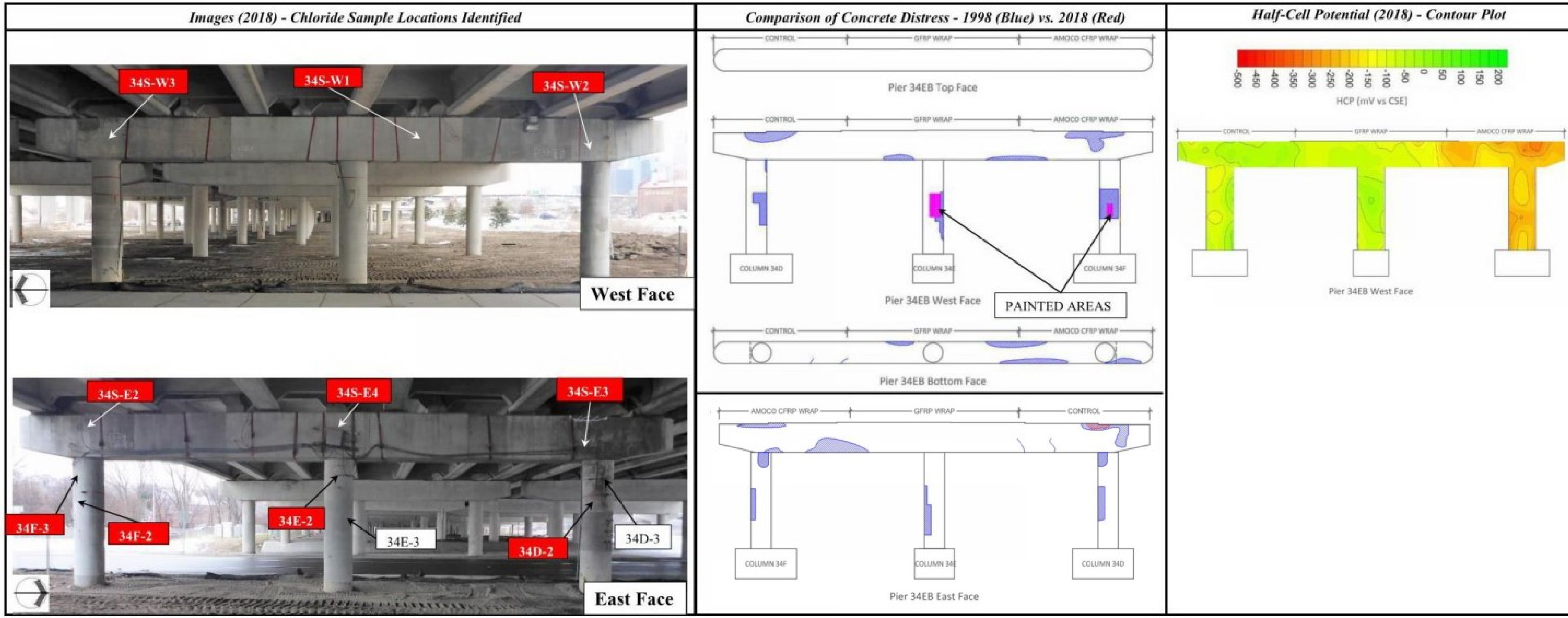
20 Year Review - Comparison of Chloride Content at Depth of Reinforcing Steel (1.5 - 2.5") vs. Corrosion Threshold (0.035% by weight of concrete)

Sample Location	Pier Cap						Column A		Column B		Column C	
Sample ID	34N-W1	34N-W2	34N-W4	34N-E2	34N-E3	34N-E4	34A-2	34A-3	34B-2	34B-3	34C-1	34C-2
1998 - Pre-ECE	0.017	0.029	0.009	N/A	N/A	N/A	0.021	0.050	0.023	0.027	0.027	0.027
1998 - Post-ECE	0.010	0.033	0.008	0.022	0.012	0.006	0.014	0.022	0.003	0.021	0.012	0.003
Surface Treatment	AMOCO	GFRP	Sealer	Sealer	AMOCO	GFRP	AMOCO	AMOCO	GFRP	GFRP	Sealer	Sealer
2018	0.080	0.052	0.074	0.135	0.163	0.030	0.024	0.194	0.016	0.103	0.032	0.010

20 Year Review - Comparison of Concrete Surface Distress

Element	*Locations*		*Corrosion Mitigation Strategy*			*Concrete Surface Distress*			
						1998 [Pre-ECE]		2018	
	Pier Cap	Column	ECE	FRP	Surface Treatment Type	Approximate Quantity (sf)	Distress Ratio (%)	Approximate Quantity (sf)	Distress Ratio (%)
Pier 34WB	North End	A	Yes	Yes	CFRP	40	13	0	0
	Middle	B	Yes	Yes	GFRP	55	13	0	0
	South End	C	Yes	No	Sealer	60	19	30	10

Figure 4.2 Pier 34WB [ECE] - Summary of Findings



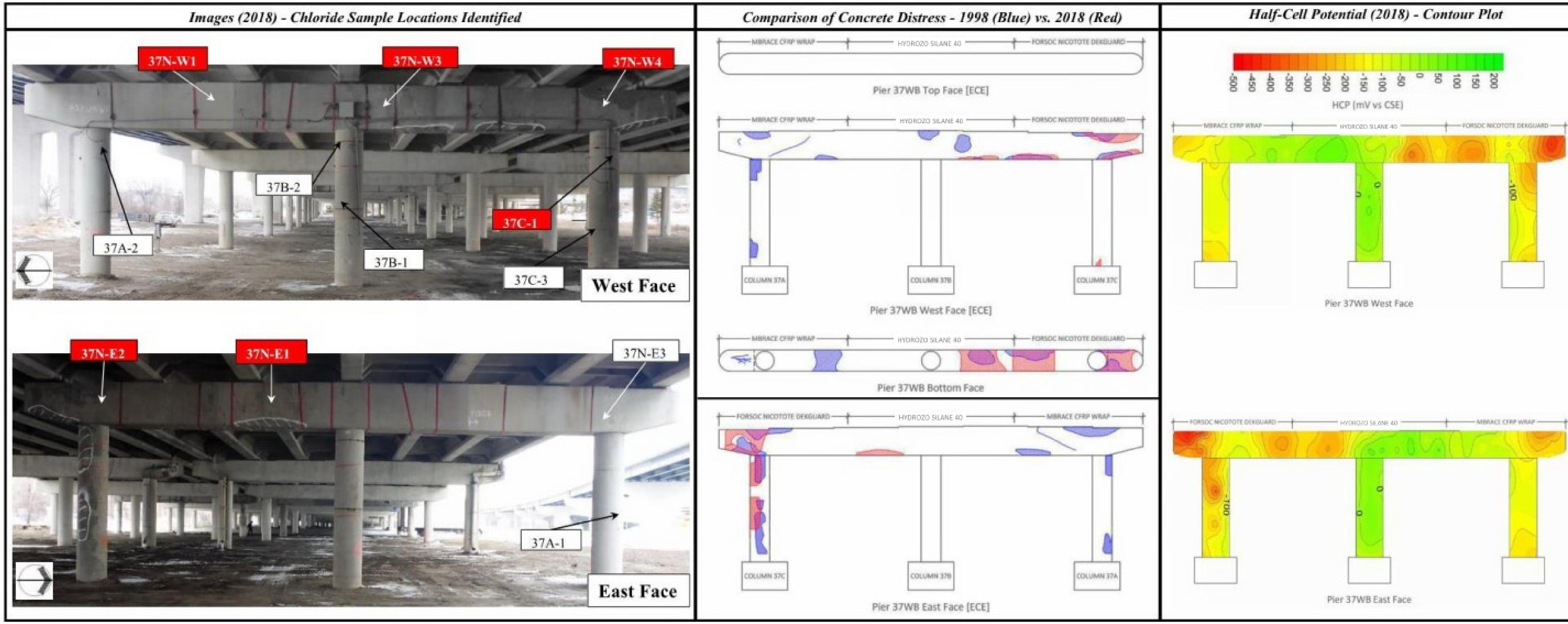
20 Year Review - Comparison of Chloride Content at Depth of Reinforcing Steel (1.5 - 2.5") vs. Corrosion Threshold (0.035% by weight of concrete)

Sample Location	Pier Cap						Column D		Column E		Column F	
Sample ID	34S-W1	34S-W2	34S-W3	34S-E2	34S-E3	34S-E4	34D-2	34D-3	34E-2	34E-3	34F-2	34F-3
1998 - Pre-ECE	0.006	0.022	0.016	0.016	0.014	0.016	N/A	N/A	N/A	0.002	0.022	N/A
1998 - Post-ECE	0.005	0.020	0.010	0.031	0.007	0.004	0.002	0.007	0.006	0.003	0.028	0.018
Surface Treatment	GFRP	AMOCO	None	AMOCO	None	GFRP	AMOCO	AMOCO	GFRP	GFRP	None	None
2018	0.039	0.158	0.062	0.205	0.050	0.036	0.089	0.004	0.061	0.008	0.142	0.185

20 Year Review - Comparison of Concrete Surface Distress

Element	*Locations*		*Corrosion Mitigation Strategy*			*Concrete Surface Distress*			
	Pier Cap	Column	ECE	FRP	Surface Treatment Type	1998 [Pre-ECE]		2018	
						Approximate Quantity (sf)	Distress Ratio (%)	Approximate Quantity (sf)	Distress Ratio (%)
Pier 34EB	North End	D	No	No	None	30	10	5	2
	Middle	E	No	Yes	GFRP	20	5	0	0
	South End	F	No	Yes	CFRP	65	21	0	0

Figure 4.3 Pier 34EB [Non-ECE] - Summary of Findings



20 Year Review - Comparison of Chloride Content at Depth of Reinforcing Steel (1.5 - 2.5") vs. Corrosion Threshold (0.035% by weight of concrete)

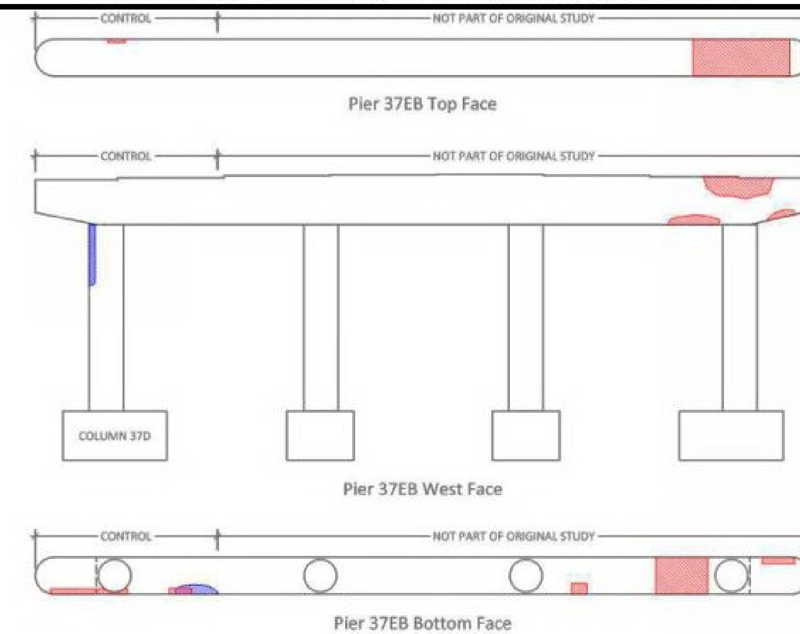
Sample Location	Pier Cap						Column A		Column B		Column C	
Sample ID	37N-W1	37N-W3	37N-W4	37N-E1	37N-E2	37N-E3	37A-1	37A-2	37B-1	37B-2	37C-1	37C-3
1998 - Pre-ECE	0.021	0.006	0.018	0.040	0.041	0.011	0.002	0.005	0.002	0.035	0.004	0.001
1998 - Post-ECE	0.006	0.008	0.007	0.016	0.014	0.008	0.004	0.007	0.004	0.015	0.009	0.001
Surface Treatment	MBrace	Silane	Fosroc	Silane	Fosroc	MBrace	MBrace	MBrace	Silane	Silane	Fosroc	Fosroc
2018	0.128	0.039	0.094	0.108	0.113	0.022	0.009	0.008	0.008	0.008	0.129	0.018

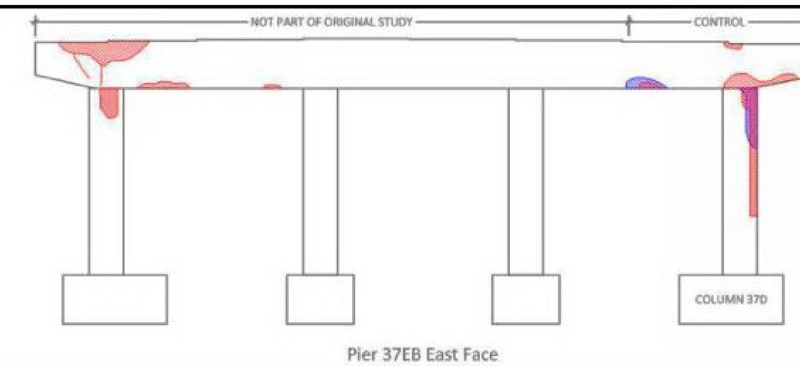
20 Year Review - Comparison of Concrete Surface Distress

Element	*Locations*		*Corrosion Mitigation Strategy*			*Concrete Surface Distress*			
						1998 [Pre-ECE]		2018	
	Pier Cap	Column	ECE	FRP	Surface Treatment Type	Approximate Quantity (sf)	Distress Ratio (%)	Approximate Quantity (sf)	Distress Ratio (%)
Pier 37WB	North End	A	Yes	Yes	CFRP	35	11	0	0
	Middle	B	Yes	No	Sealer	20	5	30	7
	South End	C	Yes	No	Sealer	45	15	50	16

Figure 4.4 Pier 37WB [ECE] - Summary of Findings

Images (2018) - Chloride Sample Locations Identified	*Comparison of Concrete Distress - 1998 (Blue) vs. 2018 (Red)*	*Half-Cell Potential (2018) - Contour Plot*





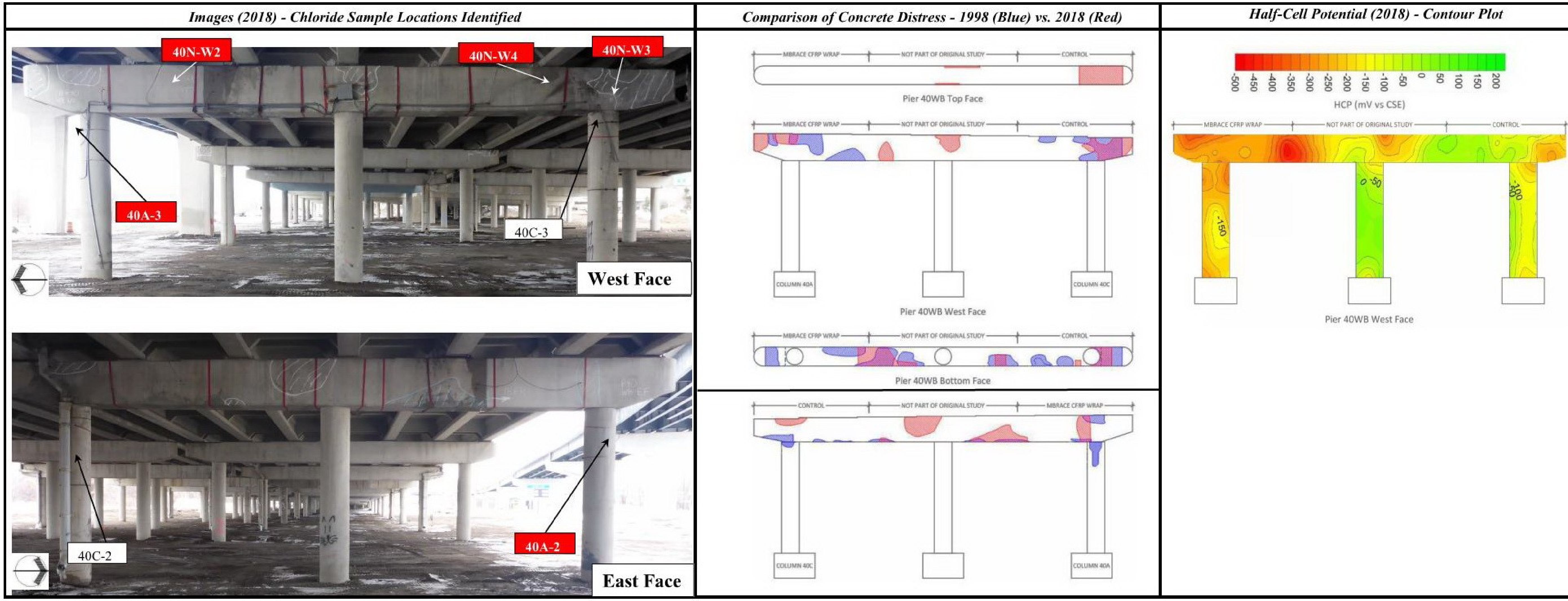
20 Year Review - Comparison of Chloride Content at Depth of Reinforcing Steel (1.5 - 2.5") vs. Corrosion Threshold (0.035% by weight of concrete)

Sample Location	**Pier Cap**	**Column D**	
Sample ID	**37S-W1**	**37D-1**	**37D-2**
1998 - Pre-ECE	0.015	0.002	0.038
1998 - Post-ECE	0.006	0.002	0.031
Surface Treatment	None	None	None
2018	0.018	0.015	0.192

20 Year Review - Comparison of Concrete Surface Distress

Element	*Locations*		*Corrosion Mitigation Strategy*			*Concrete Surface Distress*			
						1998 [Pre-ECE]		**2018**	
	Pier Cap	**Column**	**ECE**	**FRP**	**Surface Treatment Type**	Approximate Quantity (sf)	Distress Ratio (%)	Approximate Quantity (sf)	Distress Ratio (%)
Pier 37EB	North End	D	No	No	None	15	5	20	6
	Middle North	E	Not Included in Study						
	Middle South	F	Not Included in Study						
	South End	G	Not Included in Study						

Figure 4.5 Pier 37EB [Non-ECE] - Summary of Findings



20 Year Review - Comparison of Chloride Content at Depth of Reinforcing Steel (1.5 - 2.5") vs. Corrosion Threshold (0.035% by weight of concrete)

Sample Location	Pier Cap			Column A		Column C	
Sample ID	40N-W2	40N-W3	40N-W4	40A-2	40A-3	40C-2	40C-3
1998 - Pre-ECE	0.017	0.021	0.013	0.003	0.015	0.002	0.002
1998 - Post-ECE	0.021	0.013	0.004	0.008	0.019	0.006	0.003
Surface Treatment	MBrace	None	None	MBrace	MBrace	None	None
2018	0.179	0.183	0.065	0.079	0.298	0.008	0.012

20 Year Review - Comparison of Concrete Surface Distress

Element	Locations		Corrosion Mitigation Strategy			Concrete Surface Distress			
	Pier Cap	Column	ECE	FRP	Surface Treatment Type	1998 [Pre-ECE] Approximate Quantity (sf)	1998 [Pre-ECE] Distress Ratio	2018 Approximate Quantity (sf)	2018 Distress Ratio
Pier 40WB	North End	A	No	Yes	CFRP	50	16	30	10
	Middle	B	Not Included in Study						
	South End	C	No	No	Control	50	16	55	18

Figure 4.6 Pier 40WB [Non-ECE] - Summary of Findings

4.2 CHLORIDE CONTAMINATION: 20-YEAR REVIEW

4.2.1 Overview

Laboratory testing of acid-soluble chloride content was performed as a component of the 1998 study and the 2018 study to evaluate the extent of chloride contamination which existed in the five piers selected for research. Initial testing work was performed in fall 1997 prior to the ECE treatment phase and included the collection of concrete samples at 68 different locations, generally three locations per column and eight locations per pier cap. Locations of sampling consisted of visibly sound concrete either adjacent to, or away from, areas of distress and repair. In August 1998, shortly after the ECE treatment phase concluded, sampling and chloride testing work was repeated using the same procedures as the 1997 pre-ECE testing but with sample locations adjusted slightly to avoid re-drilling repaired holes. The 2018 study included the extraction and testing of concrete samples adjacent to 46 of the 1998 post-ECE sampling locations to evaluate the changes in chloride levels after 20 additional years of service.

Refer to Appendix A for a complete side-by-side tabulation of all of the pre-ECE, post-ECE and 2018 chloride content testing results. For elements which received ECE treatment, changes in chloride contents between the pre-ECE and post-ECE samples are calculated and highlighted by red (increase) or green (decrease) backgrounds. Similar evaluations are highlighted for changes in chloride content between the 1998 post-ECE and 2018 chloride levels. All chloride levels in excess of the chloride concentration are identified in red font. Where the sampling depth increment is aligned with the depth of the reinforcing steel, a bold outline accompanies the chloride concentration values.

4.2.2 Findings

In fall 1997, prior to the ECE treatment phase, almost 40 percent of the powder samples which were processed indicated a chloride content in excess of the corrosion threshold (0.035 percent by weight of concrete). The majority of the elevated chloride levels were confined to the outer 1 inch nearest the concrete surface. The chloride content levels were typically below the corrosion threshold at the depth of the reinforcing steel. Pier 34WB exhibited more severe contamination, including chloride levels in excess of the corrosion threshold at the depth of the reinforcing steel, at several locations in the columns and pier cap.

As a result of ECE treatment, a significant reduction in chloride content was observed at the majority of sampling locations and depth increments (also termed horizons) in Pier 34WB and Pier 37WB. On average, the chloride content of each horizon was reduced by approximately 45 percent. The number of horizons with chloride levels above the corrosion threshold was reduced from 54 percent to 8 percent at Pier 34WB, and from 16 percent to 2 percent at Pier 37WB. The post-ECE chloride concentration at the depth of the reinforcing steel was below the corrosion threshold at all sampling locations in these two piers. At the three piers which did not receive ECE treatment, approximately 13 percent of the post-ECE samples possessed a chloride concentration in excess of the corrosion threshold. However, only one location (40C-1) possessed chloride levels in excess of the corrosion threshold at the depth of the reinforcing steel.

Acid-soluble chloride testing performed as part of the 2018 study identified significant increases in chloride content at almost all horizons of almost all forty-six core locations. The increases occurred regardless of whether or not ECE treatment was performed, and regardless of the 1998 surface protection application. The testing results indicate that none of the surface protection measures were effective at preventing new chloride ingress since 1998. In summary:

- Chloride concentrations exceeded the corrosion threshold in 68 percent (157 of 230) of the horizons which were tested in the 2018 study, up from only 9.6 percent (35 of 365 horizons) of the 1998 post-ECE sample population. This included 77 percent of the horizons evaluated from Pier 34WB, 75 percent at Pier 34EB, 53 percent at Pier 37WB, 66 percent at Pier 37EB, and 69 percent at Pier 40WB.
- Twenty-nine of the forty-six coring locations (63 percent) which were re-evaluated in the 2018 study exhibited a chloride concentration in excess of the corrosion threshold at the depth of reinforcing steel. The 1998 post-ECE chloride concentration was below the corrosion threshold at the depth of the steel at all of these locations.
- Chloride concentrations exceeded the corrosion threshold at the depth of the reinforcing in 19 of 22 core samples (86 percent) which were obtained from pier caps in the 2018 study.
- At Pier 34WB and 37WB, which received ECE treatment, 13 of 24 core locations exhibited chloride concentrations at the depth of the reinforcing steel in excess of the corrosion threshold, up from 0 percent of the 1998 post-ECE samples.
- The percentage of horizons with chloride concentrations in excess of the corrosion threshold was 76 percent (87 of 115) at locations which received FRP wrap, 62 percent (37 of 60) at locations which received a sealer, and 55 percent (30 of 55) at the control sections.
- The percentage of horizons with chloride concentrations in excess of the corrosion threshold was 89 percent (31 of 35) at locations which received AMOCO CFRP wrap, 71 percent (25 of 35) at locations which received GFRP wrap, and 63 percent (30 of 55) at locations which received MBrace CFRP wrap.

The extent of chloride contamination in the five piers which were included in this study has substantially increased since 1998, particularly in the pier caps. These findings were unexpected because the bridge deck joints had generally been rated in good condition throughout the past 20 years, and were replaced in 2008. While obvious evidence of water leakage through the deck joints was observed in areas of Pier 34WB, Pier 34EB, and Pier 40WB during the 2018 study condition survey, it was not clear that these problems had been longstanding. Evidence of obvious joint leakage was not observed at Pier 37WB or Pier 37EB.

At the three sections which were maintained as controls, no surface protection treatments were installed as a part of the 1998 study. Although different concrete sealers were applied at three other sections, the original treatments presumably lost effectiveness years ago and the products were never reapplied. In theory, the surfaces of pier caps and columns which did not receive any surface protection treatments, or which lost the protection that was installed, would be more vulnerable to the ingress of new chlorides. Significant increases in chloride content could occur with sufficient exposure.

Significant increases in chloride content were also observed at all six sections where different FRP wrap systems were installed in 1998. Laboratory testing performed as part of the 1998 study had demonstrated the GFRP and MBrace CFRP systems, in particular, to be impermeable to moisture. In addition, field bond testing performed as part of the 2018 study found that the FRP wrap remained well-adhered to the concrete surface, excluding the fire damaged region at the east face of Pier 34WB. However, the 1998 FRP wrap installations did not cover the bottom pier cap surfaces and the column bases were also left exposed. Sealant was applied along edge terminations of the FRP wrap at the perimeter of all precast girder bearing seats (top surface of pier cap), at butt joint transitions to different FRP or surface protection systems (top, side and bottom surfaces of pier caps), along the drip extensions (bottom surface of pier caps), and at the tops of the columns. Sealant was also installed where instrumentation probes penetrated the FRP wrap. Refer to Figures 4.7 to 4.9 for examples of these conditions. It is not likely that the sealant provided long term water-tightness at any of these details. Accordingly, and despite significant surface coverage, avenues for moisture and chloride ingress existed both within and beyond the limits of the FRP wrap installations.

Figure 4.7 Pier 40WB [Non-ECE] - No FRP coverage of bottom pier cap surface (2018)

In addition, all of the chloride samples which have been collected from these and other sections were terminated at 3-1/2 inches. The chloride levels deeper in the sections were not evaluated as part of the 1998 study, or the 2018 study, and remain unknown. It is possible that some of the increase in chloride contamination in areas which received FRP wrap are attributable to residual chloride ions in the pier cap and column sections which migrated from beyond the limits of the sampling programs back toward the surface.

Figure 4.8 Pier 37WB [ECE] - Sealant at perimeter of instrument penetration through FRP (2018)

Figure 4.9 Pier 34WB [ECE] - Sealant at precast bearings on top of pier cap (2018)

Other factors associated with the chloride sampling and reporting procedures complicate the direct comparison of 1998 and 2018 chloride data for all five piers. Most significantly, the pre-ECE and post-ECE chloride content testing performed in the 1998 study utilized concrete powder samples. These samples were collected by drilling small (3/4 or 1 inch diameter typically) holes in the concrete and collecting the powder discharged from the hole, below the drill bit, in a plastic bag held at the concrete surface. The drill bit is generally marked so that each depth increment can be drilled accurately, and powder is collected throughout the drilling process within each desired sampling depth. While efficient, the powder sampling technique creates a risk of cross-contamination among the different depths of sampling. The drill bit often rubs the sides of the sample hole while progressing to deeper depths, inadvertently mixing concrete powder from shallower depths with concrete powder from the intended depth of sampling. This can skew the deeper test results in the direction of the outermost concentration levels. In addition, chlorides reside in the paste of the concrete, not the aggregates. The small diameter holes from which powder samples are collected are not likely a good representation of the paste-to-aggregate content of the concrete. Drilling into locations that encounter large aggregate, or large volumes of aggregate, will produce unrepresentatively low chloride results. Conversely, drilling into locations that are predominantly paste volume will produce unrepresentatively high chloride results. The 2018 study evaluated 4-inch diameter concrete cores instead of powder samples. Each core was tested at five depth increments by cutting precise slices from the cores at the desired depths for testing, with each slice then separately pulverized into fine powder samples for acid-soluble chloride testing. Evaluation of core samples allows for a more representative sampling of the paste and aggregates, producing both more reliable and more representative chloride concentration results than powder sampling.

In addition, all of the chloride results obtained in the 1998 study were presented in units of parts per million (ppm) by weight of cement [11]. Chloride results from the 2018 study were provided in units of percent by mass of concrete. The latter approach is typically preferred for data reporting because the mass of a concrete sample can be measured, but the cement content is typically not known. To allow for direct comparisons of the 1998 and 2018 data, the authors manually converted the original results to percent chloride by mass of concrete. This conversion assumed that the original concrete mix included six bags of cement per cubic yard of concrete (i.e., 564 pounds of cement per cubic yard of concrete), and the concrete density was 145 pounds per cubic foot (i.e., 3915 pounds per cubic yard). The 1998 chloride values were then converted using the equation shown below. If the cement content was higher than assumed, all pre-ECE and post-ECE chloride values would be greater that reported herein.

$$CL\ (\%\ by\ mass\ of\ concrete) = \frac{CL(ppm)}{1000000} * \frac{564\ lbs\ cement}{3915 lbs\ concrete} * 100$$

Some variability in results may also be attributable to differences in the sampling locations. All of the core samples which were extracted and tested as part of the 2018 study were located to be as near as possible to the pre-ECE and post-ECE sampling locations. However, identification of those locations was based on the graphical representations of drill hole locations which were included in the 1998 study reports. Particularly at the six sections which received FRP wrap, it is possible that some of the 2018 samples were collected more than several inches away from the locations of 1998 sampling. More generally, concrete is an inhomogeneous material. The extent of chloride exposure and ingress which is experienced by different locations on the pier cap and column surfaces will be non-uniform. As such, samples collected even short distances apart can result in dissimilar chloride concentrations, even if they are collected during the same time period and from the same depth.

In summary, it is clear that significant increases in chloride levels have occurred in all five piers over the past 20 years. Particularly in the pier caps, much higher levels of chloride contamination developed between August 1998 and the 2018 study than had accumulated in the first 30 years the bridge was in service. These results suggest that the substructures have experienced more frequent and extensive exposure to chlorides in the environment in the past 20 years, than in the 30 years prior. Increases in exposure are likely related to procedural changes associated with the application of road salt and anti-icing chemicals on bridge decks which have occurred in Minnesota within the past 20 years. These changes include, but may not necessarily be limited to, more frequent applications of anti-icing (brine) solutions to pre-treat bridge decks in advance of weather events, more frequent applications of deicing chemicals during and following winter weather events, use of chemicals with greater concentrations of sodium chloride, and dispersing higher volumes of deicing chemicals per square foot of bridge deck area [14].

Refer to Figures 4.10 to 4.18 for plots of the chloride concentrations with depth, termed chloride profiles, for each location which was sampled. The plots present all chloride data which was collected as part of the 1998 study (i.e., pre-ECE, post-ECE, or both) and the 2018 study, and are presented in log scale for clarity. Reference lines are included in the plots to identify the specified concrete cover depth of 2 inches, and the corrosion threshold of 0.035 percent chloride by weight of concrete. For the plots associated with the ECE-treated columns and ECE-treated pier caps of Pier 34WB and Pier 37WB, blue shading is used to highlight the change in chloride concentration between the pre-ECE and post-ECE samples.

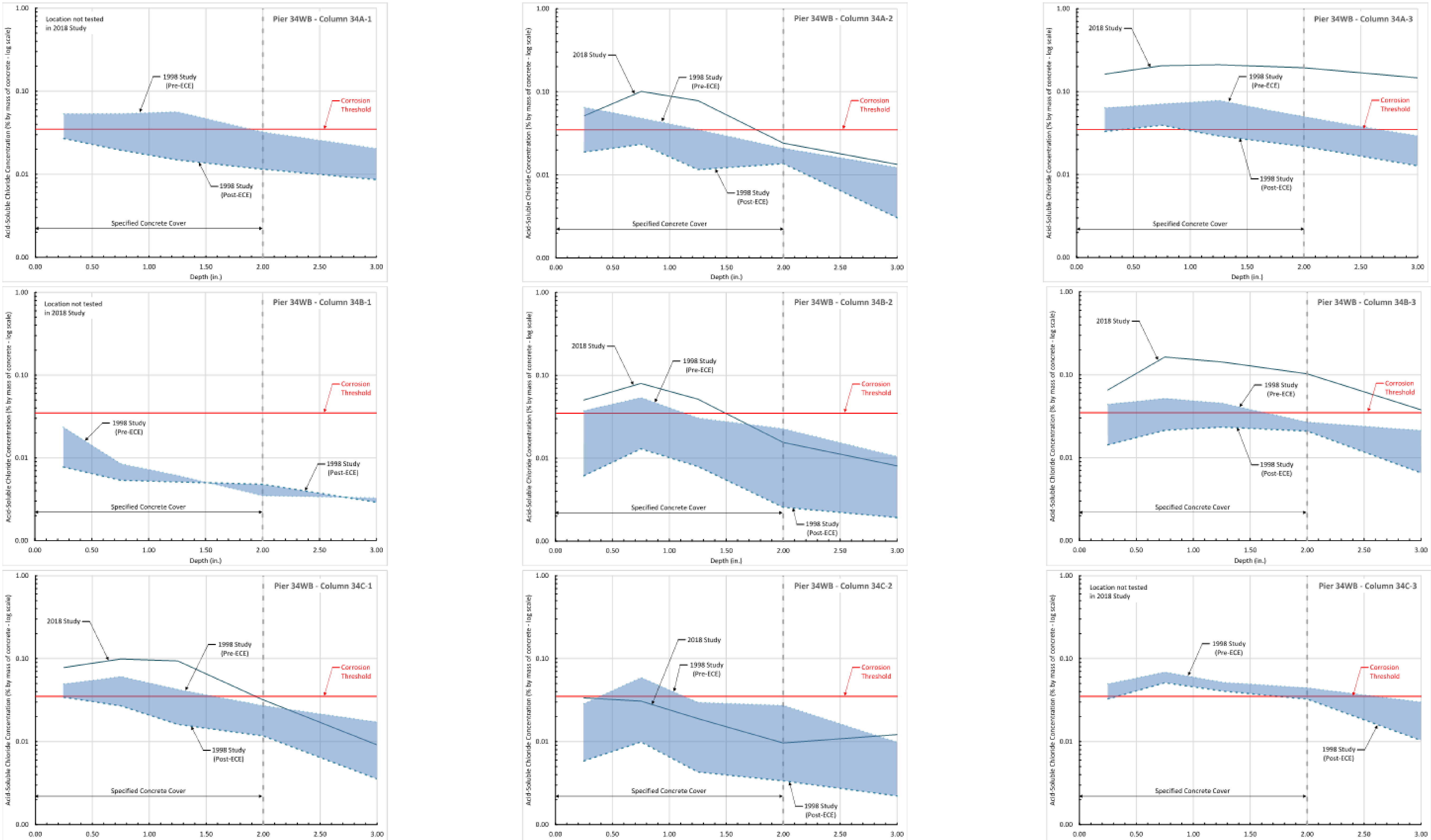


Figure 4.10 Pier 34WB [ECE] - Chloride Profiles - Columns 34A (top), 34B (middle), and 34C (bottom)

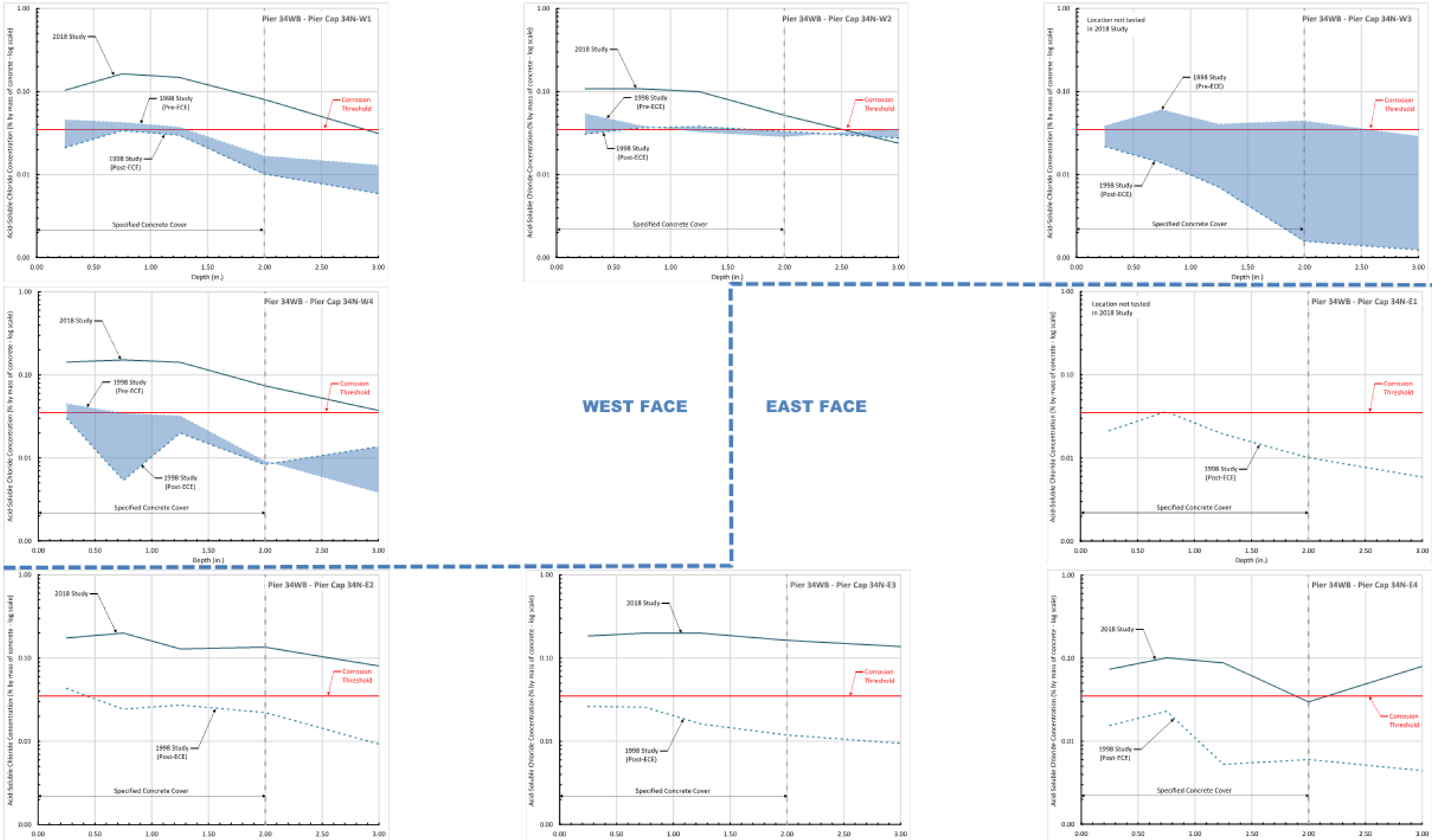


Figure 4.11 Pier 34WB [ECE] - Chloride Profiles - Pier Cap

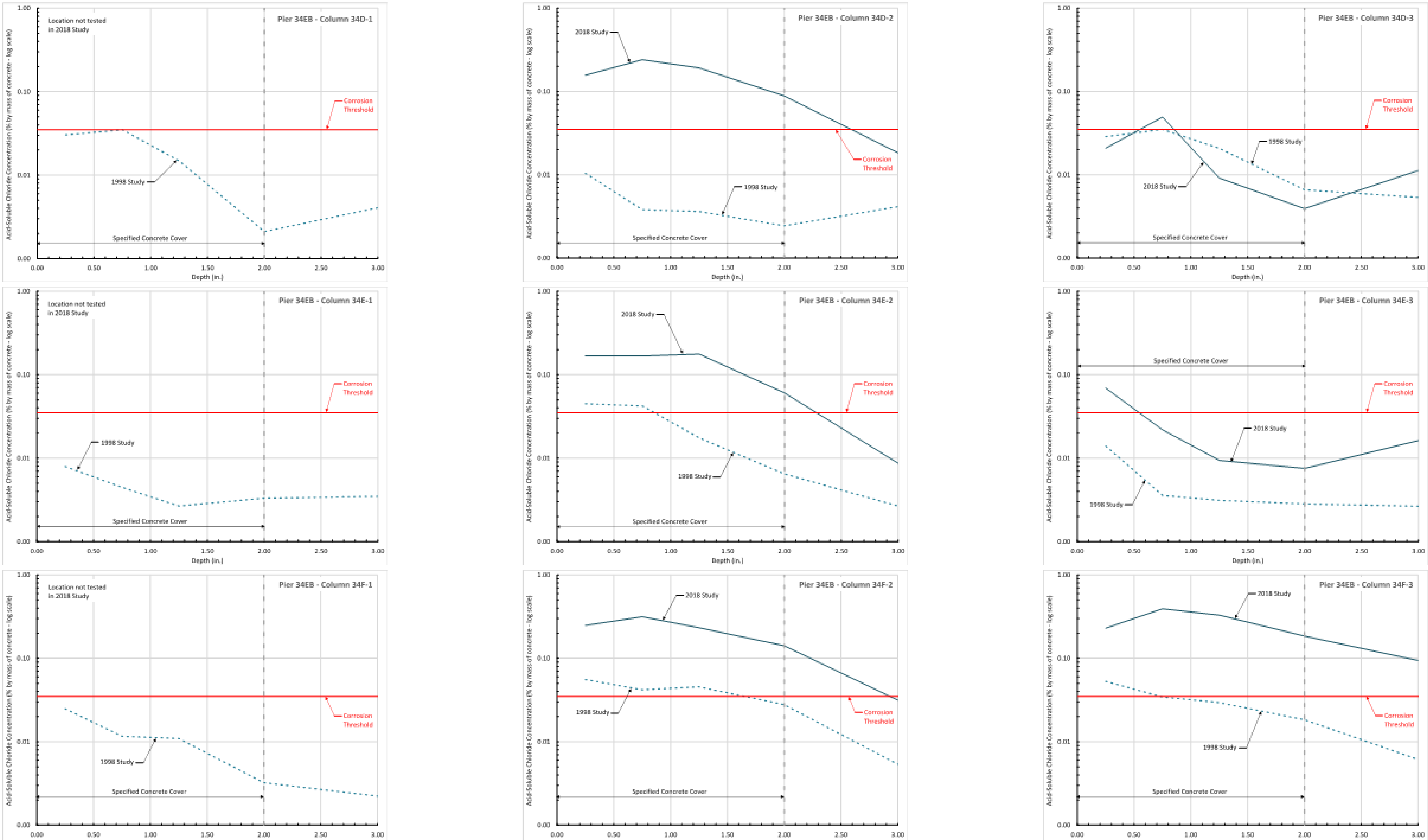


Figure 4.12 Pier 34EB [Non ECE] - Chloride Profiles - Column 34D (top), 34E (middle), and 34F (bottom)

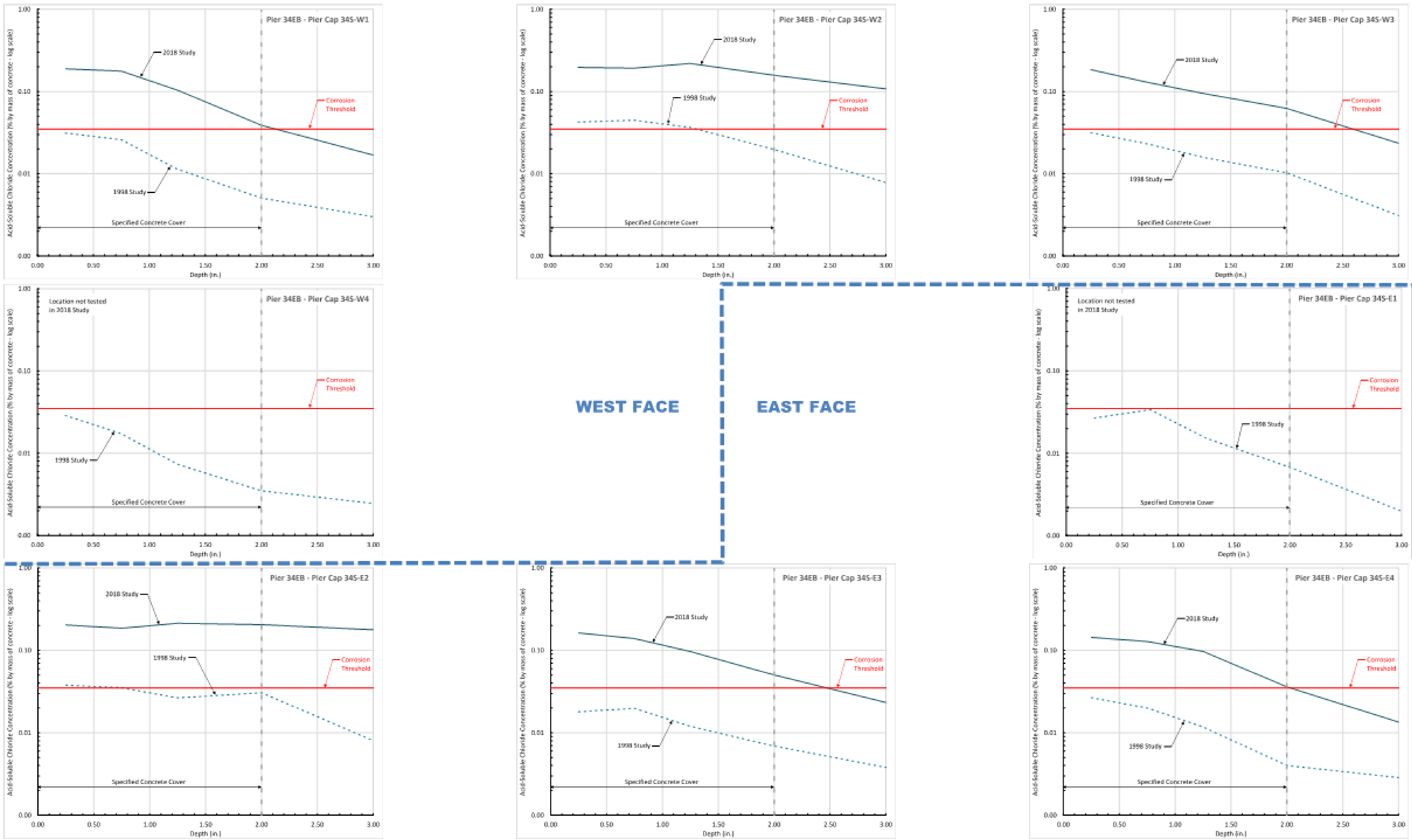


Figure 4.13 Pier 34EB [Non ECE] - Chloride Profiles - Pier Cap

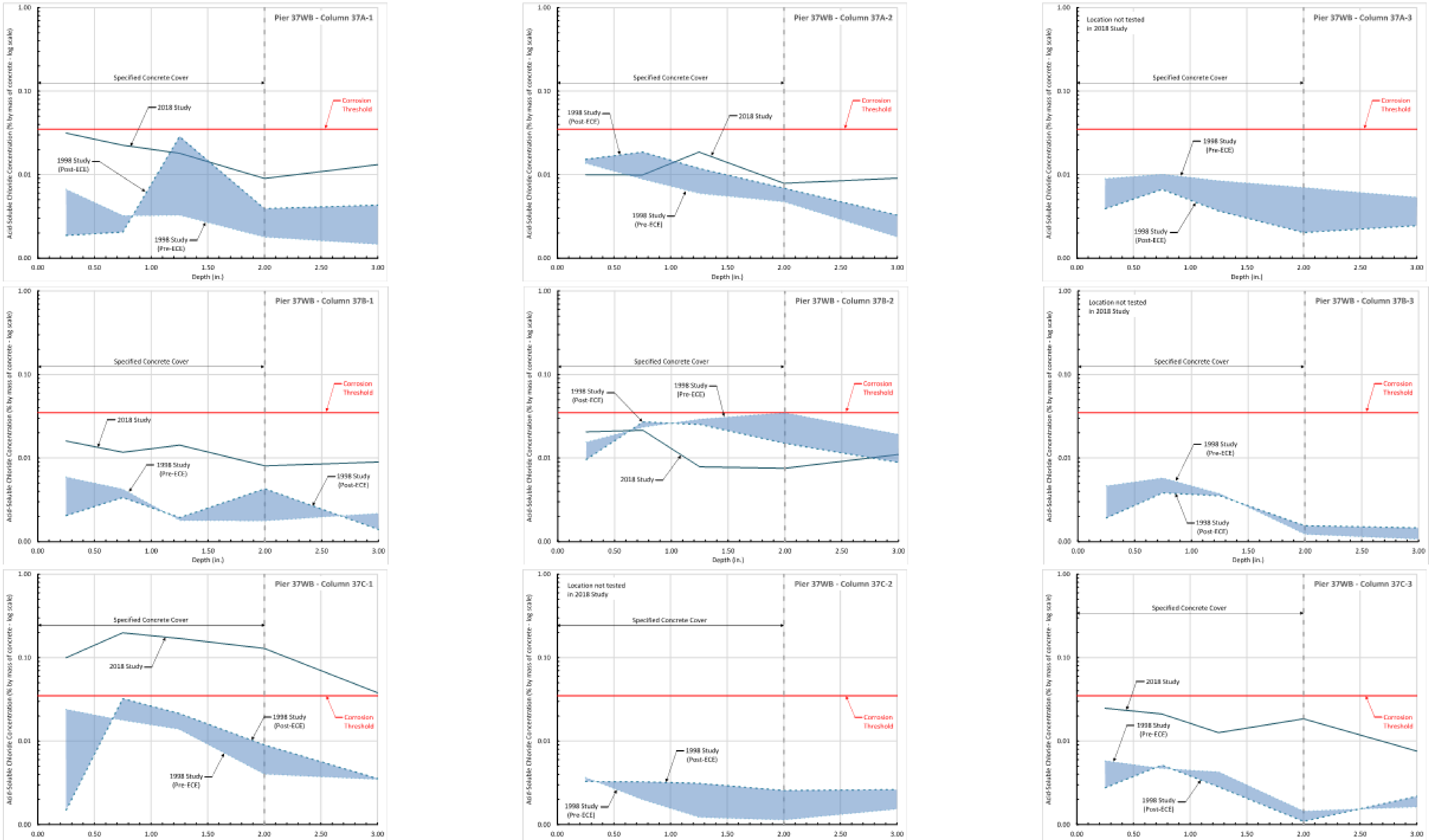


Figure 4.14 Pier 37WB [ECE] - Chloride Profiles - Columns 37A (top), 37B (middle), and 37C (bottom)

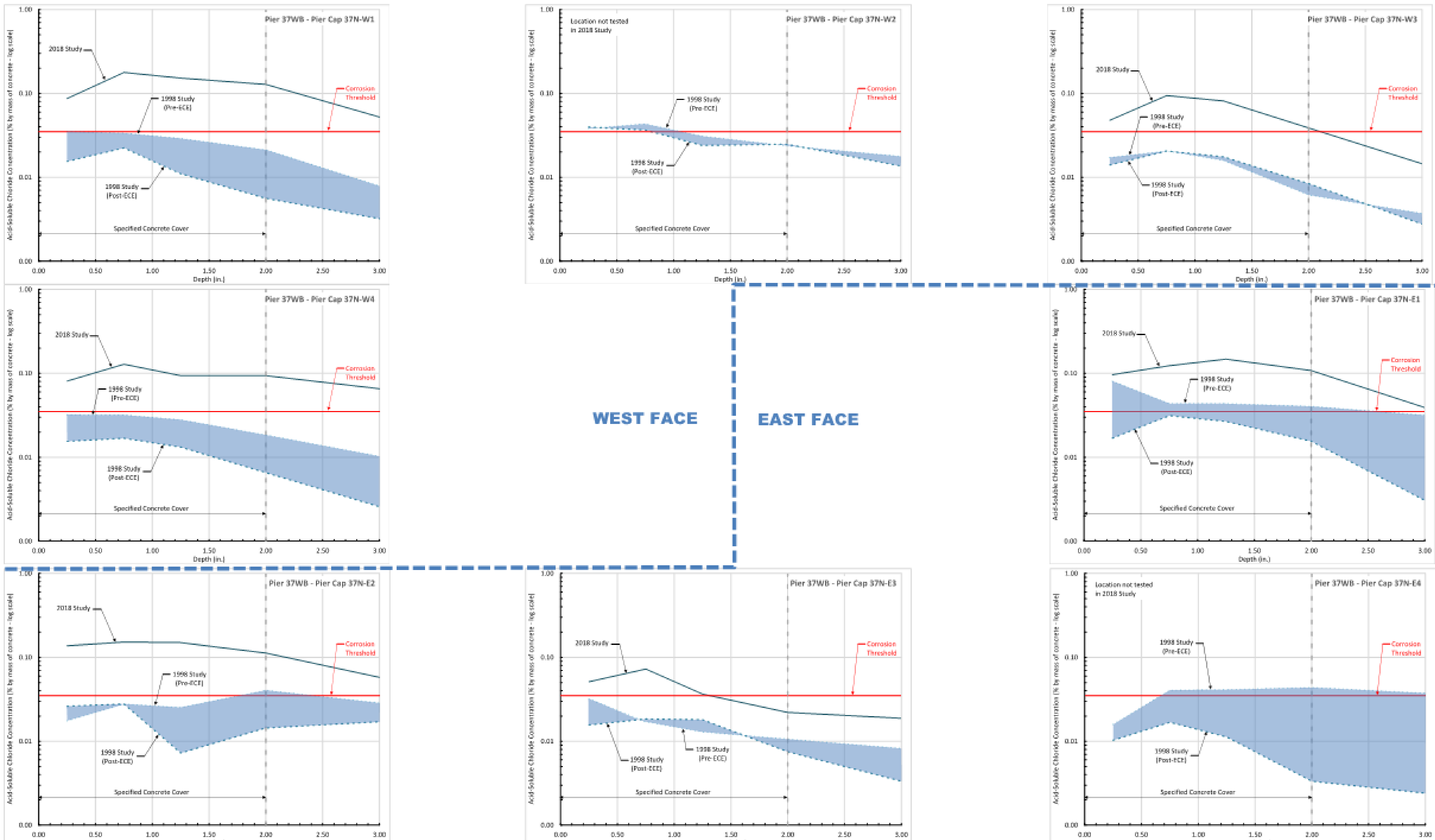


Figure 4.15 Pier 37WB [ECE] - Chloride Profiles - Pier Cap

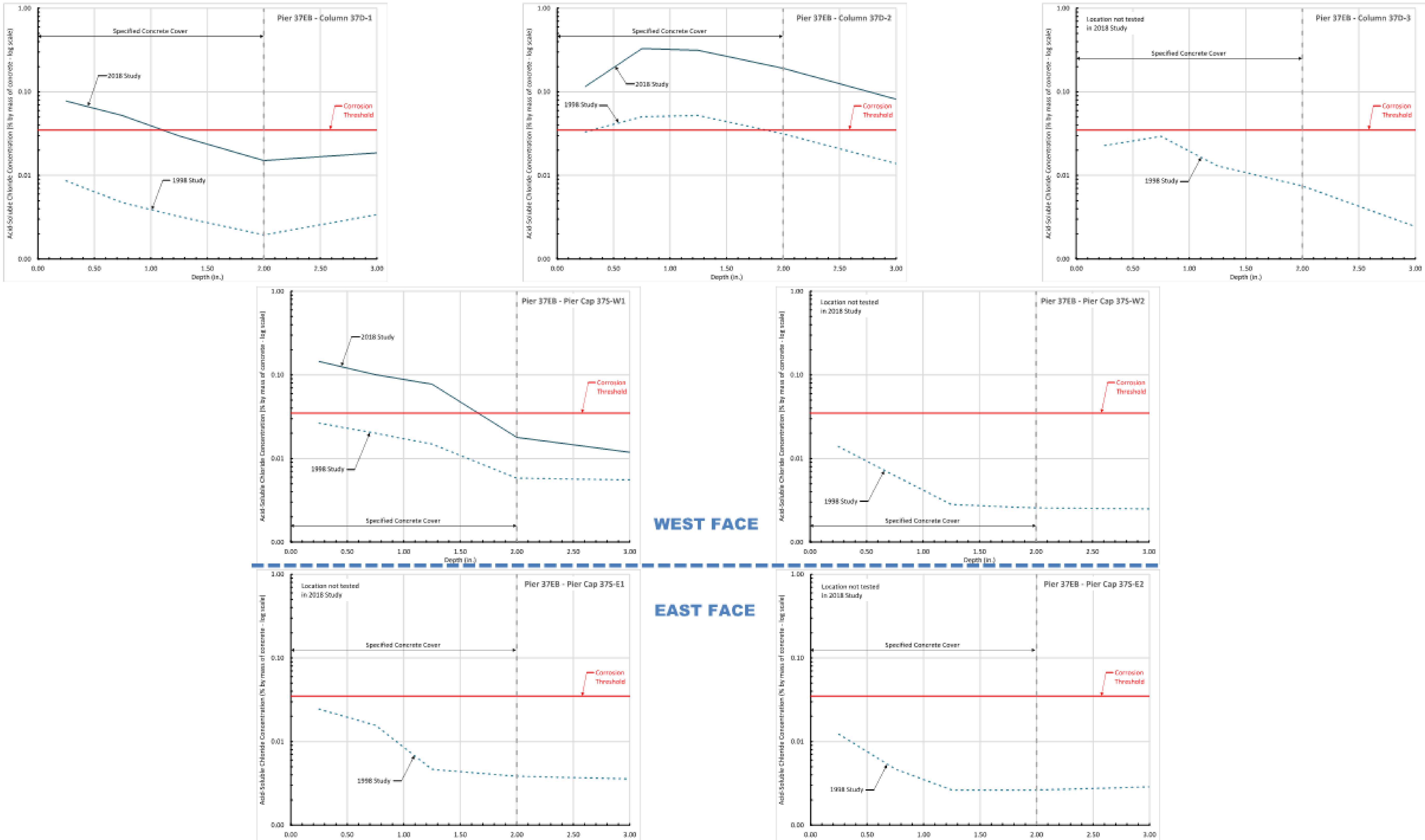


Figure 4.16 Pier 37EB [Non ECE] - Chloride Profiles - Column 37D (top row) and Pier Cap

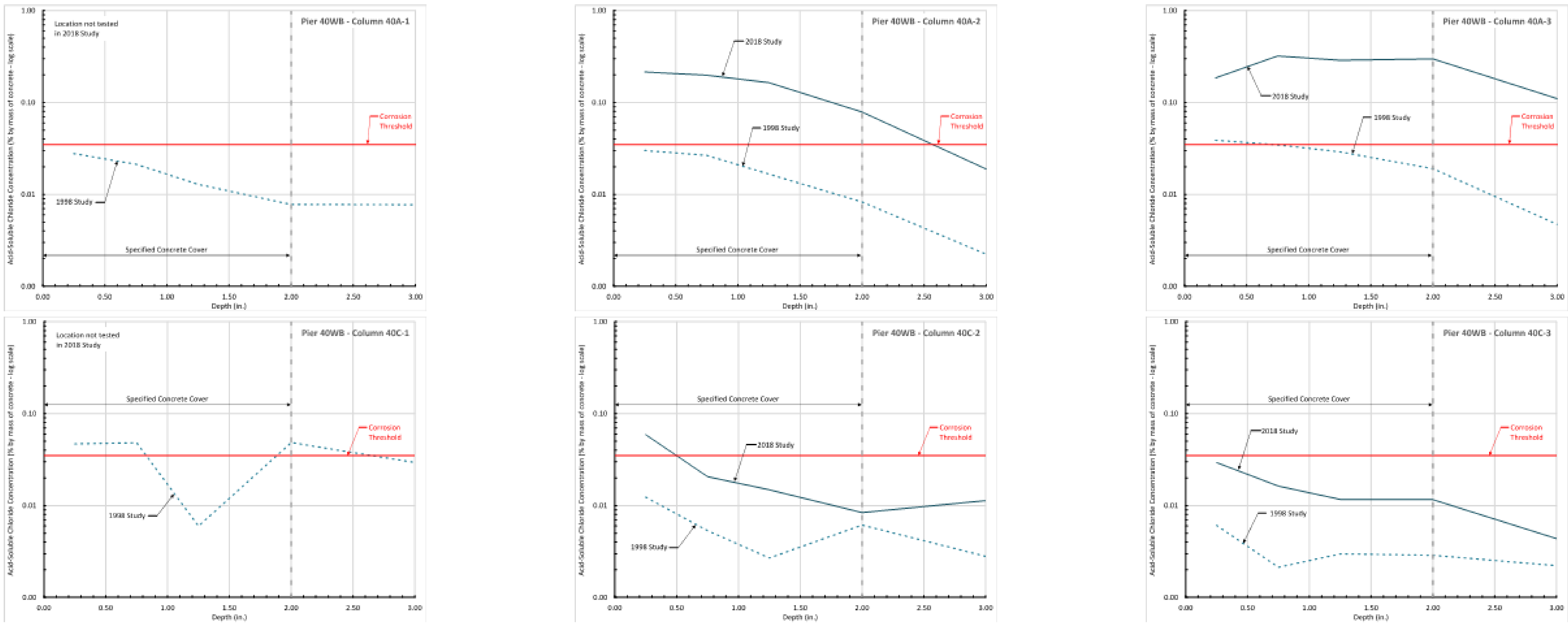


Figure 4.17 Pier 40WB [Non ECE] - Chloride Profiles - Columns 40A (top) and 40C (bottom)

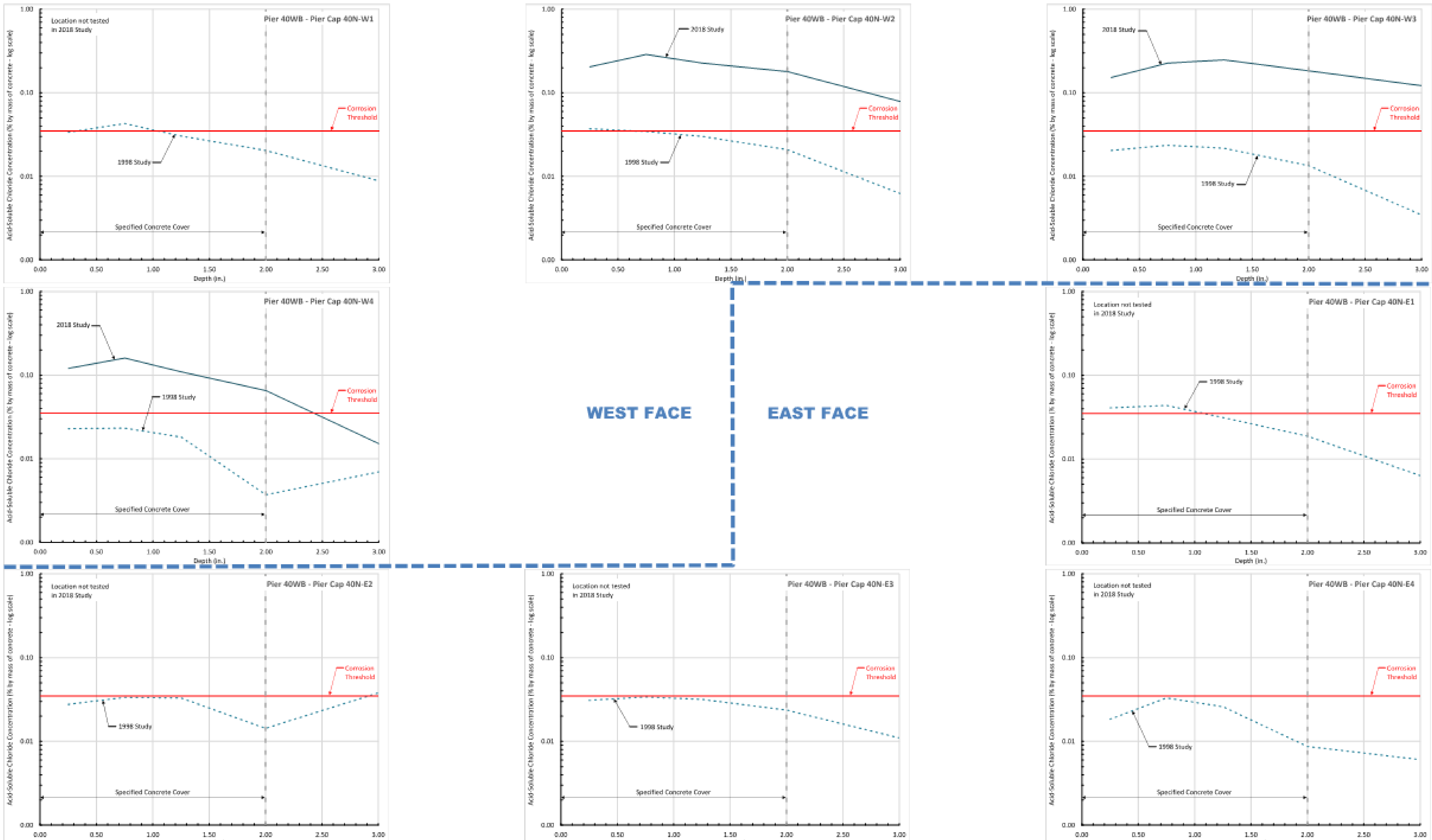


Figure 4.18 Pier 40WB [Non ECE] - Chloride Profiles - Pier Cap

4.3 INSTRUMENTATION: 20-YEAR REVIEW

4.3.1 Overview

Instrumentation was installed at several locations in all twelve columns and five pier caps which were included in the 1998 study to facilitate periodic monitoring of corrosion conditions. Instrumentation was desired to identify latent corrosion activity, particularly at sections where FRP wraps had been installed and the surface of the concrete could no longer be visually inspected, accessed for half-cell potential testing, or reliably evaluated using sounding techniques. The instrumentation included embeddable half-cell electrodes which reported corrosion potentials, and resistivity probes which served as on/off indicators of corrosion activity. Site visits were performed every few months from 1999 to 2007 to manually collect data from the installed instrumentation, and all collected data was plotted for evaluation. This data was revisited by the authors for comparison to the findings of the 2018 study.

Refer to Table 4.1 and Table 4.2 included in this section for analysis of the instrumentation data which was collected for the ECE treated and non-ECE treated structures. For either the embedded half-cell probes, or the half-cell testing performed as part of the 1998 study, locations with worst case potentials more positive than -200mV are highlighted with green shading (low risk of active corrosion), more negative than -350mV are highlighted with red shading (high risk of active corrosion), and in between -200mV and -350mV are highlighted in yellow (uncertain risk of active corrosion). Similarly, locations with maximum chloride concentrations above the corrosion threshold of 0.035 percent by weight of concrete are highlighted in red shading (high chloride levels), between 0.03 and 0.0349 percent are highlighted in yellow shading (moderate), and below 0.03 percent are highlighted in green (lower chloride levels).

4.3.2 Findings

Data obtained from the instrumentation which was installed as a component of the 1998 study is difficult to interpret because it lacked consistency between instrument types, did not perform as expected, or was not in alignment with the findings of the 2018 study. Readings collected at certain locations appeared to have been reliable while others appeared to be questionable or wrong, potentially due to damaged or defective sensors or inaccurate recording.

Examples of instrumentation data that appeared to be reasonable or reliable included:

- Initial chloride concentrations in excess of the corrosion threshold were present within at least one sampling depth range at 21 of 50 locations where instrumentation was installed. Resistivity probe corrosion failures occurred at approximately 50 percent of these locations. In contrast, resistivity probe corrosion failures only occurred at approximately 20 percent of the instrumentation locations where initial chloride concentrations at all sample depths were below the corrosion threshold.
- At 13 of 50 instrument locations, all embedded half-cell potential readings were passive and neither of the resistivity probes exhibited corrosion by 2007. Passive half-cell potentials were also measured in 2018 and no concrete distress was present at all thirteen of these locations.

- At two locations, 34C-3 and 40WB-W2, embedded half-cell potentials were indicative of likely corrosion activity and both resistivity probes had corroded by 2007. Elevated chloride levels and evidence of corrosion activity, including both concrete distress and highly negative half-cell potential readings, were identified at both of these locations in the 2018 study.
- Chloride levels in excess of the corrosion threshold existed within at least one sampling depth (i.e., 0 to 3-1/2 inches deep) at the time resistivity probes were installed at 21 of 50 instrumentation locations. Corrosion of resistivity probes occurred at over 50 percent of these locations. Conversely, resistivity probe corrosion was recorded at only 20 percent of the 29 instrument locations where post-ECE chloride levels at all depths were below the threshold.

Examples of instrumentation data that appeared questionable or in error included:

- Large resistivity probes corroded before, or the same year as, the small resistivity probes corroded at 5 of 20 locations (25 percent). Large resistivity probes were twice as thick as the small probes and, as such, were not expected to fail at a similar or faster rate.
- At 34A-2, the small resistivity probe corroded in 2003, the large resistivity probe corroded in 2004 and half-cell potentials were indicative of corrosion activity in 2005. However, in 2018, the half-cell potential was passive, the chloride concentration at the depth of the reinforcement was below the corrosion threshold and no concrete distress was present.
- Half-cell potentials collected at location 1 in Column 37C (37C-1) in 1999 were indicative of likely corrosion activity. However, none of the potentials collected between 2000 and 2007 were indicative of likely corrosion activity, and neither of the two small resistivity probes corroded. Concrete distress was identified at this location in the 2018 study.
- Half-cell potentials collected at all three probe locations in Column 37D, and one probe location in the Pier 40WB pier cap (40WB-W1), consistently indicated likely corrosion activity from 1999 to 2007. However, both resistivity probes installed at each of these four locations remained functioning, indicating no significant corrosion. Concrete distress was identified at two of the four locations in the 2018 study (37D-3 and 40WB-W1).

The installed instrumentation did not appear to be a reliable indicator of corrosion activity or inactivity. The 2018 study identified corrosion-related distress, or measured corrosion potentials indicative of probable corrosion activity (more negative than -350 mV), at 17 of the 50 locations where instruments were installed in the 1998 study. Resistivity probe corrosion and highly negative embedded half-cell potentials had been recorded by the installed instruments at only 2 of those 17 locations - Pier 34WB (34C-3) and Pier 40WB (40WB-W2). The data which was collected at the other 15 locations was inconclusive or suggested no corrosion activity was occurring. It is possible that the current damage conditions reflect corrosion activity that developed in the twelve years after MnDOT personnel ceased their instrumentation monitoring efforts in 2007. In contrast, no concrete distress and passive half-cell potential readings (more positive than -200 mV) were identified in the 2018 study at 21 of 50 instrument locations. Data consistent with no corrosion activity (i.e., no resistivity probe corrosion and passive embedded half-cell probe readings) was recorded at 52 percent of those locations, including sites in Columns 34B, 34D, 34E and 37B, and locations 34WB-W1 and 37WB-W1 in pier caps.

Table 4.1 Review of Instrumentation Data vs. 2018 Study Findings - ECE Treated Structures

Pier	Element	Location ID	Instrumentation Data (1998 to 2007)								2018 Study Data			Consistent Results?	
			Embedded Half-Cell		Small Resistivity Probe 1		Small Resistivity Probe 2		Large Resistivity Probe		Corrosion Damage and Risk Factors			Embedded Half-Cell	Instrumentation
			Corrosion Activity?	Year	Failure?	Year	Failure?	Year	Failure?	Year	Half-Cell Potential	Chloride Level at Depth of Steel	Concrete Distress?	vs. Resistivity Probes	vs. 2018 Study Data
		34A-1	No		Yes	2005	Yes	2001				N/A			
	Column A	34A-2	Yes	2005	Yes	2003			Yes	2004	Low	Low	No	Yes	
		34A-3	No		Yes	1999	No					High			
		34B-1	No		No				No			N/A		Yes	Yes
	Column B	34B-2	No		No		No				Low	Low	No	Yes	Yes
		34B-3	No		No				No			High		Yes	Yes
34WB		34C-1	No		Yes	1999	Yes	2006				Near Threshold	No		Possible
	Column C	34C-2	Uncertain		No		No				Moderate	Low			
		34C-3	Yes	2007	Yes	1999	Yes	2006				N/A	Yes	Yes	Yes
		W Face 1	No		No		No				Low	High	No	Yes	Yes
	Pier Cap	W Face 2	No		No		Yes	2007			Low	High			
		E Face 1	Uncertain		No				No		Low	N/A	Yes		Possible
		E Face 2	No		No				No		Moderate	High	No	Yes	
	Column A	37A-1	Uncertain		Yes	2007	Yes	1999			Low	Low	No	Yes	
		37A-2	No		No		Yes	2007				Low			
	Column B	37B-1	No		No		No				Low	Low	No	Yes	Yes
		37B-2	No		No		No					Low		Yes	Yes
37WB	Column C	37C-1	Uncertain		No		No				Moderate	High	Yes		
		37C-2	Uncertain		No		No					N/A	Yes		
		W Face 1	No		No				No		Low	High	No	Yes	Yes
	Pier Cap	W Face 2	No		Yes	1999			No		Low	N/A	Yes		
		E Face 1	Uncertain		No				No		Moderate	High	Yes		
		E Face 2	No		No				No		High	High	No	Yes	
													Total Locations	23	23
													Consistent or Possibly Consistent Results?	12	10
													Consistency Percentage	52%	43%

Table 4.2 Review of Instrumentation Data vs. 2018 Study Findings - Non-ECE Treated Structures

Pier	Element	Location ID	Instrumentation Data (1998 to 2007)								2018 Data			Consistent Results?	
			Embedded Half-Cell		Small Resistivity Probe 1		Small Resistivity Probe 2		Large Resistivity Probe		Corrosion Damage and Risk Factors			Embedded Half-Cell vs. Resistivity Probes	Instrumentation vs. 2018 Study Data
			Corrosion Activity?	Year	Failure?	Year	Failure?	Year	Failure?	Year	Half-Cell Potential	Chloride Level at Depth of Steel	Concrete Distress?		
		34D-1	No		No		No					N/A		Yes	Yes
	Column D	34D-2	No		No		No				Low	High		Yes	Yes
		34D-3	No		No		No					Low		Yes	Yes
	Column E	34E-1	No		Yes	2002	No				Low	N/A			
		34E-2	No		No				No			High		Yes	Yes
34EB		34F-1	Uncertain		Yes	2005			Yes	2006		N/A		Yes	Possible
	Column F	34F-2	Uncertain		Yes	2005			Yes	2005	Uncertain	High	No	Yes	Possible
		34F-3	Uncertain		Yes	2005			No			High		Yes	Possible
		W Face 1	No		No				Yes	1999	Low	High			
	Pier Cap	W Face 2	Uncertain		Yes	2007			No		Uncertain	High		Yes	Possible
		E Face 1	No		No				Yes	2004	Low	N/A			
		E Face 2	Uncertain		Yes	2004			No		Uncertain	High		Yes	Possible
		37D-1	Yes	1999	No				No			Low			
	Column D	37D-2	Yes	1999	No		No				N/A	High			
37EB		37D-3	Yes	1999	No		No					N/A	Yes		Possible
	Pier Cap	W Face 1	No		No				No		N/A	Low	No	Yes	Yes
		E Face 1	No		No				No		N/A	N/A	Yes	Yes	
		40A-1	Uncertain		No				No			N/A			
	Column A	40A-2	No		No				No		High	High	No		
		40A-3	Uncertain		Yes	1999			No			High		Yes	Possible
	Column C	40C-1	No		No				No		Uncertain	N/A		Yes	Yes
40WB		W Face 1	Yes	1999	No				No		High	N/A	Yes		
		W Face 2	Yes	2003	Yes	2005			Yes	1999	High	High	No	Yes	Possible
	Pier Cap	W Face 3	Uncertain		No				No		Uncertain	High		Possible	
		E Face 1	Uncertain		No				No		High	N/A	Yes	Possible	
		E Face 2	Uncertain		No				No		High	N/A		Possible	
		E Face 3	Yes	2007	No				No		Low	N/A	No		
Total Locations														27	27
Consistent or Possibly Consistent Results?														17	14
Consistency Percentage														63%	52%

4.4 ANALYSIS OF TREATMENT EFFECTIVENESS - PERFORMANCE AND COST

4.4.1 Overview

Following the execution of concrete surface repairs, different preservation strategies were installed as a component of the 1998 study on different sections of five piers of Bridge 27831. The objective of the 2018 study was to evaluate the effectiveness of each of these different strategies after 20 years of service. The strategies included:

- ECE treatment and FRP wrap (CFRP or GFRP) installation
- ECE treatment and application of concrete sealers (three different penetrating silanes)
- FRP wrap (CFRP or GFRP) installation
- Nothing

AMOCO CFRP, GFRP and MBrace CFRP systems were generically considered as types of “FRP wrap” for the purposes of our evaluation. This decision considered the similar composition of the different systems, with respect to the prevention of chloride ingress, and the relatively similar performance which was observed with respect to the rate of distress recurrence and the extent of chloride contamination. Laboratory testing performed as a part of the 1998 study suggested, for practical purposes, each FRP product could be considered relatively impermeable [11]. The general similarities in performance, regardless of the specific FRP type, were discussed in Sections 4.1 and 4.2. In addition, each FRP product was installed at only two treatment study sections. AMOCO CFRP and GFRP were each installed at one location on Pier 34WB and one location on Pier 34EB. MBrace CFRP was installed at one location on Pier 37WB and one location on Pier 40WB. Incomplete information was available regarding the history of moisture exposure at each of these piers between the 1998 study and the 2018 study. The location of these sites, and the performance of the deck joint directly above, was assumed to more significantly influence the 20 year performance of the treatment than the specific FRP system which was installed. For similar reasons, the three different sealer products (Hydrozo Enviroseal, Hydrozo Silane 40 and Fosroc Nicotote Dekguard) were also combined and generically considered as “sealers.”

The authors elected to consider factors associated with performance and cost in this evaluation. The performance factors included the rate of concrete distress recurrence within the limits of the sections where the strategy was implemented, as discussed in Section 4.1 and summarized in Figure 4.19 on the next page, and the half-cell potential and chloride testing results which were obtained and correlate to risks of latent or future corrosion, as discussed in Section 4.1 and 4.2. Refer to Table 4.3, at the end of this section, for a summary of the information considered. Within Table 4.3, the half-cell potential testing and chloride testing results are separated by the pier cap and column elements, with cells color shaded to distinguish risk levels. Color shading of the half-cell potential results references the guidance in ASTM C876 for interpretation of half-cell potential testing results [13]. Locations with worst case potentials more positive than -200mV are highlighted green (low risk of active corrosion), more negative than -350mV are highlighted red (high risk of active corrosion), and in between -200mV and -350mV are highlighted yellow (uncertain risk of active corrosion). Similarly, locations with maximum chloride concentrations above the corrosion threshold of 0.035 percent by weight of concrete are highlighted in red (high chloride levels), between 0.03 and 0.0349 percent are highlighted in yellow (moderate chloride levels), and below 0.03 percent are highlighted in green (low chloride levels).

The cost factors which were considered were the installation costs associated with implementation of each strategy during the 1998 study, and the costs of needed concrete surface repairs in areas of new or recurrent distress which were identified by the 2018 study. The installation costs were obtained from proposals which were issued by Vector Construction to MnDOT in 1997 for ECE treatment and FRP wrap installation at two piers of similar height and width. The costs for needed concrete surface repairs considered contract unit prices which were provided by MnDOT for chipping and shotcrete repairs to be performed at the substructures of Bridge 27831 beginning in 2020 [14]. The unit prices were increased by 15 percent by the authors to account for other costs associated with the repair project (e.g., mobilization, general conditions, etc.), and the repair quantities were approximated as the areas of surface distress identified in the 2018 study, multiplied by a 20 percent growth factor.

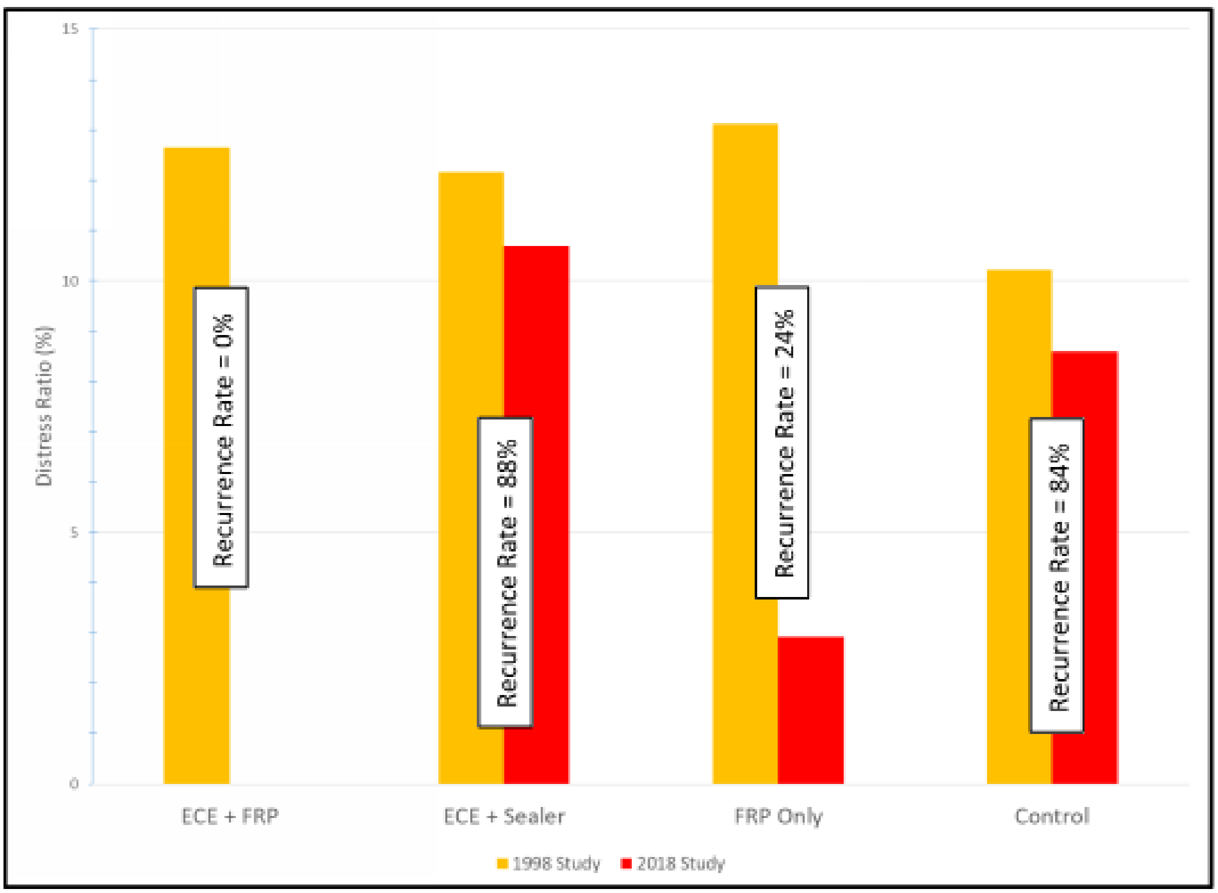


Figure 4.19 Comparison of distress ratio by treatment - 1998 vs. 2018

4.4.2 Findings

Three sections received both ECE treatment and FRP wrap installation as a component of the 1998 study, including the north and middle end sections of Pier 34WB and the north end section of Pier 37WB. The approximate cost of the installation of these treatments in 1998 was $91,000. As shown in Figure 4.19, after 20 additional years of service, no distress recurred and no new distress developed in any of the three sections. All corrosion potentials obtained in the 2018 study from these sections were passive (more positive than -200mV), except at Pier 37WB where corrosion potentials indicative of possible corrosion were measured in the north end of the pier cap and the base of Column 37A. However, and despite the FRP installation, laboratory test results indicate that significant chloride contamination has occurred over the past 20 years. Chloride levels in excess of the corrosion threshold exist at the depth of the reinforcing steel at locations in the pier caps of all three sections, and two of three columns. The elevated chloride levels create a risk for future corrosion of the reinforcing steel. Overall, the combination of ECE and FRP wrap installation was the most effective treatment and resulted in the best performance. This combination also had the highest installation cost, and total cost, in comparison to the other treatment strategies which were evaluated.

Three sections received ECE treatment followed by the application of concrete sealers as a component of the 1998 study, including the south end of Pier 34WB and the middle and south end sections of Pier 37WB. The approximate cost of the installation of these treatments in 1998 was $39,000. After 20 additional years of service, corrosion related distress recurred in all three sections. The rate of distress recurrence was 88 percent, corresponding to approximately 130 square feet of needed surface repairs. This recurrence rate, and the total quantity of distress, were the highest of all four strategies which were evaluated. Half-cell potentials indicative of possible or probable corrosion activity were also measured in the pier cap in all three sections, aligned with locations of distress recurrence. Chloride levels in excess of the corrosion threshold were also present at all six sampling locations in the pier caps. These findings indicate that corrosion of the reinforcing steel is the primary cause of the recurrent distress. In summary, the combination of ECE treatment and the application of a concrete sealer was the least effective corrosion mitigation strategy which was implemented in the 1998 study and resulted in the worst performance.

FRP wrap was installed at three sections which did not receive ECE treatment, including the middle and south end sections of Pier 34EB and the north end section of Pier 40WB, at an approximate cost of $57,000 in 1998. No distress recurred at either of the application sites at Pier 34EB, and all half-cell potentials were indicative of low (passive) or uncertain (moderate) corrosion risk. No obvious evidence of significant or long-standing deck joint leakage was observed at either of these sections. In contrast, the distress recurrence rate at the Pier 40WB section was 60 percent and half-cell potentials indicative of probable corrosion activity were measured at the locations of unsound concrete. Wetness and water staining indicative of joint leakage was observed across the pier cap of this section at the time of the 2018 study. The mixed results appeared directly related to the differing moisture exposure conditions - the section which was positioned below an obviously leaky deck joint exhibited poor performance. In addition, and similar to the other strategies, chloride contents in excess of the corrosion threshold exist at the depth of the reinforcing steel at all sampled locations in all three pier caps and all three columns which received this treatment. The presence of high chloride levels suggests future corrosion of the reinforcing may occur behind the FRP wrap. Overall, the FRP wrap installation strategy produced mixed performance results at the second highest calculated total cost.

After shotcrete repairs were performed, three sections - the north end of Pier 34EB and the south ends of Pier 37EB and 40WB - did not receive ECE treatment and no FRP wrap or concrete sealers were installed. These sections were maintained as controls, essentially representing a strategy of “no action,” with no installation costs needing to be considered. The 2018 study identified recurrent distress in all three sections, and some new areas of distress at the Pier 37EB section, all of which now warrants surface repairs. The overall rate of recurrence was 84 percent, just below the rate observed at sections which received ECE treatment and concrete sealers. However, the risk factors for possible future corrosion activity were generally less concerning than was observed with the three other strategies. Passive corrosion potentials were measured in the pier cap and column of the Pier 34EB section, and the column of the Pier 40WB section, and Pier 37EB possessed the only sample location from a pier cap where chloride levels are below the corrosion threshold at the depth of the reinforcing steel. Although the overall performance of the “no action” strategy was poor, the total cost of this strategy was significantly lower than the other three strategies which were investigated by the 1998 study.

Table 4.3 Comparison of Corrosion Mitigation Strategies After 20 Years of Service - Performance

Corrosion Mitigation Strategy					Concrete Distress Conditions					Risk Factors for Future Corrosion			
Description		Installation Locations			1998		2018			2018 Study Data			
ECE	Surface Treatment Type	Pier	Section of Pier Cap	Column	Approximate Total Area (SF)	Distress Ratio[1]	Approximate Total Area (SF)	Distress Ratio[1]	Recurrence Rate (%)	Risk of Corrosion as identified by Half-Cell Potential Testing[2]		Chloride Level over threshold at Depth of Reinforcement?	
										Pier Cap	Column	Pier Cap	Column
Y	FRP Wrap	34WB	North End	A	130	13%	0	0%	0%	Low	Low	Yes	Yes
		34WB	Middle	B						Low	Low	Yes	Yes
		37WB	North End	A						Moderate	Low	Yes	No
Y	Sealer	34WB	South End	C	125	12%	110	11%	88%	Moderate	Low	Yes	Near[3]
		37WB	Middle	B						Moderate	Low	Yes	No
		37WB	South End	C						High	High	Yes	Yes
Y	None	*Not studied*											
N	FRP Wrap	34EB	Middle	E	135	13%	30	3%	22%	Low	Low	Yes	Yes
		34EB	South End	F						Moderate	Moderate	Yes	Yes
		40WB	North End	A						High	High	Yes	Yes
N	Sealer	*Not studied*											
N	None	34EB	North End	D	95	10%	80	9%	84%	Low	Low	Yes	Yes
		37EB	North End	D						N/A	N/A	No	Yes
		40WB	South End	C						Moderate	Low	Yes	No

1. The Distress Ratio was calculated as the area of concrete distress (square feet) present within a section divided by the total surface area of the section. The 2018 distress may include areas not previously repaired.

2. Corrosion risk levels reference guidance in ASTM C876 for interpretation of half-cell potential testing results more positive than -200 mV (low risk), more negative than -350 mV (high risk), or in between (moderate risk) [13].

3. The chloride threshold value referenced herein is 0.035 percent by weight of concrete. The chloride level at the depth of the reinforcement in Column 34C was 0.032 percent by weight of cement.

Table 4.4 Comparison of Corrosion Mitigation Strategies After 20 Years of Service - Cost

Corrosion Mitigation Strategy - 1998 Study								Concrete Repairs - 2019			Total Cost[4,5] (1998 + 2019)
Description		Installation Locations			Installation Cost[1] (Approximate square feet treated)			Approximate Quantity[2] (SF)	Shotcrete Repair[3] ($/SF)	Approximate Total Cost[4]	
ECE	Surface Treatment Type	Pier	Section of Pier Cap	Column	ECE	Surface Treatment	Mobilization/ Demobilization				
Y	FRP Wrap	34WB	North End	A	$33,805 (1025)	$51,625 (900)	$5,160	0	$165	0	$91,000
		34WB	Middle	B							
		37WB	North End	A							
Y	Sealer	34WB	South End	C	$33,805 (1025)	N/A	$5,160	130		$22,000	$61,000
		37WB	Middle	B							
		37WB	South End	C							
N	FRP Wrap	34EB	Middle	E	0	$51,625 (900)	$5,160	35		$6,000	$63,000
		34EB	South End	F							
		40WB	North End	A							
N	None	34EB	North End	D	0	0	$0	95		$16,000	$16,000
		37EB	North End	D							
		40WB	South End	C							

1. Proposals issued by Vector Construction to MnDOT in 1997 presented total costs for ECE treatment and/or FRP wrap installation at two piers of similar height and width. Although ECE treatment was performed on the entire pier, FRP wrap was not applied on the majority of the bottom surface or at the bases of columns. The proposal did not explicitly detail any installation costs for the concrete sealers and, as such, no costs are tabulated herein.

2. Repair quantities were approximated as the areas of surface distress identified in Table 4.3 multiplied by a 20 percent growth factor.

3. Unit cost were provided by Paul Pilarski of MnDOT from SP2789-151 contract prices for chipping and shotcrete repairs at the substructures of Bridge 27831, increased by 15% to account for additional projects costs [14].

4. Total costs were rounded up to the nearest thousand.

5. For simplicity, the total cost of the 1998 corrosion mitigation strategy and 2019 repairs are presented in actual dollar spent and were not adjusted to account for inflation.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

In 1997, MnDOT initiated a research project to study the effectiveness of new strategies for mitigating corrosion in chloride-contaminated reinforced concrete bridge substructures, including electrochemical chloride extraction (ECE) and fiber reinforced polymer (FRP) wrap installation. The study focused on five reinforced concrete piers of the substructure of Bridge No. 27831 in Minneapolis. Four different corrosion mitigation strategies were investigated, and each strategy was installed at three different locations among the five piers.

In 2018, after the bridge had seen an additional 20 years of service, the research reported herein was initiated with the objective of using a combination of field inspection, non-destructive evaluation, and laboratory testing techniques to assess the condition of the five piers and evaluate the long-term performance of the four corrosion mitigation strategies.

The following conclusions can be drawn from this study:

- The three pier sections that received ECE treatment and FRP wrap installation exhibited very good performance. No new distress developed, no distress recurred in repaired areas, and no evidence of probable corrosion activity was identified by half-cell potential testing. Although successful, this strategy was judged to be the least cost-effective in comparison to other approaches studied at this bridge.
- The highest incidence of recurrent distress (88 percent) occurred in areas that received ECE treatment followed by the application of a penetrating sealer. Half-cell potentials indicative of possible or probable corrosion activity were measured in areas of recurrent distress. The overall performance of these sections was comparable to the control areas, so little value was gained from the implementation of these mitigation efforts. These results indicate that ECE treatment does not eliminate the risk of future corrosion activity, and the effectiveness of the treatment can be short-lived if chloride and moisture exposure persists.
- Mixed performance was observed at the sections where ECE treatment was not performed, but FRP wrap was installed. No new or recurrent distress and passive or moderate corrosion potentials were identified in two sections where no obvious evidence of deck joint leakage problems was apparent. In contrast, the third section was visibly wet and water-stained and exhibited a significant rate of recurrent distress, as well as several areas of highly negative corrosion potentials. The installation of FRP wrap, alone was not effective at mitigating corrosion in areas of high moisture exposure resulting from deck joint failure.
- Three pier sections were maintained as controls and received no ECE treatment and no surface protection. Although the majority of the distress that was repaired in 1997 had recurred by 2018, the “no action” approach was judged to be the most cost-effective corrosion mitigation strategy employed as a part of the 1998 study.

- ECE treatment caused significant and immediate reductions in the extent of chloride contamination present at the pier caps and columns of Pier 34WB and Pier 37WB and resulted in re-passivation of the reinforcing steel. These conditions were not sustained. Laboratory testing performed in 2018 measured chloride concentrations in excess of pre-ECE treatment levels at almost all locations sampled. Field testing performed in 2018 measured moderate or high-risk corrosion potentials in four of six study sections that received ECE treatment. Two of the three sections wrapped with FRP have remained passive.
- Significant chloride contamination has occurred in all five piers within the past 20 years. None of the FRP wrap types or concrete sealer products prevented the ingress of new chlorides into the concrete in the manner in which these systems were installed for this study. Chloride concentrations typically exceed the corrosion threshold at the depth of the reinforcing steel, creating a risk of corrosion activity in future years.
- The instrumentation installed as a component of the 1998 study did not prove to be a reliable indicator of corrosion activity.

5.2 RECOMMENDATIONS

The 2018 study was intended to obtain information that could better inform future decisions by MnDOT for the rehabilitation of reinforced concrete bridge substructures suffering from chloride contamination and chloride-induced corrosion damage in Minnesota. The information was derived from the results of field and laboratory testing performed to evaluate the performance of five piers of Bridge 27831 where four different corrosion mitigation strategies had been installed 20 years prior.

It is difficult to offer definitive or wide-ranging recommendations regarding the different corrosion mitigation strategies because each was installed on only three sections, and it is unlikely that all sections, or groups of sections, experienced the same moisture and chloride exposure conditions over the 20 years between application and evaluation. Regardless of strategy, locations of significant or frequent exposure to moisture and chlorides would be at greater risk of corrosion activity than locations that remained dry and were not exposed to deicing salts or chemicals containing chloride ions.

The following recommendations can be drawn from this study:

- The most effective corrosion mitigation strategy that can be performed to extend the service life of original or repaired concrete elements was to minimize water and chloride exposure through the diligent maintenance, effective repair, or timely replacement of bridge deck joints and deck drainage systems.
- Although effective for strengthening applications, FRP wrap systems may not function as waterproofing barriers when installed on existing bridge elements. The presence of an FRP wrap will obscure the concrete surface, inhibiting visual inspection and will not alone prevent corrosion activity and distress from developing or recurring behind the FRP.
- Combining ECE treatment with FRP wrap installation was the most effective corrosion mitigation strategy evaluated in this study (the only treatment that resulted in no recurring distress), but it was also the most expensive. The cost of the ECE and FRP installation work at Bridge 27831 was

not influenced by any of the typical cost drivers, which may be encountered on other bridge structures for maintenance of traffic (none was required) or access (all work was performed on ground-supported scaffold). Although very good performance was achieved, this strategy may only warrant consideration for applications or structures with special circumstances, which would make periodic interventions for traditional deck or substructure repairs impractical.

AFTERWORD (BY PAUL PILARSKI, MNDOT)

Substructure repair in Minnesota has been typically required because of chloride-induced corrosion, which is primarily due to exposure. Exposure conditions include leaking expansion joints, leaking or splashing drainage systems, saturated slopes and ponds near roadways, or high exposure to deicing chemicals near plowed roadways. Repair of these concrete elements has sometimes required reduction of service in the form of traffic restrictions during repair work, and closure of any lanes near the work area. Typical repairs in Minnesota are performed by chipping to expose the corroding reinforcement, sandblasting clean the reinforcement and supplementing it where greater than 25% section loss exists, and replacing the concrete through the dry mix shotcrete method. The shotcrete method has improved over the years but still remains challenging to restore the section fully for several reasons:

- The delamination and the repair chipping process creates a redistribution of forces to remaining sections without shoring and load relief.
- High-strength concrete mixes are usually used, and material properties include rapid strength gain. These materials are necessary to bond to the surface, but the cured elastic modulus is different from the parent concrete. Materials with differing elastic modulus will share load unequally.
- Repair materials are often not thermally compatible with the substrate.
- Moist or wet curing necessary to prevent cracking is very difficult on vertical and overhead surfaces, which represent the majority of concrete repair locations in substructures.

Concrete repairs performed for MnDOT could be characterized by partial success with many instances where the repair either quickly map-cracked, debonded, or underwent delamination beyond the limits of the prior repair. The 1998 study was an attempt by MnDOT to try various treatments that showed the promise of better performance than typical repair procedures. Based on conversations with MnDOT bridge maintenance personnel on staff during the 1998 work, a common remark was that the study “was very expensive and took a long time.” However, for this location, no traffic impacts were associated with the ECE application and the construction activities were largely without adverse service impacts.

The 1998 bridge study was the only ECE application MnDOT had performed. Today, MnDOT has more history and experience with FRP. This experience includes FRP repairs to concrete bridge girders that sustained impact and, more recently, as a strengthening approach to pier caps and/or as a confining mechanism to concrete repairs where some section loss has occurred. Many recent studies suggest FRP confining pressure significantly slows corrosion even in chloride laden environments [6, 7, 8]. It is believed that reducing the ability of concrete repair shrinkage cracks to appear at the surface, or to be opened further with slight expansive forces, greatly reduces the availability of moisture and oxygen to get at the reinforcing steel, both of which are key components in the corrosion process. Although the benefits of FRP on improved repair durability have been viewed favorably by MnDOT, the growing inventory of aging bridges and stagnant bridge maintenance budget create challenges.

FRP installations are not without controversy, though. MnDOT struggles with asset management and inspection when hybrid structural systems (e.g., when FRP is used for strengthening) exist. For this reason, simplifying the load path and load carrying components has been the favored approach if at all possible. But, in limited cases, MnDOT is using FRP where the risk appears manageable within the inspection program. This may mean limiting FRP to areas with low chloride exposure conditions or low levels of deterioration. It may also mean leaving large areas uncovered to create ample opportunity for visual and sounding examination near concrete wrapped in bands of FRP.

As is the case with many agencies, staff turnover or upward mobility can often result in forgotten lessons and forgotten research. The 1998 study investigated many treatments that are still available today. In fact, there are more products on the market today to repair bridges and enhance durability than ever before. The products may change, but the philosophies of repair enhancement are very similar to those conceived and implemented in 1998. This study enabled a look into long-term performance of each of these strategies and is considered extremely useful for gaging the long-term benefits of such approaches.

The initial review of the pier delamination locations and field conditions appeared consistent with other locations along the bridge. In other words, at all study areas, corrosion and delamination conditions were worse near drainage scuppers and at overhangs. Since all piers were under joint locations, it is not surprising that new corrosion would occur due to leaking joints. However, it was surprising that the locations of prior repair were generally holding up fairly well unless water exposure was extensive. Normally, prior repair locations have not been well-recorded and it is difficult to gage recurrence. The good performance observed in this study is contradictory to many DOT surveys that give concrete repairs a service life of five to ten years, if not cathodically protected.

The principal interest to MnDOT with the 2018 study was to determine whether there was a clear winner among the different treatment approaches for *long-term* performance, in terms of cost to the DOT to gain that benefit, including the construction impacts and the implications for future inspection. The 2018 pier sounding was performed by Collins Engineers by another contract. Sounding maps were generated and overlaid onto 1998 repair maps scaled from the 1998 report sketches. Without considering chloride or half-cell data, the following general observations were initially made by MnDOT:

Patch performance of BR 27831 - Piers repaired in 1998, re-sounded in 2018.		
	ECE	*No ECE*
FRP	Excellent performance	Good performance of existing patches where water did not penetrate
Silane	Fair performance Pier 34 WB, Poor performance Pier 37 WB	Not investigated
Control	Not investigated	Fair performance — limited recurrence at Pier 34EB but significant recurrence and new distress (some adjacent to old repairs) at Pier 37EB and Pier 40WB

The initial observations were scrutinized with the staining pattern of the pier caps to discern if expansion joint leakage had contributed to the new corrosion, which in many cases it did. The simple message was that the best performance will be reached when the concrete is kept dry, especially in previously chloride contaminated concrete. With this observation, the DOT believes maintaining bridge joints, replacing joints on appropriate intervals, improving bridge joints, and improving drainage details should remain essential strategies toward substructure protection.

Bridge designers can often be narrowly focused on structural detailing, but greater detail coordination with drainage plans would be a valuable investment in terms of the service life of the structure. Where externally mounted scuppers and downspouts are unavoidable, significantly oversized closed piping systems with more modest elbow bends and cleanouts (upper-right image) are preferred over segmented downspout and trough systems (upper-left image), which offer installation flexibility but typically exhibit poor performance. At ground level, extensions to the downspout in the form of snorkels (bottom left) or large runoff basins (bottom right) are essential to preventing column corrosion.




For the subject pier study, the delamination locations did include some unusual observations at silane-treated piers. In those instances, bottom face delamination was present without corresponding side face distress. Usually side face spalling is seen to accompany pier cap underside deterioration when leaking joints are causing the issue. This is because the wetting zone from leaking joints will include the pier cap side face prior to reaching the pier cap underside, and vertical stirrup reinforcement is as near the face of concrete as anywhere on the pier cap. On the silane-treated piers, some locations showed new delamination along the underside of the pier cap only, while the side faces were without deterioration. One theory on this observation is that the silane may not enable full release of moisture. However, the FRP covered areas would be equally resistant to moisture release from interior regions of the concrete. Nonetheless, it is unlikely that MnDOT will pursue silane treatments or impervious coatings on the underside of chloride-contaminated pier caps.

FRP showed better success at recurrence prevention and is still under exploration for use by MnDOT. The FRP installation of the 1998 study left the bottom of the pier cap uncovered due to concerns about trapping moisture. The precaution is due to the inability to inspect for delamination and corrosion effects. In the spring of 2020, a substructure repair project on the study bridge will remove the 1998 FRP, inspect surfaces by hammer sounding, document deterioration, and repair any delamination under these previously covered areas. In selected areas, excavation windows to the reinforcement will be used to investigate for any corrosion in the presence of the known high chloride contamination. The investigation will be particularly insightful because chloride sampling has shown a high chloride environment behind the FRP. Direct observations to determine if the reinforcement has corroded in this environment will help develop higher confidence in the employment of FRP on contaminated concrete.

MnDOT proceeded with further trials on FRP enhancement of concrete repairs in 2017 based on industry research. Carbon fiber reinforforced polymer (CFRP) was used due to its relatively low cost difference with glass fiber. CFRP also provides a higher strength supplement to compensate for minor reinforcement section loss due to corrosion that inherently caused the initial delamination. In this project, bands of FRP were detailed to confine repair areas at the pier caps. The use of bands was sufficient for strengthening objectives, allowed for potential moisture relaease between bands, and permitted future hammer sounding inspection of concrete surfaces between the locations of the FRP bands (see upper image).

Following installation, the FRP bands and pier and column surfaces were coated over by breathable masonry finishes improving aesthetics to be nearly unobservable.

The ECE treatment comes at high construction costs and repair construction duration, at least for the scale of the project under study. From an agency investment perspective, the ECE tool may be applicable to historic concrete bridges where the service life expectation is undefined. For historic bridges, avoiding repeated repairs is of paramount concern because both the project delivery cost and repair cost are significantly more than for non-historic bridges. Historic bridge repairs should utilize strategies that have the most promise of durability and effectiveness to avoid repeat investment.

When used in combination with FRP, the ECE treatment resulted in no new delaminations after 20 years (Refer back to Chapter 4). However, when in combination with a sealer, the performance of the ECE was not as good. The combination of ECE and sealers shows little benefit over the control case in terms of reducing the distress ratio. While the chloride levels of post-ECE treatment showed great reductions, the end results after 20 years showed similar delamination level as compared to the control. It also appears that the chloride reductions achieved following ECE treatment were not sustained because the surface protection measures (sealer) did not perform. The chloride levels at depth of reinforcement have returned to above threshold values. Such observations make the applicability of ECE questionable at best for the DOT.

The combination of FRP and ECE shows the largest success in this regard, but it has been challenging to gain acceptance of the FRP application on historic bridges due to concerns about unknown long-term effects and visual consequences.

There may be locations where ECE treatment in combination with FRP are more appropriate. Such locations may be hammerhead piers where replacement cost is high next to traffic. However, the long construction duration could also impact traffic diversion requirements. It is difficult at this time to see where ECE is best suited from an agency standpoint given the limited success, high cost, and possible reliance on being partnered with FRP installation, or exceptional deck joint maintenance, to sustain its benefits long term.

In light of this study, repairs to existing chloride-contaminated concrete piers should consist of spot concrete repairs alone for low-exposure regions. Repairs to larger areas of pier caps would benefit from confining concrete repairs with FRP, using large galvanic protection anodes, or a combination thereof. While galvanic protection anodes were not part of this study, MnDOT has used galvanic protection products with sporadic and inconsistent deployments. Initial observations are that the embedded anodes are somewhat effective while surface applied anodes may be more effective. For columns in the splash zone, FRP wrapped repairs or enlarged concrete collars with increased cover have been durable approaches. At this time, there has not been a cost-benefit study on concrete column collars and FRP-wrapped repairs in bands other than anecdotal comparisons to failed spot repairs in these regions.

In summary, the best strategy the DOT could employ remains investing in maintaining bridge joints and reducing direct water and chloride exposure whenever possible. Where not possible, increased concrete cover and/or corrosion-resistant reinforcement should be used. Better coordination of the water path between bridge designers and drainage designers would improve service life and maintenance issues.

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APPENDIX A: CHLORIDE DATA - 1998 STUDY VS. 2018 STUDY

Table A-1 Tabulation of Chloride Data Pre- and Post-ECE (1998 Study) vs. 20 Years Later (2018 Study)

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)					
				1998 Study			2018 Study		
				Pre-ECE	Post-ECE	Change (%)	Sample ID	Chloride Level	Change (%)
34WB	ECE + AMOCO CFRP	34A-1	0 - 0.5	0.054	0.027	-50.0	N/A	N/A	N/A
			0.5 - 1	0.054	0.020	-63.0		N/A	N/A
			1 - 1.5	0.056	0.015	-73.2		N/A	N/A
			1.5 - 2.5	0.032	0.012	-62.5		N/A	N/A
			2.5 - 3.5	0.021	0.009	-57.1		N/A	N/A
		34A-2	0 - 0.5	0.065	0.019	-70.8	1734A-2	0.052	173.7
			0.5 - 1	0.048	0.023	-52.1		0.101	339.1
			1 - 1.5	0.035	0.012	-65.7		0.078	550.0
			1.5 - 2.5	0.021	0.014	-33.3		0.024	71.4
			2.5 - 3.5	0.012	0.003	-75.0		0.013	333.3
		34A-3	0 - 0.5	0.063	0.033	-47.6	1734A-3	0.162	390.9
			0.5 - 1	0.071	0.039	-45.1		0.205	425.6
			1 - 1.5	0.078	0.029	-62.8		0.211	627.6
			1.5 - 2.5	0.050	0.022	-56.0		0.194	781.8
			2.5 - 3.5	0.029	0.013	-55.2		0.147	1030.8
	ECE + GFRP	34B-1	0 - 0.5	0.023	0.008	-65.2	N/A	N/A	N/A
			0.5 - 1	0.009	0.005	-44.4		N/A	N/A
			1 - 1.5	0.006	0.005	-16.7		N/A	N/A
			1.5 - 2.5	0.003	0.005	66.7		N/A	N/A
			2.5 - 3.5	0.003	0.003	0.0		N/A	N/A
		34B-2	0 - 0.5	0.037	0.006	-83.8	1734B-2	0.050	733.33
			0.5 - 1	0.054	0.013	-75.9		0.080	515.38
			1 - 1.5	0.031	0.008	-74.2		0.052	550.0
			1.5 - 2.5	0.023	0.003	-87.0		0.016	433.33
			2.5 - 3.5	0.011	0.002	-81.8		0.008	300.00
		34B-3	0 - 0.5	0.044	0.014	-68.2	1734B-3	0.066	371.43
			0.5 - 1	0.052	0.021	-59.6		0.164	680.95
			1 - 1.5	0.046	0.023	-50.0		0.143	521.74
			1.5 - 2.5	0.027	0.021	-22.2		0.103	390.48
			2.5 - 3.5	0.021	0.007	-66.7		0.038	442.86

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)					
				1998 Study			2018 Study		
				Pre-ECE	Post-ECE	Change (%)	Sample ID	Chloride Level	Change (%)
34WB	ECE + Sealer	34C-1	0 - 0.5	0.050	0.035	-32.0	1734C-1	0.078	129.4
			0.5 - 1	0.061	0.027	-55.7		0.099	266.7
			1 - 1.5	0.043	0.016	-62.8		0.094	487.5
			1.5 - 2.5	0.027	0.012	-55.6		0.032	166.7
			2.5 - 3.5	0.017	0.004	-76.5		0.009	125.0
		34C-2	0 - 0.5	0.029	0.006	-79.3	1734C-2	0.034	466.7
			0.5 - 1	0.059	0.010	-83.1		0.031	210.0
			1 - 1.5	0.030	0.004	-86.7		0.019	375.0
			1.5 - 2.5	0.027	0.003	-88.9		0.010	233.3
			2.5 - 3.5	0.010	0.002	-80.0		0.012	500.0
		34C-3	0 - 0.5	0.050	0.033	-34.0	N/A	N/A	N/A
			0.5 - 1	0.069	0.051	-26.1		N/A	N/A
			1 - 1.5	0.052	0.041	-21.2		N/A	N/A
			1.5 - 2.5	0.044	0.033	-25.0		N/A	N/A
			2.5 - 3.5	0.030	0.010	-66.7		N/A	N/A
	ECE + AMOCO CFRP	34N-W1	0 - 0.5	0.046	0.021	-54.3	1734N-W1	0.104	395.2
			0.5 - 1	0.043	0.035	-20.9		0.164	382.4
			1 - 1.5	0.038	0.030	-21.1		0.148	393.3
			1.5 - 2.5	0.017	0.010	-41.2		0.080	700.0
			2.5 - 3.5	0.013	0.006	-53.8		0.031	416.7
	ECE + GFRP	34N-W2	0 - 0.5	0.054	0.031	-42.6	1734N-W2	0.108	248.4
			0.5 - 1	0.039	0.037	-5.1		0.108	191.9
			1 - 1.5	0.033	0.038	15.2		0.100	163.2
			1.5 - 2.5	0.029	0.033	13.8		0.052	57.6
			2.5 - 3.5	0.035	0.028	-20.0		0.024	-14.3
	ECE + Sealer	34N-W3	0 - 0.5	0.039	0.022	-43.6	N/A	N/A	N/A
			0.5 - 1	0.060	0.014	-76.7		N/A	N/A
			1 - 1.5	0.041	0.007	-82.9		N/A	N/A
			1.5 - 2.5	0.044	0.002	-95.5		N/A	N/A
			2.5 - 3.5	0.029	0.001	-96.6		N/A	N/A
	ECE + Sealer	34N-W4	0 - 0.5	0.045	0.030	-33.3	1734N-W4	0.143	367.7
			0.5 - 1	0.035	0.005	-85.7		0.151	2920.0
			1 - 1.5	0.032	0.020	-37.5		0.142	610.0
			1.5 - 2.5	0.009	0.008	-11.1		0.074	825.0
			2.5 - 3.5	0.004	0.014	250.0		0.037	164.3

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)					
				1998 Study			2018 Study		
				Pre-ECE	Post-ECE	Change (%)	Sample ID	Chloride Level	Change (%)
34WB	ECE + Sealer	34N-E1	0 - 0.5	N/A	0.021	N/A	N/A	N/A	N/A
			0.5 - 1	N/A	0.036	N/A		N/A	N/A
			1 - 1.5	N/A	0.019	N/A		N/A	N/A
			1.5 - 2.5	N/A	0.010	N/A		N/A	N/A
			2.5 - 3.5	N/A	0.006	N/A		N/A	N/A
	ECE + Sealer	34N-E2	0 - 0.5	N/A	0.043	N/A	1734N-E2	0.175	307.0
			0.5 - 1	N/A	0.024	N/A		0.200	733.3
			1 - 1.5	N/A	0.027	N/A		0.129	377.8
			1.5 - 2.5	N/A	0.022	N/A		0.135	513.6
			2.5 - 3.5	N/A	0.009	N/A		0.081	800.0
	ECE + AMOCO CFRP	34N-E3	0 - 0.5	N/A	0.026	N/A	1734N-E3	0.184	607.7
			0.5 - 1	N/A	0.026	N/A		0.200	669.2
			1 - 1.5	N/A	0.016	N/A		0.199	1143.8
			1.5 - 2.5	N/A	0.012	N/A		0.163	1258.3
			2.5 - 3.5	N/A	0.009	N/A		0.137	1422.2
	ECE + GFRP	34N-E4	0 - 0.5	N/A	0.016	N/A	1734N-E4	0.074	362.5
			0.5 - 1	N/A	0.023	N/A		0.101	339.1
			1 - 1.5	N/A	0.005	N/A		0.088	1660.0
			1.5 - 2.5	N/A	0.006	N/A		0.030	400.0
			2.5 - 3.5	N/A	0.004	N/A		0.080	1900.0

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)				
				1998 Study		2018 Study		
				Pre-ECE	Post-ECE	Sample ID	Chloride Level	Change (%)
34EB	Control	34D-1	0 - 0.5	0.037	0.030	N/A	N/A	N/A
			0.5 - 1	0.032	0.035		N/A	N/A
			1 - 1.5	0.018	0.015		N/A	N/A
			1.5 - 2.5	0.006	0.002		N/A	N/A
			2.5 - 3.5	0.002	0.004		N/A	N/A
		34D-2	0 - 0.5	0.052	0.010	1734D-2	0.157	1470.0
			0.5 - 1	0.036	0.004		0.241	5925.0
			1 - 1.5	0.030	0.004		0.192	4700.0
			1.5 - 2.5	N/A	0.002		0.089	4350.0
			2.5 - 3.5	N/A	0.004		0.018	350.0
		34D-3	0 - 0.5	0.030	0.029	1734D-3	0.021	-27.6
			0.5 - 1	0.035	0.035		0.049	40.0
			1 - 1.5	0.026	0.021		0.009	-57.1
			1.5 - 2.5	N/A	0.007		0.004	-42.9
			2.5 - 3.5	N/A	0.005		0.011	120.0
	GFRP	34E-1	0 - 0.5	0.004	0.008	N/A	N/A	N/A
			0.5 - 1	0.005	0.004		N/A	N/A
			1 - 1.5	0.005	0.003		N/A	N/A
			1.5 - 2.5	0.003	0.003		N/A	N/A
			2.5 - 3.5	0.002	0.003		N/A	N/A
		34E-2	0 - 0.5	0.027	0.045	1734E-2	0.169	275.6
			0.5 - 1	0.038	0.042		0.168	300.0
			1 - 1.5	0.027	0.017		0.177	941.2
			1.5 - 2.5	N/A	0.006		0.061	916.7
			2.5 - 3.5	N/A	0.003		0.009	200.0
		34E-3	0 - 0.5	0.007	0.014	1734E-3	0.069	392.9
			0.5 - 1	0.002	0.004		0.022	450.0
			1 - 1.5	0.001	0.003		0.009	200.0
			1.5 - 2.5	0.002	0.003		0.008	166.7
			2.5 - 3.5	0.001	0.003		0.016	433.3

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)				
				1998 Study		2018 Study		
				Pre-ECE	Post-ECE	Sample ID	Chloride Level	Change (%)
34EB	AMOCO CFRP	34F-1	0 - 0.5	0.025	0.025	N/A	N/A	N/A
			0.5 - 1	0.020	0.012		N/A	N/A
			1 - 1.5	0.020	0.011		N/A	N/A
			1.5 - 2.5	0.004	0.003		N/A	N/A
			2.5 - 3.5	0.003	0.002		N/A	N/A
		34F-2	0 - 0.5	0.058	0.056	1734F-2	0.248	342.9
			0.5 - 1	0.057	0.042		0.315	650.0
			1 - 1.5	0.039	0.046		0.233	406.5
			1.5 - 2.5	0.022	0.028		0.142	407.1
			2.5 - 3.5	0.013	0.005		0.032	540.0
		34F-3	0 - 0.5	0.063	0.053	1734F-3	0.230	334.0
			0.5 - 1	0.073	0.035		0.394	1025.7
			1 - 1.5	0.049	0.029		0.329	1034.5
			1.5 - 2.5	N/A	0.018		0.185	927.8
			2.5 - 3.5	N/A	0.006		0.094	1466.7
	GFRP	34S-W1	0 - 0.5	0.026	0.031	1734S-W1	0.189	509.7
			0.5 - 1	0.020	0.026		0.177	580.8
			1 - 1.5	0.028	0.011		0.104	845.5
			1.5 - 2.5	0.006	0.005		0.039	680.0
			2.5 - 3.5	0.003	0.003		0.017	466.7
	AMOCO CFRP	34S-W2	0 - 0.5	0.057	0.043	1734S-W2	0.196	355.8
			0.5 - 1	0.043	0.045		0.192	326.7
			1 - 1.5	0.024	0.037		0.219	491.9
			1.5 - 2.5	0.022	0.020		0.158	690.0
			2.5 - 3.5	0.005	0.008		0.108	125.0
	Control	34S-W3	0 - 0.5	0.039	0.032	1734S-W3	0.184	475.0
			0.5 - 1	0.034	0.023		0.130	465.2
			1 - 1.5	0.025	0.016		0.095	493.8
			1.5 - 2.5	0.016	0.010		0.062	520.0
			2.5 - 3.5	0.010	0.003		0.024	700.0
	GFRP	34S-W4	0 - 0.5	0.033	0.029	N/A	N/A	N/A
			0.5 - 1	0.022	0.017		N/A	N/A
			1 - 1.5	0.013	0.007		N/A	N/A
			1.5 - 2.5	0.006	0.004		N/A	N/A
			2.5 - 3.5	0.004	0.002		N/A	N/A

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)				
				1998 Study		2018 Study		
				Pre-ECE	Post-ECE	Sample ID	Chloride Level	Change (%)
34EB	GFRP	34S-E1	0 - 0.5	0.023	0.027	N/A	N/A	N/A
			0.5 - 1	0.013	0.035		N/A	N/A
			1 - 1.5	0.011	0.015		N/A	N/A
			1.5 - 2.5	0.003	0.007		N/A	N/A
			2.5 - 3.5	0.003	0.002		N/A	N/A
	AMOCO CFRP	34S-E2	0 - 0.5	0.041	0.038	1734S-E2	0.203	434.2
			0.5 - 1	0.024	0.035		0.186	431.4
			1 - 1.5	0.030	0.027		0.213	688.9
			1.5 - 2.5	0.016	0.031		0.205	561.3
			2.5 - 3.5	0.012	0.008		0.178	2125.0
	Control	34S-E3	0 - 0.5	0.036	0.018	1734S-E3	0.163	805.6
			0.5 - 1	0.025	0.020		0.139	595.0
			1 - 1.5	0.016	0.012		0.097	708.3
			1.5 - 2.5	0.014	0.007		0.050	614.3
			2.5 - 3.5	0.009	0.004		0.023	475.0
	GFRP	34S-E4	0 - 0.5	0.029	0.027	1734S-E4	0.143	429.6
			0.5 - 1	0.021	0.020		0.128	540.0
			1 - 1.5	0.029	0.012		0.097	708.3
			1.5 - 2.5	0.016	0.004		0.036	800.0
			2.5 - 3.5	0.005	0.003		0.013	333.3

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)					
				1998 Study			2018 Study		
				Pre-ECE	Post-ECE	Change (%)	Sample ID	Chloride Level	Change (%)
37WB	ECE + MBrace CFRP	37A-1	0 - 0.5	0.007	0.002	-71.4	1737A-1	0.031	1450.0
			0.5 - 1	0.003	0.002	-33.3		0.023	1050.0
			1 - 1.5	0.003	0.028	833.3		0.018	-35.7
			1.5 - 2.5	0.002	0.004	100.0		0.009	125.0
			2.5 - 3.5	0.001	0.004	300.0		0.013	225.0
		37A-2	0 - 0.5	0.014	0.015	7.1	1737A-2	0.010	-33.3
			0.5 - 1	0.009	0.019	111.1		0.010	-47.4
			1 - 1.5	0.006	0.012	100.0		0.019	58.3
			1.5 - 2.5	0.005	0.007	40.0		0.008	14.3
			2.5 - 3.5	0.002	0.003	50.0		0.009	200.0
		37A-3	0 - 0.5	0.009	0.004	-55.6	N/A	N/A	N/A
			0.5 - 1	0.010	0.007	-30.0		N/A	N/A
			1 - 1.5	0.008	0.004	-50.0		N/A	N/A
			1.5 - 2.5	0.007	0.002	-71.4		N/A	N/A
			2.5 - 3.5	0.005	0.002	-60.0		N/A	N/A
	ECE + Sealer	37B-1	0 - 0.5	0.006	0.002	-66.7	1737B-1	0.016	700.0
			0.5 - 1	0.004	0.003	-25.0		0.012	300.0
			1 - 1.5	0.002	0.002	0.0		0.014	600.0
			1.5 - 2.5	0.002	0.004	100.0		0.008	100.0
			2.5 - 3.5	0.002	0.001	-50.0		0.009	800.0
		37B-2	0 - 0.5	0.016	0.010	-37.5	1737B-2	0.021	110.0
			0.5 - 1	0.023	0.027	17.4		0.021	-22.2
			1 - 1.5	0.029	0.025	-13.8		0.008	-68.0
			1.5 - 2.5	0.035	0.015	-57.1		0.008	-46.7
			2.5 - 3.5	0.019	0.009	-52.6		0.011	22.2
		37B-3	0 - 0.5	0.005	0.002	-60.0	N/A	N/A	N/A
			0.5 - 1	0.006	0.004	-33.3		N/A	N/A
			1 - 1.5	0.004	0.004	0.0		N/A	N/A
			1.5 - 2.5	0.001	0.002	100.0		N/A	N/A
			2.5 - 3.5	0.001	0.001	0.0		N/A	N/A

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)					
				1998 Study			2018 Study		
				Pre-ECE	Post-ECE	Change (%)	Sample ID	Chloride Level	Change (%)
37WB	ECE + Sealer	37C-1	0 - 0.5	0.024	0.001	-95.8	1737C-1	0.100	9900.0
			0.5 - 1	0.018	0.032	77.8		0.199	521.9
			1 - 1.5	0.014	0.021	50.0		0.170	709.5
			1.5 - 2.5	0.004	0.009	125.0		0.129	1333.3
			2.5 - 3.5	0.003	0.004	33.3		0.038	850.0
		37C-2	0 - 0.5	0.004	0.003	-25.0	N/A	N/A	N/A
			0.5 - 1	0.002	0.003	50.0		N/A	N/A
			1 - 1.5	0.001	0.003	200.0		N/A	N/A
			1.5 - 2.5	0.001	0.003	200.0		N/A	N/A
			2.5 - 3.5	0.002	0.003	50.0		N/A	N/A
		37C-3	0 - 0.5	0.006	0.003	-50.0	1737C-3	0.025	733.3
			0.5 - 1	0.005	0.005	0.0		0.021	320.0
			1 - 1.5	0.004	0.003	-25.0		0.013	333.3
			1.5 - 2.5	0.001	0.001	0.0		0.018	1700.0
			2.5 - 3.5	0.002	0.002	0.0		0.008	300.0
	ECE + MBrace CFRP	37N-W1	0 - 0.5	0.035	0.015	-57.1	1737N-W1	0.087	480.0
			0.5 - 1	0.035	0.022	-35.3		0.178	709.1
			1 - 1.5	0.029	0.011	-62.1		0.152	1281.8
			1.5 - 2.5	0.021	0.006	-71.4		0.128	2033.3
			2.5 - 3.5	0.008	0.003	-62.5		0.052	1633.3
	ECE + Sealer	37N-W2	0 - 0.5	0.038	0.040	5.3	N/A	N/A	N/A
			0.5 - 1	0.043	0.037	-14.0		N/A	N/A
			1 - 1.5	0.031	0.024	-22.6		N/A	N/A
			1.5 - 2.5	0.024	0.025	4.2		N/A	N/A
			2.5 - 3.5	0.018	0.014	-22.2		N/A	N/A
	ECE + Sealer	37N-W3	0 - 0.5	0.017	0.014	-17.6	1737N-W3	0.048	242.9
			0.5 - 1	0.021	0.021	0.0		0.094	347.6
			1 - 1.5	0.016	0.018	12.5		0.081	350.0
			1.5 - 2.5	0.006	0.008	33.3		0.039	387.5
			2.5 - 3.5	0.004	0.003	-25.0		0.015	400.0
	ECE + Sealer	37N-W4	0 - 0.5	0.032	0.016	-50.0	1737N-W4	0.081	623.1
			0.5 - 1	0.032	0.017	-46.9		0.128	1242.9
			1 - 1.5	0.028	0.013	-53.6		0.094	2100.0
			1.5 - 2.5	0.018	0.007	-61.1		0.094	464.7
			2.5 - 3.5	0.010	0.003	-70.0		0.066	2100.0

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)					
				1998 Study			2018 Study		
				Pre-ECE	Post-ECE	Change (%)	Sample ID	Chloride Level	Change (%)
37WB	ECE + Sealer	37N-E1	0 - 0.5	0.080	0.017	-78.8	1737N-E1	0.096	464.7
			0.5 - 1	0.044	0.031	-29.5		0.123	296.8
			1 - 1.5	0.044	0.027	-38.6		0.148	448.1
			1.5 - 2.5	0.040	0.016	-60.0		0.108	575.0
			2.5 - 3.5	0.032	0.003	-90.6		0.039	1200.0
	ECE + Sealer	37N-E2	0 - 0.5	0.018	0.026	44.4	1737N-E2	0.138	430.8
			0.5 - 1	0.028	0.028	0.0		0.152	442.9
			1 - 1.5	0.025	0.007	-72.0		0.150	2042.9
			1.5 - 2.5	0.041	0.014	-65.9		0.113	707.1
			2.5 - 3.5	0.029	0.017	-41.4		0.058	241.2
	ECE + MBrace CFRP	37N-E3	0 - 0.5	0.032	0.016	-50.0	1737N-E3	0.051	218.8
			0.5 - 1	0.017	0.018	5.9		0.073	305.6
			1 - 1.5	0.013	0.018	38.5		0.037	105.6
			1.5 - 2.5	0.011	0.008	-27.3		0.022	175.0
			2.5 - 3.5	0.008	0.003	-62.5		0.019	533.3
	ECE + Sealer	37N-E4	0 - 0.5	0.016	0.010	-37.5	N/A	N/A	N/A
			0.5 - 1	0.041	0.017	-58.5		N/A	N/A
			1 - 1.5	0.041	0.011	-73.2		N/A	N/A
			1.5 - 2.5	0.044	0.003	-93.2		N/A	N/A
			2.5 - 3.5	0.037	0.002	-94.6		N/A	N/A

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)				
				1998 Study		2018 Study		
				Pre-ECE	Post-ECE	Sample ID	Chloride Level	Change (%)
37EB	Control	37D-1	0 - 0.5	0.009	0.009	1737D-1	0.078	766.7
			0.5 - 1	0.003	0.005		0.052	940.0
			1 - 1.5	0.002	0.003		0.030	900.0
			1.5 - 2.5	0.002	0.002		0.015	650.0
			2.5 - 3.5	0.002	0.003		0.019	533.3
		37D-2	0 - 0.5	0.021	0.033	1737D-2	0.116	251.5
			0.5 - 1	0.045	0.051		0.330	547.1
			1 - 1.5	0.059	0.052		0.314	503.8
			1.5 - 2.5	0.038	0.031		0.192	519.4
			2.5 - 3.5	0.020	0.014		0.082	485.7
		37D-3	0 - 0.5	0.036	0.023	N/A	N/A	N/A
			0.5 - 1	0.031	0.029		N/A	N/A
			1 - 1.5	0.036	0.013		N/A	N/A
			1.5 - 2.5	0.028	0.008		N/A	N/A
			2.5 - 3.5	0.019	0.002		N/A	N/A
		37S-W1	0 - 0.5	0.039	0.027	1737S-W1	0.145	437.0
			0.5 - 1	0.030	0.020		0.100	400.0
			1 - 1.5	0.014	0.015		0.077	413.3
			1.5 - 2.5	0.015	0.006		0.018	200.0
			2.5 - 3.5	0.006	0.006		0.012	100.0
		37S-W2	0 - 0.5	0.041	0.014	N/A	N/A	N/A
			0.5 - 1	0.024	0.006		N/A	N/A
			1 - 1.5	0.030	0.003		N/A	N/A
			1.5 - 2.5	0.016	0.003		N/A	N/A
			2.5 - 3.5	0.012	0.003		N/A	N/A
		37S-E1	0 - 0.5	0.035	0.024	N/A	N/A	N/A
			0.5 - 1	0.027	0.016		N/A	N/A
			1 - 1.5	0.019	0.005		N/A	N/A
			1.5 - 2.5	0.014	0.004		N/A	N/A
			2.5 - 3.5	0.005	0.004		N/A	N/A
		37S-E2	0 - 0.5	0.016	0.012	N/A	N/A	N/A
			0.5 - 1	0.011	0.005		N/A	N/A
			1 - 1.5	0.005	0.003		N/A	N/A
			1.5 - 2.5	0.003	0.003		N/A	N/A
			2.5 - 3.5	0.002	0.003		N/A	N/A

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)				
				1998 Study		2018 Study		
				Pre-ECE	Post-ECE	Sample ID	Chloride Level	Change (%)
40WB	MBrace CFRP	40A-1	0 - 0.5	0.037	0.028	N/A	N/A	N/A
			0.5 - 1	0.024	0.021		N/A	N/A
			1 - 1.5	0.012	0.013		N/A	N/A
			1.5 - 2.5	0.003	0.008		N/A	N/A
			2.5 - 3.5	0.003	0.008		N/A	N/A
		40A-2	0 - 0.5	0.018	0.030	1740A-2	0.215	616.7
			0.5 - 1	0.011	0.027		0.198	633.3
			1 - 1.5	0.006	0.017		0.165	870.6
			1.5 - 2.5	0.003	0.008		0.079	887.5
			2.5 - 3.5	0.001	0.002		0.019	850.0
		40A-3	0 - 0.5	0.035	0.039	1740A-3	0.185	374.4
			0.5 - 1	0.026	0.035		0.322	820.0
			1 - 1.5	0.023	0.029		0.288	893.1
			1.5 - 2.5	0.015	0.019		0.298	1468.4
			2.5 - 3.5	0.005	0.005		0.110	2100.0
	Control	40C-1	0 - 0.5	0.075	0.047	N/A	N/A	N/A
			0.5 - 1	0.053	0.048		N/A	N/A
			1 - 1.5	0.030	0.006		N/A	N/A
			1.5 - 2.5	0.026	0.048		N/A	N/A
			2.5 - 3.5	0.019	0.029		N/A	N/A
		40C-2	0 - 0.5	0.013	0.012	1740C-2	0.059	391.7
			0.5 - 1	0.006	0.005		0.021	320.0
			1 - 1.5	0.003	0.003		0.015	400.0
			1.5 - 2.5	0.002	0.006		0.008	33.3
			2.5 - 3.5	0.002	0.003		0.011	266.7
		40C-3	0 - 0.5	0.003	0.006	1740C-3	0.029	383.3
			0.5 - 1	0.002	0.002		0.016	700.0
			1 - 1.5	0.002	0.003		0.012	300.0
			1.5 - 2.5	0.002	0.003		0.012	300.0
			2.5 - 3.5	0.002	0.002		0.004	100.0
	MBrace CFRP	40N-W1	0 - 0.5	0.045	0.035	N/A	N/A	N/A
			0.5 - 1	0.038	0.043		N/A	N/A
			1 - 1.5	0.038	0.030		N/A	N/A
			1.5 - 2.5	0.020	0.020		N/A	N/A
			2.5 - 3.5	0.011	0.009		N/A	N/A

Pier	Treatment	Sample ID	Depth (in.)	Acid-Soluble Chloride Concentration Results (% by mass of concrete)				
				1998 Study		2018 Study		
				Pre-ECE	Post-ECE	Sample ID	Chloride Level	Change (%)
40WB	MBrace CFRP	40N-W2	0 - 0.5	0.044	0.037	1740N-W2	0.205	454.1
			0.5 - 1	0.041	0.035		0.288	747.1
			1 - 1.5	0.032	0.030		0.227	656.7
			1.5 - 2.5	0.017	0.021		0.179	752.4
			2.5 - 3.5	0.009	0.006		0.079	1216.7
	Control	40N-W3	0 - 0.5	0.017	0.020	1740N-W3	0.153	665.0
			0.5 - 1	0.021	0.024		0.226	841.7
			1 - 1.5	0.035	0.022		0.248	1027.3
			1.5 - 2.5	0.021	0.013		0.183	1307.7
			2.5 - 3.5	0.010	0.003		0.122	3966.7
	Control	40N-W4	0 - 0.5	0.023	0.023	1740N-W4	0.120	421.7
			0.5 - 1	0.017	0.023		0.160	595.7
			1 - 1.5	0.013	0.018		0.110	511.1
			1.5 - 2.5	0.013	0.004		0.065	1525.0
			2.5 - 3.5	N/A	0.007		0.015	114.3
	MBrace CFRP	40N-E1	0 - 0.5	0.041	0.041	N/A	N/A	N/A
			0.5 - 1	0.028	0.044		N/A	N/A
			1 - 1.5	0.028	0.031		N/A	N/A
			1.5 - 2.5	0.017	0.019		N/A	N/A
			2.5 - 3.5	0.012	0.006		N/A	N/A
	MBrace CFRP	40N-E2	0 - 0.5	0.051	0.028	N/A	N/A	N/A
			0.5 - 1	0.045	0.033		N/A	N/A
			1 - 1.5	0.046	0.033		N/A	N/A
			1.5 - 2.5	0.024	0.014		N/A	N/A
			2.5 - 3.5	0.017	0.038		N/A	N/A
	Control	40N-E3	0 - 0.5	0.025	0.031	N/A	N/A	N/A
			0.5 - 1	0.035	0.035		N/A	N/A
			1 - 1.5	0.020	0.032		N/A	N/A
			1.5 - 2.5	0.020	0.024		N/A	N/A
			2.5 - 3.5	0.006	0.011		N/A	N/A
	Control	40N-E4	0 - 0.5	0.033	0.018	N/A	N/A	N/A
			0.5 - 1	0.026	0.033		N/A	N/A
			1 - 1.5	0.016	0.026		N/A	N/A
			1.5 - 2.5	0.010	0.009		N/A	N/A
			2.5 - 3.5	0.004	0.006		N/A	N/A

APPENDIX B: INSTRUMENTATION DATA

Pier 34N

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T
2						Resistivity Probes (Q)						Half-Cells (V)								
3						Probe 1			Probe 2			Probe 1			Relative Humidity (%)			Ambient Temp (deg C)		
4	Temp. {°F	RH			Date	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
5	30		Pier 34N	Column A	1/4/2000	1.1	0.9	2700000	0.9	0.9	0.7	-0.026	-0.057	0.396	0	0	0	0	0	0
6	67	38%	Pier 34N	Column A	4/24/2000	1.1	0.9	1400000	1	1	0.8	-0.056	-0.074	0.386	56.4	35.7	63.5	17.1	16.3	15.6
7	74	44%	Pier 34N	Column A	6/26/2000	1.2	1	1500000	1.1	1	0.8	-0.075	-0.181	0.387	65.5	43.1	37	22.1	22.3	24.1
8	79	69%	Pier 34N	Column A	8/24/2000	1.2	1	1500000	1.1	1.1	0.9	-0.058	-0.127	0.402	70.2	73	74.9	25.3	23.4	23.2
9	57	58%	Pier 34N	Column A	11/2/2000	1.2	1	2000000	1.1	0.9	0.7	-0.039	-0.081	0.407	58	54	64.5	13.3	11.7	11.5
10	43	69%	Pier 34N	Column A	3/20/2001	1	0.9	1700000	0.9	0.9	0.8	-0.029	-0.056	0.401	62	64.6	63.5	4.2	4	3.5
11	59	52%	Pier 34N	Column A	5/31/2001	1.3	1	1500000	1	0.9	0.8	-0.042	-0.063	0.411	61.4	46.7	60.5	18	16.2	16
12	73	76%	Pier 34N	Column A	8/3/2001	1.2	1	1300000	1.1	1	0.8	-0.051	-0.072	0.39	67	68.9	66.3	24.6	24.9	26.2
13	39	75%	Pier 34N	Column A	10/17/2001	1.1	0.9	2000000	1.1	0.9	0.8	-0.027	-0.045	0.276	56	63.6	65.6	3.5	2.3	2.3
14	30		Pier 34N	Column A	12/20/2001	1	0.9	3000000	2500000	0.9	0.8	-0.026	-0.046	0.24	0	0	0	0	0	0
15	26		Pier 34N	Column A	2/21/2002	1.1	1	3000000	2200000	0.9	0.8	-0.019	-0.02	0.283	0	0	0	0	0	0
16	46	80%	Pier 34N	Column A	4/29/2002	1.1	0.9	2500000	2000000	0.9	0.8	-0.027	-0.034	0.365	74.6	64.2	66.1	5.9	4.6	4.4
17	72	78%	Pier 34N	Column A	6/20/2002	1.2	1	1900000	1500000	1	0.9	-0.036	-0.05	0.346	61.3	69.1	66.8	23.1	21.8	21.2
18	69	59%	Pier 34N	Column A	9/3/2002	1.2	1	1800000	1500000	1	0.9	-0.034	-0.046	0.335	45.5	72.5	66.8	21.4	20.4	20.5
19	22		Pier 34N	Column A	11/5/2002	1.2	1	2800000	2000000	0.9	0.8	-0.022	-0.028	0.273	0	0	0	0	0	0
20	25		Pier 34N	Column A	1/28/2003	1.1	14.9	3000000	2000000	0.3	0.7	-0.014	-0.027	0.297	0	0	0	0	0	0
21	18		Pier 34N	Column A	3/7/2003	1	24000	2800000	2500000	0.9	0.7	-0.015	-0.022	0.227	0	0	0	0	0	0
22	44	75%	Pier 34N	Column A	5/20/2003	1.1	2700000	1900000	1.7	1	0.8	-0.025	-0.034	0.293	55.7	56.7	64.2	9.8	8.7	8.8
23	66	78%	Pier 34N	Column A	7/11/2003	1.3	1900000	1700000	1400000	0.9	0.9	-0.028	-0.036	0.372	69.5	68.6	66.6	19.9	18.2	18.3
24	47	73%	Pier 34N	Column A	9/25/2003	1.2	1500000	2300000	1700000	0.9	0.8	-0.018	-0.026	0.301	56.8	55.3	59	9.2	8.1	8.5
25	24		Pier 34N	Column A	11/13/2003	1.1	10.8	3500000	2300000	0.9	0.7	-0.013	-0.016	0.232	0	0	0	0	-3	0
26	24		Pier 34N	Column A	11/13/2003	1.1	10.8	3500000	2300000	0.9	0.7	-0.013	-0.016	0.232	0	0	0	0	-3	0
27	30		Pier 34N	Column A	2/19/2004	1.2	2300000	2800000	2100000	0.9	0.7	-0.012	-0.012	0.234	0	0	0	0	0	0
28	43	78%	Pier 34N	Column A	3/29/2004	1.2	2300000	2500000	2200000	0.9	0.7	-0.01	-0.002	0.332	83.4	81.9	82.1	6.3	5.3	5.2
29	59	68%	Pier 34N	Column A	6/2/2004	1.2	1500000	1800000	1600000	0.9	0.9	-0.018	-0.022	0.305	77.5	66.3	64	14.6	13.2	19.1
30	71	88%	Pier 34N	Column A	8/26/2004	NR	1500000	1650000	1300000	0.9	0.9	-0.027	-0.039	0.306	70.8	71.7	69.9	23.7	22	21.9
31	50	94%	Pier 34N	Column A	10/20/2004	0.603	1900000	2300000	NR	1500000	0.6	-0.017	-0.026	0.24	65.9	68.1	66.3	8.4	6.9	6.8
32	0		Pier 34N	Column A	12/15/2004	NR	3.1	2700000	2000000	0.9	0.7	-0.007	-0.008	0.2	0	0	0	0	0	0
33	11		Pier 34N	Column A	2/23/2005	5000000	500000	2800000	NR	2400000	0.9	-0.013	-0.015	0.164						
34	48	65%	Pier 34N	Column A	4/22/2005	NR	2000000	1800000	NR	1800000	1	-0.023	-0.3	0.178		48.8	44		10.3	10.4
35	72	77%	Pier 34N	Column A	7/22/2005	NR	1200000	1100000	NR	1350000	1	-0.037	-0.048	0.243	0	59.6	59.9	0	25.8	25.9
36	64	65%	Pier 34N	Column A	9/16/2005	0	1400000	1200000	0	1200000	0.9	-0.03	-0.043	0.197	0	59.6	57.7	0	17.1	17.3
37	15		Pier 34N	Column A	12/21/2005	NR	2400000	4500000	2400000	0.9	0.7	-0.004	0	0.197	0	0	0	0	0	0
38	22		Pier 34N	Column A	2/28/2006	NR	1100000	2300000	1500000	0.9	0.8	-0.018	-0.032	0.153	0	0	0	0	0	0
39	56	46%	Pier 34N	Column A	5/22/2006	NR	1250000	1360000	1320000	1.1	0.8	-0.021	-0.038	0.078	0	31.3	52.5	0	12.1	11.9
40	86	37%	Pier 34N	Column A	7/18/2006	NR	1400000	1200000	1300000	1.3	0.9	-0.032	-0.052	0.091	0	39.5	50.1	0	26.9	26.9
41	55	88%	Pier 34N	Column A	9/12/2006	1800000	1700000	1700000	1.5e	0.9	0.9	-0.013	-0.025	0.1	0	63.9	63.1	0	12.8	12.7
42	57	72%	Pier 34N	Column A	11/8/2006	NR	1600000	e.8e6	NR	1500000	0.9	-0.018	-0.032	0.082	0	58.8	57.7	0	12.5	12
43	34	82%	Pier 34N	Column A	1/8/2007	No Info - B	0	0	0	0	0	0	0	0	0	0	0	0	0	0
44	36	78%	Pier 34N	Column A	3/12/2007	4200000	1750000	2400000	NR	1700000	0.9	-0.014	-0.017	0.158	NR	NR	NR	0	0	0
45	74	62%	Pier 34N	Column A	6/1/2007	4000000	0.2	NR	18000000	0.2	NR	NR	NR	NR	0	0	0	0	0	0
46	54	76%	Pier 34N	Column A	10/19/2007	No Info - B	0	0	0	0	0	0	0	0	0	0	0	0	0	0
47	30		Pier 34N	Column A	12/3/2007	1600000	1800000	2900000	29800000	1800000	1	-0.005	-0.006	0.146	0	0	0	0	0	0
48	48	75%	Pier 34N	Column A	4/29/2009	No Info - B	0	0	0	0	0	0	0	0	0	0	0	0	0	0

	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
2		Resistivity Probes (Q)						Half-Cells (V)								
3		Probe 1			Probe 2			Probe 1			Relative Humidity (%)			Ambient Temp (deg C)		
4		1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
5	Column B	0.6	0.5	0.5	0.7	0.5	0.6	-0.05	-0.028	-0.042						
6	Column B	0.6	0.6	0.5	0.8	0.6	0.5	-0.063	-0.054	-0.062	64.2	53.8	39.5	16.4	15.3	15.4
7	Column B	0.6	0.4	0.5	0.8	0.6	0.5	-0.067	-0.058	-0.068	38.9	44.9	42.8	24.3	22.5	22.1
8	Column B	0.6	0.5	0.5	0.9	0.6	0.5	-0.069	-0.058	-0.072	70.6	73.5	71.5	25.1	23.6	23.2
9	Column B	0.7	0.6	0.5	0.7	0.6	0.5	-0.066	-0.047	-0.058	53.8	50.8	52.5	13.3	11.7	11.6
10	Column B	0.7	0.6	0.6	0.8	0.6	0.5	-0.056	-0.036	-0.046	63.2	67.9	65.8	4.5	4.1	4
11	Column B	0.7	0.5	0.5	0.8	0.6	0.6	-0.066	-0.046	-0.058	55.4	57.1	50.2	17.6	10.2	15.9
12	Column B	0.6	0.5	0.5	0.8	0.5	0.5	-0.068	-0.054	-0.066	64.7	70.8	64.7	24.6	25	26.4
13	Column B	0.7	0.6	0.5	0.8	0.6	0.5	-0.059	-0.029	-0.047	58	63.1	74.4	4.3	2.7	2.5
14	Column B	0.6	0.5	0.5	0.8	0.5	0.5	-0.058	-0.025	-0.036						
15	Column B	0.6	0.5	0.5	0.8	0.5	0.5	-0.057	0.01	-0.042						
16	Column B	0.6	0.5	0.5	0.8	0.5	0.5	-0.054	-0.012	-0.044	70.3	68.6	66.9	6.3	4.9	4.8
17	Column B	0.6	0.5	0.5	0.9	0.6	0.6	-0.058	-0.038	-0.053	60.4	60.6	51.7	22.8	21.4	21.4
18	Column B	0.7	0.6	0.5	0.9	0.5	0.5	-0.058	-0.036	-0.05	52.7	55	53.5	21.6	20.1	20.2
19	Column B	0.6	0.4	0.5	0.8	0.5	0.5	-0.049	0.054	-0.03						
20	Column B	0.6	0.5	0.4	0.7	0.5	0.4	-0.037	0.083	-0.027						
21	Column B	0.5	0.5	0.4	0.7	0.5	0.5	-0.038	0.129	-0.026						
22	Column B	0.6	0.4	0.5	0.8	0.5	0.5	-0.052	0.151	-0.04	62.8	58.6	53.5	9.8	8.3	8
23	Column B	0.7	0.6	0.5	0.9	0.6	0.6	-0.052	0.173	-0.042	66.4	70.1	64.4	20.1	18.3	18.3
24	Column B	0.6	0.5	0.5	0.8	0.6	0.5	-0.047	0.154	-0.034	56.3	63.1	66.5	10.2	8.3	8.3
25	Column B	0.6	0.4	0.4	0.7	0.5	0.5	-0.043	0.147	-0.026						
26	Column B	0.6	0.4	0.4	0.7	0.5	0.5	-0.043	0.147	-0.026						
27	Column B	0.6	0.6	0.5	0.7	0.6	0.5	-0.027	-0.145	-0.022						
28	Column B	0.6	0.6	0.5	0.7	0.5	0.5	-0.05	0.107	-0.036	79.8	73.5	83.1	6.3	5	5.3
29	Column B	0.6	0.6	0.5	0.9	0.5	0.6	-0.048	0.108	-0.034	66.4	68.8	66.7	14.4	12.9	12.8
30	Column B	0.7	0.6	0.6	0.9	0.5	0.5	-0.046	0.137	-0.037	68.8	70.8	71.9	23.4	21.9	21.8
31	Column B	0.6	0.6	0.5	0.8	0.6	0.5	-0.039	0.16	-0.024	70.6	69.2	70.2	8.6	6.8	6.9
32	Column B	0.6	0.6	0.5	0.7	0.5	0.5	-0.037	0.176	-0.013						
33	Column B	0.7	0.6	0.5	0.8	0.6	0.4	-0.035	0.192	-0.015						
34	Column B	0.7	0.5	0.5	0.8	0.6	0.5	-0.039	0.141	-0.032		61	54		10.2	10.1
35	Column B	0.7	0.6	0.5	0.9	0.6	0.5	-0.044	0.125	-0.049		68	60		25.6	25.5
36	Column B	0.7	0.4	0.5	0.8	0.6	0.5	-0.04	0.148	-0.045		66.4	59.4		17.5	17.1
37	Column B	0.6	0.4	0.5	0.7	0.5	0.5	-0.035	0.24	0.011						
38	Column B	0.7	0.6	0.5	0.8	0.6	0.6	-0.031	0.226	-0.015						
39	Column B	0.7	0.5	0.4	0.8	0.5	0.5	-0.038	0.154	-0.028		40.3	36.8		11.5	11.9
40	Column B	1	1	0.4	0.9	0.7	0.6	-0.038	0.154	-0.043		61.6	49.2		26.7	26.7
41	Column B	0.6	0.6	0.4	0.9	0.6	0.6	-0.035	0.146	-0.028		65.7	63.2		12.4	12.3
42	Column B	0.6	0.6	0.5	0.9	0.6	0.5	-0.032	0.222	-0.021		63.9	57.3		11.9	12.1
43	Column B	0	0	0	0	0	0	0	0	0						
44	Column B	0.6	0.5	0.5	0.8	0.6	0.6	-0.034	0.219	-0.017	NR	NR	NR			
45	Column B	NR	11000000	NR	NR	NR	NR	NR	NR	NR						
46	Column B	0	0	0	0	0	0	0	0	0						
47	Column B	0.6	0.6	0.5	0.8	0.7	0.5	-0.033	0.216	-0.009						
48	Column B	0	0	0	0	0	0	0	0	0						

	AK	AL	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ
2		Resistivity Probes (Q)						Half-Cells (V)								
3		Probe 1			Probe 2			Probe 1			Relative Humidity (%)			Ambient Temp (deg C)		
4		1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
5	Column C	4500000	1	2500000	1.1	1	0.9	-0.046	-0.04	-0.058						
6	Column C	2100000	1.1	0.53	1.2	1.2	1	-0.054	0.228	-0.078	50.6	40.8	55.3	15.2	15.5	16
7	Column C	2000000	1.1	1300000	1.3	1.2	1.1	-0.058	0.148	-0.078	41.7	45.8	47.7	23.6	22.2	22.1
8	Column C	2000000	1.2	1100000	1.3	1.2	1	-0.064	0.097	-0.077	28.3	73.9	71.3	24.9	23.5	23.2
9	Column C	2000000	1.1	1500000	1.2	1.1	0.9	-0.064	0.139	-0.073	59.1	42.4	48.9	13.5	11.9	11.6
10	Column C	1900000	1.1	2400000	1	1.1	0.8	-0.064	-0.004	-0.079	73.5	64.2	69.4	4.4	4.2	4.3
11	Column C	3000000	1.2	1600000	1.1	1.5	1	-0.066	0.086	-0.083	54.8	57.3	54.8	17.5	15.9	16
12	Column C	5000000	1.2	1100000	1.2	1.2	0.9	-0.065	0.092	-0.088	62.7	68	64.7	26.4	24.6	24.9
13	Column C	10200000	1	2.5	1	1.1	0.9	-0.061	-0.003	-0.063	60.1	62.1	61.4	4.8	3.2	2.9
14	Column C	3500000	1	3000000	0.9	1	0.9	-0.066	-0.011	-0.057						
15	Column C	8000000	1	3000000	1	0.9	0.9	-0.06	-0.015	-0.058						
16	Column C	5000000	1	2500000	1.1	1.1	0.9	-0.054	-0.022	-0.065	66.1	68.4	68.8	6.2	4.8	5.2
17	Column C	3500000	1.1	1500000	1.2	1.1	0.9	-0.054	-0.032	-0.07	60.9	65.9	56.8	22.9	21.6	21.2
18	Column C	2000000	1.2	1200000	1.2	1.2	1	-0.059	-0.057	-0.069	53.8	60.9	49.5	21.6	20.3	19.9
19	Column C	8500000	0.9	3	1	0.9	0.9	-0.063	-0.07	-0.049						
20	Column C	13000000	0.9	3000000	1	1	0.9	-0.052	-0.097	-0.043						
21	Column C	8000000	1	3500000	1	1	0.9	-0.044	-0.092	-0.085						
22	Column C	3500000	1.2	1500000	1.2	1.1	0.9	-0.062	-0.071	-0.09	55	63.3	54.3	10.2	8.9	8.4
23	Column C	3000000	1.2	1200000	1.2	1.2	0.9	-0.056	-0.078	-0.075	63.3	69.8	64.7	20.1	18.3	18.5
24	Column C	6000000	1.1	1500000	1.1	1	0.9	-0.059	-0.09	-0.06	34.5	52.3	51.5	10.2	8.6	8.4
25	Column C	9500000	1	2500000	1	0.9	0.9	-0.061	-0.042	-0.065						
26	Column C	9500000	1	2500000	1	0.9	0.9	-0.061	-0.042	-0.065						
27	Column C	10000000	1.1	2500000	1	1.1	0.9	-0.035	-0.106	-0.089						
28	Column C	3300000	1.1	1500000	1.1	1.1	0.9	-0.073	-0.184	-0.101	87.7	69.3	50.9	6.7	5.6	5.2
29	Column C	3500000	1.2	1100000	1.2	1.1	0.9	-0.065	-0.03	-0.083	62.4	65	64	14	12.5	12.4
30	Column C	2500000	1.2	900000	1.2	1.2	1	-0.058	0.008	-0.069	66.1	70.9	67.5	23.3	21.8	21.7
31	Column C	5200000	1.1	1300000	1.1	1	0.9	-0.056	0.02	-0.06	68.7	67.3	67.7	8.7	7	6.8
32	Column C	5500000	1	2500000	1	1	0.9	-0.062	-0.034	-0.066						
33	Column C	8000000	0.9	2500000	0.9	0.9	0.9	-0.06	-0.009	-0.093						
34	Column C	3700000	1.2	1	1.1	1.1	0.9	-0.057	-0.087	-0.101		50	59.8		10.1	10.1
35	Column C	3000000	1.2	600000	1.2	1.2	1	-0.054	-0.063	-0.09		59.7	53.8		25.6	25.8
36	Column C	2600000	1.1	750000	1.2	1.2	1	-0.057	-0.071	-0.102		58.1	53.9		17.1	17.3
37	Column C	9400000	0.9	S.5E6	0.9	0.9	0.9	-0.054	-0.115	-0.102						
38	Column C	7000000	1	950000	1	1	0.9	-0.052	-0.13	0.126						
39	Column C	5000000	1	1100000	1000000	1.1	0.9	-0.054	-0.061	-0.14		33.9	47.2		11.7	11.3
40	Column C	3000000	1400000	1200000	3500000	1.3	2.5	-0.041	-0.047	-0.038		39.2	39.8		26.4	26.5
41	Column C	4400000	1	1200000	7500000	1.2	1100000	-0.053	-0.04	-0.138		64.1	61.9		12.2	12.2
42	Column C	4500000	1	1100000	2800000	1.2	1300000	-0.044	-0.094	-0.123		59.5	55.4		12.3	11.7
43	Column C	0	0	0	0	0	0	0	0	0						
44	Column C	5500000	1	55000	3500000	1.1	300000	-0.058	-0.119	-0.34	NR	NR	NR			
45	Column C	0.6	0.7	0.4	0.7	0.7	29000000	NR	NR	NR						
46	Column C	0	0	0	0	0	0	0	0	0						
47	Column C	8500000	0.9	1300000	1800000	1	2200000	-0.046	-0.077	-0.128						
48	Column C	0	0	0	0	0	0	0	0	0						

	BA	BB	BC	BD	BE	BF	BG	BH	BI	BJ	BK	BL	BM	BN	BO	BP	BQ	BR
2		Resistivity Probes (Q)						Half-Cells (V)			Resistivity Probes (Q)						Half-Cells (V)	
3		Probe 1			Probe 2			Probe 1			Probe 1			Probe 2			Probe 1	
4		1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2
5	West face	0.7	0.5		0.8	0.5			-0.043	East Face	0.8	0.9		0.8	0.9		-0.057	-0.021
6	West face	0.8	0.6		0.8	0.6			-0.059	East Face	0.8	0.9		0.9	0.9		-0.059	-0.106
7	West face	0.7	0.6		0.8	0.6			-0.066	East Face	0.8	0.9		0.8	1		-0.065	-0.098
8	West face	0.8	0.6		0.9	0.6			-0.064	East Face	.8.	9		0.9	1		-0.059	-0.088
9	West face	0.8	0.5		0.9	0.6			-0.058	East Face	0.8	0.9		0.9	1.1		-0.047	-0.078
10	West face	0.8	0.6		0.8	0.6			-0.05	East Face	0.8	0.9		0.8	0.9		-0.058	-0.101
11	West face	0.8	0.6		0.9	0.6			-0.057	East Face	0.8	0.9		0.9	1		-0.058	-0.102
12	West face	0.8	0.6		0.9	0.6			-0.061	East Face	0.8	1		0.9	1		-0.065	-0.094
13	West face	0.8	0.6		0.8	0.6			-0.049	East Face	0.8	0.9		0.8	0.9		-0.043	-0.082
14	West face	0.6	0.6		0.8	0.6			-0.036	East Face	0.8	0.9		0.8	0.9		-0.028	-0.11
15	West face	0.7	0.6		0.8	0.6			-0.048	East Face	0.8	0.9		0.8	0.9		-0.046	-0.112
16	West face	0.8	0.6		0.8	0.6			-0.047	East Face	0.7	0.9		0.8	0.9		-0.043	-0.089
17	West face	0.8	0.6		0.9	0.6			-0.045	East Face	0.8	0.9		0.9	0.9		-0.05	-0.1
18	West face	0.8	0.5		0.8	0.6			-0.037	East Face	0.8	0.9		0.9	1		-0.047	-0.091
19	West face	0.7	0.8		0.8	0.6			-0.029	East Face	0.7	0.9		0.8	1		-0.023	-0.086
20	West face	0.7	0.6		0.7	0.5			-0.034	East Face	0.7	0.8		0.8	0.9		-0.026	-0.047
21	West face	0.7	0.5		0.7	0			-0.036	East Face	0.5	0.7		0.7	0.9		-0.053	-0.089
22	West face	0.7	0.6		0.8	0.6			-0.034	East Face	0.8	0.9		0.8	1		-0.04	-0.08
23	West face	0.9	0.7		0.9	0.7			-0.031	East Face	0.9	1.2		0.9	1		-0.041	-0.012
24	West face	0.8	0.6		0.8	0.6			-0.028	East Face	0.8	0.8		0.8	0.9		-0.03	-0.001
25	West face	0.7	0.6		0.8	0.6			-0.025	East Face	0.8	0.9		0.8	0.9		-0.021	-0.001
26	West face	0.7	0.6		0.8	0.6			-0.025	East Face	0.8	0.9		0.8	0.9		-0.021	-0.001
27	West face	1.1	0.6		0.8	0.6			-0.03	East Face	0.7	0.9		0.8	0.9		-0.051	-0.002
28	West face	0.7	0.5		0.7	0.6			-0.038	East Face	0.8	0.9		0.8	0.9		-0.03	-0.032
29	West face	0.8	0.6		0.9	0.6			-0.025	East Face	0.8	0.9		0.9	0.9		-0.031	-0.019
30	West face	0.8	0.6		0.9	0.6			-0.025	East Face	0.8	0.9		0.9	1		-0.03	-0.013
31	West face	0.7	0.6		0.8	0.5			-0.022	East Face	0.8	0.9		0.8	1		-0.017	0
32	West face	0.7	0.6		0.8	0.6			-0.017	East Face	0.8	0.9		0.9	0.9		-0.017	-0.005
33	West face	0.7	0.6		0.8	0.6		-0.036		East Face	0.7	0.9		0.8	0.9		-0.008	0
34	West face	0.8	0.6		0.8	0.6		-0.033		East Face	0.8	0.9		0.9	0.9		-0.033	-0.001
35	West face	0.8	0.6		0.9	0.6		-0.033	-0.047	East Face	0.8	1		0.8	1		-0.001	
36	West face	0.8	0.6		0.8	0.6		-0.029	-0.035	East Face	0.8	1		0.9	1		-0.005	
37	West face	0.7	0.5		0.8	0.5			-0.026	East Face	0.7	0.8		0.7	0.9		-0.047	-0.002
38	West face	0.8	0.6		0.7	0.6			-0.03	East Face	0.8	0.9		0.8	1		-0.033	-0.003
39	West face	0.7	0.5		0.8	0.5			-0.028	East Face	1	0.9		0.8	1		-0.03	0.017
40	West face	0.8	0.6		0.9	0.7			-0.031	East Face	0.9	0.9		0.9	1		-0.045	-0.001
41	West face	0.7	0.6		0.8	0.6			-0.023	East Face	0.8	0.9		0.9	1		-0.003	-0.003
42	West face	0.7	0.6		0.8	0.6		-0.021	-0.027	East Face	0.8	0.9		0.9	0.9		0	
43	West face	0	0		0	0				East Face	0	0		0	0			
44	West face	0.7	0.6		0.8	0.6		-0.027	-0.049	East Face	0.7	0.9		0.9	0.9		-0.009	
45	West face	NR	NR		NR	20000000			NR	East Face	0.2	0.4		0.4	NR		-0.197	NR
46	West face	0	0		0	0				East Face	0	0		0	0			
47	West face	0.6	0.6		0.8	0.6		-0.021	-0.03	East Face	0.8	0.9		0.8	0.9		-0.004	
48	West face	0	0		0	0				East Face	0	0		0	0			

	BS	BT	BU	BV	BW	BX	BY	BZ	CA	CB	CC	CD	CE	CF	CG	CH	CI	CJ	CK
1	Pier 34S																		
2	Pier 34S				Resistivity Probes (Q)						Half-Cells (V)								
3					Probe 1			Probe 2			Probe 1			Relative Humidity (%)			Ambient Temp (deg C)		
4			Temp. {°F	Date	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
5	Pier 34S	Column D	30	1/4/2000	1.1	0.9	0.9	1.2	0.9	0.9	-0.071	-0.033	-0.035						
6	Pier 34S	Column D	65	4/24/2000	1.2	1	1	1.3	1	1	-0.074	-0.049	-0.071	46.4	44.7	45	16.1	14.8	15
7	Pier 34S	Column D	73	6/26/2000	1.3	1	1	1.3	1.1	0.9	-0.08	-0.048	-0.064	49.6	44.6	41.8	23.4	22.4	22.2
8	Pier 34S	Column D	79	8/24/2000	1.2	1	1	1.3	1.1	1	-0.077	-0.046	-0.059	67.2	72.1	67.5	24.9	23.4	23.4
9	Pier 34S	Column D	57	11/2/2000	1	0.9	0.9	1.2	1	0.8	-0.08	-0.043	-0.057	50.9	47.1	50.8	13.4	11.8	11.8
10	Pier 34S	Column D	43	3/20/2001	1	0.9	0.9	1.2	0.9	0.9	-0.079	-0.041	-0.043	59.3	63.1	57.6	4.4	3.6	3.9
11	Pier 34S	Column D	59	5/31/2001	1.2	1	1	1.3	1	0.9	-0.077	-0.044	-0.051	52.8	51.6	48.2	17.4	16	15.7
12	Pier 34S	Column D	73	8/3/2001	1.2	1.1	1	1.4	1.1	1	-0.083	-0.045	-0.054	61.7	66.3	61.5	26.5	25.1	24.8
13	Pier 34S	Column D	39	10/17/2001	1.2	0.9	0.9	1.2	0.9	0.9	-0.071	-0.034	-0.038	56.1	60.3	59.2	5.1	3.2	3.2
14	Pier 34S	Column D	30	12/20/2001	1	0.9	0.9	1.1	0.9	0.9	-0.075	-0.042	-0.05						
15	Pier 34S	Column D	26	2/21/2002	1.5	0.9	0.9	1.1	0.9	0.9	-0.076	-0.041	-0.045						
16	Pier 34S	Column D	46	4/29/2002	1.2	1	0.9	1.2	0.9	0.9	-0.074	-0.041	-0.053	65.5	70.6	67.8	6.6	5.1	5
17	Pier 34S	Column D	72	6/20/2002	1.2	1	1	1.3	1	0.9	-0.076	-0.04	-0.052	61.8	59.9	54.5	22.7	21.4	21.7
18	Pier 34S	Column D	69	9/3/2002	1.3	1	1	1.3	1.1	1	-0.076	-0.04	-0.049	50.4	50.1	50.3	21.5	19.8	20.2
19	Pier 34S	Column D	22	11/5/2002	1.1	1	0.9	1.1	1.1	0.9	-0.065	-0.031	-0.038						
20	Pier 34S	Column D	25	1/28/2003	1.1	0.9	0.9	1.1	0.9	0.9	-0.061	-0.028	-0.026						
21	Pier 34S	Column D	18	3/7/2003	1.2	0.9	0.9	1.1	0.9	0.9	-0.066	-0.035	-0.047						
22	Pier 34S	Column D	44	5/20/2003	1.2	0.9	0.9	1.3	1	1	-0.065	-0.042	-0.056	55.3	55.9	51.7	10.3	8.6	8.7
23	Pier 34S	Column D	66	7/11/2003	1.2	1.3	1	1	1	1	0	0	0	62.8	68.1	63.4	20.3	18.8	18.8
24	Pier 34S	Column D	47	9/25/2003	1.2	1	0.9	1.2	1	0.9	-0.063	-0.032	-0.001	39.5	57.1	56.7	10.3	8.4	9.1
25	Pier 34S	Column D	24	11/13/2003	1.1	0.9	0.9	1.1	0.9	0.9	-0.061	-0.028	-0.001						
26	Pier 34S	Column D	24	11/13/2003	1.1	0.9	0.9	1.1	0.9	0.9	-0.061	-0.028	-0.001						
27	Pier 34S	Column D	30	2/19/2004	1.3	0.9	0.9	1.1	0.9	0.9	-0.065	-0.027	0.005						
28	Pier 34S	Column D	43	3/29/2004	1.2	1	0.9	1.2	1	0.9	-0.069	-0.041	-0.016	87.8	84.2	82	6.8	5.6	5.6
29	Pier 34S	Column D	59	6/2/2004	1.7	1	0.9	1.2	1	0.9	-0.064	-0.038	-0.005	63.4	72.1	64	13.7	12.3	12.3
30	Pier 34S	Column D	71	8/26/2004	1.4	1.3	1	1.3	1.1	1	-0.059	-0.032	-0.003	66.6	71.6	64.2	23.1	21.7	21.6
31	Pier 34S	Column D	50	10/20/2004	1.2	0.9	0.9	1.1	0.9	0.9	-0.055	-0.026	0.002	55.8	66.7	63.4	8.7	6.9	6.9
32	Pier 34S	Column D	0	12/15/2004	1.1	1	0.9	1.2	0.9	0.9	-0.058	-0.041	-0.01						
33	Pier 34S	Column D	11	2/23/2005	1	0.9	0.9	1.3	0.9	0.9	-0.064	-0.043	-0.022						
34	Pier 34S	Column D	48	4/22/2005	1.4	1	1	1.3	1	0.9	-0.055	-0.045	-0.008		66.1	67		10.1	10.3
35	Pier 34S	Column D	72	7/22/2005	1.4	1.1	1	1.3	1.1	0.9	-0.058	-0.042	-0.006		70.1	56.2		26	26.1
36	Pier 34S	Column D	64	9/16/2005	1.2	1	1	1.3	1	0.9	-0.054	-0.037	-0.011		62.5	58		17.4	17.1
37	Pier 34S	Column D	15	12/21/2005	1.1	0.9	0.9	1.1	0.9	0.9	-0.054	-0.041	-0.003						
38	Pier 34S	Column D	22	2/28/2006	1.3	0.9	0.9	1.2	0.9	0.9	-0.057	-0.045	-0.026						
39	Pier 34S	Column D	56	5/22/2006	1.2	1	0.9	1.2	1.1	0.9	-0.05	-0.044	-0.003		41.2	59.8		11.3	11.5
40	Pier 34S	Column D	86	7/18/2006	1.5	1.1	1.1	1.4	1.2	1	-0.049	-0.045	-0.004		59.2	56.1		26.3	26.4
41	Pier 34S	Column D	55	9/12/2006	1.6	1	0.9	1.3	1	0.9	-0.047	-0.037	-0.006						
42	Pier 34S	Column D	57	11/8/2006	1.2	1	1	1.2	1	0.9	-0.04	-0.035	-0.002		58.2	54.2		11.7	12
43	Pier 34S	Column D	34	1/8/2007	1.2	0.9	0.9	1	0.9	0.9	-0.064	-0.04	-0.005						
44	Pier 34S	Column D	36	3/12/2007	1.1	1	0.9	1.2	1	1	-0.055	-0.037	-0.004	NR	NR	NR			
45	Pier 34S	Column D	74	6/1/2007	1.4	1.1	1	1.4	1.1	1	-0.036	-0.04	-0.008		80.3	78.6		20.6	20.8
46	Pier 34S	Column D	54	10/19/2007	1.3	1	0.9	1.4	1	0.9	-0.044	-0.03	-0.001		46.7	62.7		10.7	10.4
47	Pier 34S	Column D	30	12/3/2007	0.9	0.9	0.6	1.2	1	0.9	-0.05	-0.038	-0.001						
48	Pier 34S	Column D	48	4/29/2009	1	0.9	0.9	1.2	1	0.9	-0.057	-0.038	-0.005		69.2	79.7		9.8	9.5

	CL	CM	CN	CO	CP	CQ	CR	CS	CT	CU	CV	CW	CX
2		Resistivity Probes (Q)											
3		Probe 1			Probe 2			Relative Humidity (%)			Ambient Temp (deg C)		
4		1	2	3	1	2	3	1	2	3	1	2	3
5	Column E	0.2	0.4	0.6	0.5	-0.042	-0.031						
6	Column E	0.2	0.4	0.5	0.4	-0.052	-0.057	37.6	38.5	48.3	17.7	15.4	15.2
7	Column E	0.2	0.4	0.5	0.5	-0.053	-0.056	53.4	48.4	48	23.7	22	22
8	Column E	0.2	0.5	0.5	0.5	-0.057	-0.072	63.5	71.6	68.7	26.5	24	24
9	Column E	0.2	0.4	0.4	0.5	-0.047	-0.048	55.4	51.8	54.4	14.3	11.9	11.9
10	Column E	0.2	0.4	0.6	0.4	-0.045	-0.037	57.2	64.2	58.7	6.2	3.8	4.2
11	Column E	0.3	0.4	0.5	0.5	-0.044	-0.048	41.1	45	53.4	17.5	15.9	15.9
12	Column E	0.2	0.5	0.5	0.5	-0.046	-0.049	57.8	65.6	63.5	27.6	25.3	25.2
13	Column E	0.1	0.4	0.6	0.5	-0.047	-0.051	48.7	58	59	10.5	4.6	4.4
14	Column E	0.2	0.4	0.6	0.5	-0.041	-0.034						
15	Column E	0.2	0.4	0.6	0.4	-0.045	-0.037						
16	Column E	0.2	0.4	0.6	0.5	-0.046	-0.042	65	67.4	64.3	8.1	5.7	5.6
17	Column E	0.2	0.4	0.6	0.6	-0.044	-0.049	59	59.5	59.4	23.6	21.4	21.7
18	Column E	0.1	0.4	0.6	275000	-0.044	-0.046	47.2	50.9	49.9	27.4	21.4	21.1
19	Column E	0.3	0.4	0.8	3500000	-0.048	-0.045	55.3	61.7	60.2	-1.8	-3.4	-3.6
20	Column E	0.4	0.4	0.6	2200000	-0.049	-0.047						
21	Column E	0.4	0.4	0.7	1500000	-0.054	-0.055						
22	Column E	0.2	0.5	0.7	6500000	-0.05	-0.054	52.7	54.2	53.3	11.2	8.9	9.1
23	Column E	0.4	0.4	0.7	4500000	0	0	62.3	67	63.1	21.1	19	14.1
24	Column E	0.3	0.4	0.7	10000000	-0.038	-0.038	38.3	48.5	52.5	18.9	10	10.1
25	Column E	0.3	0.4	0.6	21000000	-0.042	-0.041						
26	Column E	0.3	0.4	0.6	21000000	-0.042	-0.041						
27	Column E	0.5	0.4	0.6	19500000	-0.038	-0.035						
28	Column E	0.6	0.4	0.7	4000000	-0.039	-0.039	70.6	61.1	60.6	8	5.8	6.1
29	Column E	0.6	0.4	0.6	8000000	-0.04	-0.044	64.1	68.3	60.2	14.4	12.4	12.6
30	Column E	0.8	0.4	0.7	7000000	-0.036	-0.039	67.5	74	66.8	23.8	21.9	21.7
31	Column E	0.3	0.4	0.6	18000000	-0.033	-0.038	70.2	16	66.4	8.8	7.1	7
32	Column E	0.3	0.5	0.7	26000000	-0.036	-0.044						
33	Column E	0.4	0.4	0.7	30000000	-0.036	-0.043						
34	Column E	0.2	0.4	0.5	12000000	-0.037	-0.048		43.7	62.3		10.6	10.6
35	Column E	0.6	0.4	0.6	7000000	-0.034	-0.044		60	53.4		26.1	25.9
36	Column E	0.7	0.4	0.6	12000000	-0.03	-0.042		64.7	54.6		17.3	17.1
37	Column E	0.6	0.3	0.5	22000000	-0.027	-0.034						
38	Column E	0.6	0.3	0.5	30000000	-0.031	-0.043						
39	Column E	0.6	0.4	0.6	18000000	-0.029	-0.052		54.3	46.6		11.9	12
40	Column E	0.8	0.5	0.8	8500000	-0.028	-0.045		44.2	44.9		26.6	26.5
41	Column E	0.5	0.4	0.6	20000000	-0.02	-0.034						
42	Column E	0.3	0.4	0.6	23000000	-0.016	-0.03		57.5	54.2		12.2	12.1
43	Column E	0.2	0.4	0.6	35000000	-0.016	-0.034						
44	Column E	0.5	0.4	0.6	30000000	-0.018	-0.033	NR	NR	NR			
45	Column E	0.3	0.4	0.6	4900000	-0.023	-0.046		60.6	77.7		20.8	20.9
46	Column E	0.4	0.4	0.6	23000000	-0.016	-0.033		49.3	65.6		10.4	10.4
47	Column E	0.5	0.5	0.7	22000000	-0.016	-0.026						
48	Column E	0.7	0.4	0.6	1.6E+08	-0.016	-0.044	48.7	53		9.2	9.9	

	CY	CZ	DA	DB	DC	DD	DE	DF	DG	DH	DI	DJ	DK	DL	DM	DN
2		Resistivity Probes (Q)						Half-Cells (V)								
3		Probe 1			Probe 2			Probe 1			Relative Humidity (%)			Ambient Temp (deg C)		
4		1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
5	Column F	1	0.9	0.9	1.2	1.1	0.9	-0.093	-0.041	-0.067						
6	Column F	1.3	1.1	0.9	1.3	1.2	1	-0.055	-0.079	-0.103	NR	30.4	60.1	20.8	20.9	20.8
7	Column F	1.2	1.1	0.9	1.3	1.2	1	-0.061	-0.083	-0.099	42	41.2	46.8	26.7	26.2	26.7
8	Column F	1.3	1.1	0.9	1.4	1.3	1	-0.072	-0.084	-0.099	57.2	61.5	57.7	28	27.8	28
9	Column F	1.2	1.2	0.9	1.3	1.3	0.9	-0.13	-0.072	-0.089	72.8	85.1	86.5	13.2	13.2	13.2
10	Column F	1.1	1	0.9	1.3	1	1	-0.098	-0.085	-0.071	52.2	68.1	67.4	6.4	6.2	6.4
11	Column F	1.3	1.1	0.9	1.4	1.2	0.9	-0.032	-0.071	-0.088	57	58.1	47	17.3	17.5	17.3
12	Column F	1.3	1.1	0.9	1.4	1.3	1	-0.138	-0.077	-0.08	51.9	50.7	45.7	31.5	32.4	31.5
13	Column F	1.1	1	0.9	1.3	1.1	0.9	-0.088	-0.061	-0.065	68.6	58	43	9.8	7.2	9.8
14	Column F	1.1	1	0.9	1.1	1.1	0.9	-0.083	-0.05	-0.064						
15	Column F	1.1	1	0.9	1.2	1	0.9	-0.086	-0.062	-0.071						
16	Column F	1.2	1	0.9	1.1	1.1	0.9	-0.125	-0.06	-0.065	94.4	74.6	90.4	15.6	12	15.6
17	Column F	1.3	1.1	0.9	1.3	1.2	0.9	-0.118	-0.101	-0.088	60	60.7	49.1	27.9	27.4	27.9
18	Column F	1.3	1.1	0.9	1.4	1.2	1	-0.112	-0.09	-0.088	44.7	56.8	50.9	29.5	26.4	29.5
19	Column F	1.1	0.9	0.9	1.2	1.1	0.9	-0.077	-0.061	-0.075	57.6	63.2	60.9	-3.9	-3.7	-3.9
20	Column F	0.9	0.9	0.9	1.1	1	0.9	-0.058	-0.047	-0.052						
21	Column F	0.9	0.9	0.8	1.1	1	0.9	-0.072	-0.055	-0.057						
22	Column F	1	1.1	0.9	1.2	1.2	0.9	-0.102	-0.089	-0.087	71.1	51.7	51.5	12.8	11.5	12.8
23	Column F	1.2	1.1	0.9	1.4	1.2	1	0	0	0	64.5	59.8	56.9	26.1	26.2	26.1
24	Column F	1.1	1.1	0.9	1.3	1.1	0.9	-0.068	-0.127	-0.081	33	58.9	44.5	16.7	14.8	16.7
25	Column F	1.1	1	0.9	1.1	1.1	0.9	-0.059	-0.09	-0.065	71.7	60.5	76	1.3	-0.8	1.3
26	Column F	1.1	1	0.9	1.1	1.1	0.9	-0.059	-0.09	-0.065	71.7	60.5	76	1.3	-0.8	1.3
27	Column F	1	0.9	0.9	1.2	1	0.9	-0.035	-0.069	-0.038						
28	Column F	1	1	0.9	1.3	1.2	0.9	-0.052	-0.114	-0.071	89.3	83.8	68.7	9.6	8.2	9.6
29	Column F	1.1	1	0.9	1.4	1.2	4400000	-0.068	-0.135	-0.097	72.5	81.7	70.6	15	14.8	15
30	Column F	1.2	1.2	0.9	1.4	1.3	1600000	-0.07	-0.122	-0.1	67.9	71.1	66.3	22.2	22.2	22.2
31	Column F	NR	NR	0.9	1.3	1	2600000	-0.052	-0.008	-0.073		69.1	68.5	8.6	69	6.9
32	Column F	1.1	NR	0.9	1.3	1	3750000	-0.04	-0.063	-0.044						
33	Column F	1.1	1	0.9	1.1	1	3500000	-0.036	-0.078	-0.04						
34	Column F	3.6	1.3	0.9	1.3	1.2	2000000	-0.058	-0.12	-0.093		57.6	78		15.6	13.8
35	Column F	6000000	1300	82	1.3	1.1	750000	-0.066	-0.149	-0.139		62.9	62.1		25.7	25.8
36	Column F	6000000	400000	81	1.2	1	1400000	-0.057	-0.117	-0.134		67.6	55.8		17	16.8
37	Column F	2300000	157000	75.2	80000	0.9	4000000	-0.036	-0.011	-0.037						
38	Column F	3.5	3000000	120000	NR	1	2000000	-0.041	-0.07	-0.085						
39	Column F	2500000	25000000	35000	NR	1.3	560000	-0.052	-0.115	0.195		70.5	41.2		21.7	23.3
40	Column F	NR	NR	55000	NR	200000	0.8	-0.06	-0.127	0.006		65.6	NR		36	37.2
41	Column F	16000000	3000000	203000	NR	1	1000000	-0.048	-0.061	-0.105						
42	Column F	NR	6000000	350000	NR	1	600000	-0.032	-0.085	-0.073		65.8	52.9		12.3	12.9
43	Column F	2.8	50000	900000	860000	0.9	1200000	-0.028	-0.06	-0.092						
44	Column F	3.3	700000	330000	1500000	1	520000	-0.031	-0.091	-0.121		NR	NR			
45	Column F	11000	1.2	275000	NR	1.2	117000	-0.046	-0.108	-0.161		77.1	57.5		22.9	22.9
46	Column F	8000000	75000	320000	14000000	1	250000	-0.044	-0.08	-0.043		69.7	70.1		10.5	10.5
47	Column F	7500000	275000	400000	32000000	0.9	1300000	-0.043	-0.067	-0.042						
48	Column F	NR	NR	NR	NR	1	220000	-0.04	-0.109	-0.171		66.2	52.2		10.2	10.1

	DO	DP	DQ	DR	DS	DT	DU	DV	DW	DX	DY	DZ	EA	EB
2		Resistivity Probes (Q)				Half-Cells (V)			Resistivity Probes (Q)				Half-Cells (V)	
3		Probe 1		Probe 2		Probe 1			Probe 1		Probe 2		Probe 1	
4		1	2	1	2	1	2		1	2	1	2	1	2
5	West face	0.8	2200000	0.7	0.3	-0.005	-0.096	East Face	0.6	1.2	0.5	0.9	-0.01	-0.107
6	West face	0.8	703000	0.8	0.4	-0.043	-0.127	East Face	0.6	1.4	0.8	1	-0.052	-0.135
7	West face	0.8	1070000	0.8	0.6	-0.045	-0.13	East Face	0.6	1.5	0.6	0.9	-0.054	-0.135
8	West face	0.8	820000	0.9	0.6	-0.047	-0.127	East Face	0.5	1.5	0.8	1	-0.056	-0.133
9	West face	0.8	1100000	0.8	0.4	-0.034	-0.123	East Face	0.6	1.5	0.6	1	-0.042	-0.128
10	West face	0.8	1500000	0.8	0.4	-0.014	-0.092	East Face	0.6	1.3	0.6	0.9	-0.024	-0.122
11	West face	0.9	950000	0.9	0.5	-0.029	-0.123	East Face	0.6	1.6	0.6	1.1	-0.042	-0.132
12	West face	0.8	1400000	0.9	0.5	-0.006	-0.105	East Face	0.6	1.5	0.7	0.9	-0.029	-0.119
13	West face	0.8	1600000	0.8	0.4	-0.035	-0.093	East Face	0.5	1.4	0.6	0.9	-0.043	-0.102
14	West face	0.7	1700000	0.7	0.5	-0.009	-0.091	East Face	0.6	1.3	0.6	0.9	-0.011	-0.104
15	West face	0.7	2000000	0.8	0.5	-0.013	-0.078	East Face	0.5	1.3	0.6	0.9	-0.016	-0.093
16	West face	0.8	1800000	0.8	0.6	-0.023	-0.081	East Face	0.5	1.3	0.6	0.9	-0.029	-0.095
17	West face	0.8	1350000	0.8	0.7	-0.016	-0.102	East Face	0.6	1.5	0.6	1	-0.035	-0.113
18	West face	0.8	1300000	0.9	0.6	0.077	-0.096	East Face	0.5	1.5	0.7	1	-0.021	-0.107
19	West face	0.8	1600000	0.8	0.6	-0.033	-0.091	East Face	0.6	1.3	0.7	0.9	-0.029	-0.101
20	West face	0.8	2200000	0.7	0.5	-0.033	-0.063	East Face	0.5	1.2	0.6	0.9	-0.03	-0.084
21	West face	0.7	2500000	0.7	0.4	-0.039	-0.073	East Face	0.4	1.3	0.7	0.9	-0.035	-0.093
22	West face	0.8	1200000	0.8	0.3	-0.034	-0.111	East Face	0.6	1.4	0.6	0.9	-0.054	-0.111
23	West face	0.8	1200000	0.8	0.4	0	0	East Face	0.6	1.5	0.6	0.9	0	0
24	West face	0.8	1500000	0.8	0.4	0.013	-0.089	East Face	0.6	1.4	0.6	0.9	-0.01	-0.096
25	West face	0.7	1600000	0.8	0.3	-0.024	-0.079	East Face	0.6	1.2	0.6	0.9	-0.029	-0.086
26	West face	0.7	1600000	0.8	0.3	-0.024	-0.079	East Face	0.6	1.2	0.6	0.9	-0.029	-0.086
27	West face	0.7	2500000	0.7	0.5	-0.023	-0.069	East Face	0.6	1.4	0.6	0.9	-0.015	-0.083
28	West face	0.7	2200000	0.8	3.4	-0.008	-0.072	East Face	0.6	1.4	0.6	1	-0.016	-0.086
29	West face	0.7	1800000	0.8	0.6	-0.007	-0.087	East Face	0.6	2700000	0.6	3600000	-0.025	-0.096
30	West face	0.8	1300000	0.9	0.8	-0.014	-0.093	East Face	0.7	1700000	0.8	1700000	-0.018	-0.101
31	West face	0.7	1500000	0.8	0.8	0.003	-0.082	East Face	0.6	1600000	0.6	2000000	-0.01	-0.083
32	West face	0.6	2500000	0.7	2.2	-0.028	-0.05	East Face	0.5	1900000	0.6	1600000	-0.021	-0.054
33	West face	0.6	3300000	0.7	0.6	-0.036	-0.058	East Face	0.5	1600000	0.6	1500000	-0.027	-0.057
34	West face	0.7	2000000	0.9	0.2	-0.012	-0.094	East Face	0.6	1300000	0.6	1200000	-0.031	-0.1
35	West face	0.7	1030000	0.8	0.3	0.011	-0.104	East Face	0.5	980000	0.7	1700000	-0.02	-0.114
36	West face	0.7	1300000	0.8	0.3	0.007	-0.095	East Face	0.6	1100000	0.6	1700000	-0.012	-0.103
37	West face	0.6	3000000	0.7	0.5	-0.026	-0.033	East Face	0.5	2000000	0.6	2900000	-0.012	-0.036
38	West face	0.6	2000000	0.8	0.3	-0.056	-0.08	East Face	0.7	1500000	0.3	2600000	-0.042	-0.078
39	West face	0.7	1100000	0.8	0.4	-0.034	-0.119	East Face	0.6	1000000	0.7	1500000	-0.039	-0.112
40	West face	0.8	955000	2.9	0.8	-0.003	-0.129	East Face	0.6	1500000	1600000	1600000	-0.028	-0.119
41	West face	0.8	1200000	0.8	3.2	-0.01	-0.106	East Face	0.6	1300000	0.7	1600000	-0.012	-0.103
42	West face	0.7	1500000	0.9	2.5	0.002	-0.099	East Face	0.6	1300000	0.7	1600000	0.001	-0.098
43	West face	0.6	1500000	0.8	0.9	-0.057	-0.124	East Face	0.6	1200000	0.7	2600000	-0.034	-0.105
44	West face	0.7	1100000	0.8	0.5	-0.045	-0.129	East Face	0.6	2000000	0.7	2200000	-0.032	-0.102
45	West face	0.8	650000	0.8	0.9	-0.034	-0.146	East Face	0.6	370000	0.7	850000	-0.035	-0.122
46	West face	0.8	1000000	0.9	0.6	-0.033	-0.127	East Face	0.6	730000	0.6	2500000	-0.023	-0.11
47	West face	0.6	1500000	0.8	0.9	-0.026	-0.094	East Face	0.6	2100000	0.6	2500000	-0.01	-0.088
48	West face	0.6	0.7	0.8	52.9	-0.061	-0.167	East Face	0.6	3000000	0.6	8000000	-0.037	-0.137

	EC	ED	EE	EF	EG	EH	EI	EJ	EK	EL	EM	EN	EO	EP	EQ	ER	ES	ET	EU	EV	EW	EX	EY	EZ
1																								
2	Pier 37N				Resistivity Probes (Q)				Half-Cells (V)															
3					Probe 1		Probe 2		Probe 1		Relative Humidity (%)							Ambient Temp (deg C)						
4			Temp. {°F)	Date	1	2	1	2	1	2	1	2	3	4	5	6	7	1	2	3	4	5	6	7
5	Pier 37N	Column A	30	1/4/2000	0.1	0.9	2300000	0.9	-0.101	-0.052														
6	Pier 37N	Column A	67	4/24/2000	0.2	0.9	1700000	0.9	-0.095	-0.077	87.2			54.5			67.7	13.2			13.4			14.4
7	Pier 37N	Column A	73	6/26/2000	0.2	1	2500000	1	-0.103	-0.082	60.5			56.4			92.5	22.2			20.7			20.3
8	Pier 37N	Column A	79	8/24/2000	0.2	1	2000000	1	-0.103	-0.079	83.3			83			93.5	23.8			22.5			22.3
9	Pier 37N	Column A	57	11/2/2000	0.1	1	3300000	0.9	-0.1	-0.07	63			59.6			71.5	12.8			10.8			11.1
10	Pier 37N	Column A	43	3/20/2001	0.2	1	3650000	0.9	-0.092	-0.06	80.3			71.3			78.3	0			2.6			2
11	Pier 37N	Column A	59	5/31/2001	0.4	0.9	2000000	1	-0.1	-0.075	66.1			50.3			76.3	17.5			16.3			16.3
12	Pier 37N	Column A	73	8/3/2001	0.6	1.1	2000000	1.1	-0.109	-0.079	68.3			67.9			84.8	27.2			25.7			25.6
13	Pier 37N	Column A	39	10/17/2001	0.7	0.9	8750000	0.9	-0.098	-0.055	68			62.8			74.9	5			3.7			3.8
14	Pier 37N	Column A	30	12/20/2001	0.4	0.9	9000000	0.9	-0.093	-0.052														
15	Pier 37N	Column A	26	2/21/2002	0.2	0.9	1700000	0.9	-0.091	-0.054														
16	Pier 37N	Column A	46	4/29/2002	0.9	0.9	13000000	0.9	-0.09	-0.055	71.4			64.3			66.7	7.6			6.8			6.4
17	Pier 37N	Column A	72	6/20/2002	1.2	1	5000000	1	-0.094	-0.065	75.5			63.1			70	23.2			21.8			21.7
18	Pier 37N	Column A	69	9/3/2002	1	1	5500000	1.1	-0.099	-0.065	65.5			61.7			73.2	21.7			20.4			20.4
19	Pier 37N	Column A	22	11/5/2002	0.5	1	20000000	0.9	-0.093	-0.051														
20	Pier 37N	Column A	25	1/28/2003	0.2	0.9	8000000	0.9	-0.082	-0.049														
21	Pier 37N	Column A	18	3/7/2003	0.4	0.9	5200000	0.9	-0.082	-0.057														
22	Pier 37N	Column A	44	5/20/2003	0.5	0.9	4200000	1.1	-0.09	-0.064	67.9			55.3			69.9	11			9.3			9.6
23	Pier 37N	Column A	66	7/11/2003	0.4	0.9	6500000	1	-0.097	-0.062	69.3			71.3			76.7	20.4			19			18.8
24	Pier 37N	Column A	47	9/25/2003	0.5	0.9	18000000	0.9	-0.101	-0.058	67.4			68.2			69.7	10.4			8.7			8.4
25	Pier 37N	Column A	24	11/13/2003	0.4	1.1	27000000	0.9	-0.091	-0.053														
26	Pier 37N	Column A	24	11/13/2003	0.4	1.1	27000000	0.9	-0.091	-0.053														
27	Pier 37N	Column A	30	2/19/2004	0.3	0.9	3500000	0.9	-0.083	-0.063														
28	Pier 37N	Column A	43	3/29/2004	0.6	4.4	8900000	0.9	-0.093	-0.072	82.8			74.1			65.7	7.1			5.3			5.3
29	Pier 37N	Column A	59	6/2/2004	0.8	0.9	15000000	1	-0.096	-0.071	52.9			81.2			74.5	16.4			13.4			13.3
30	Pier 37N	Column A	71	8/26/2004	1	1.2	10000000	1.2	-0.092	-0.067	79.4			80.8			76.8	23.5			22			21.9
31	Pier 37N	Column A	50	10/20/2004	0.5	1	18000000	0.9	-0.085	-0.064	63.5	66.2	67.2					6.9	7.2	7.1				
32	Pier 37N	Column A	0	12/15/2004	0.2	1	10000000	1.4	-0.08	-0.065														
33	Pier 37N	Column A	11	2/23/2005	0.5	0.9	2400000	4700000	-0.111	-0.07														
34	Pier 37N	Column A	48	4/22/2005	1.3	0.9	700000	3200000	-0.125	-0.079				68.6			59.1				10.3			10.3
35	Pier 37N	Column A	72	7/22/2005	0.6	1.2	2000000	5500000	-0.115	-0.077				75.7			69.7				25.9			25.7
36	Pier 37N	Column A	64	9/16/2005	0.4	1	3250000	6500000	-0.105	-0.073					67.1	68.3						16.7	16.7	
37	Pier 37N	Column A	15	12/21/2005	1.3	0.9	1900000	3300000	-0.077	-0.066														
38	Pier 37N	Column A	22	2/28/2006	2.6	0.9	2500000	4700000	-0.08	-0.069														
39	Pier 37N	Column A	56	5/22/2006	0.4	0.9	1900000	16	-0.108	-0.074				44.7			59				12.2			12
40	Pier 37N	Column A	86	7/18/2006	0.4	1	2000000	4600000	-0.113	-0.076				65.9			58.7				12.7	26.7		26.5
41	Pier 37N	Column A	55	9/12/2006	0.6	1.1	1200000	4000000	-0.095	-0.072				67.8			66.3				12.7			12.7
42	Pier 37N	Column A	57	11/8/2006	5.1	0.9	2000000	5700000	-0.06	-0.069														
43	Pier 37N	Column A	34	1/8/2007	0	0	0	0	0	0														
44	Pier 37N	Column A	36	3/12/2007	0	0	0	0	0	0														
45	Pier 37N	Column A	80	6/11/2007	2.4	1	2000000	4500000	-0.075	-0.071				62.2			60.4				25.8			25.4
46	Pier 37N	Column A	54	10/19/2007	39.3	0.9	1500000	4100000	-0.076	-0.072				47.4			67.5				10.5			10.4
47	Pier 37N	Column A	30	12/3/2007	5.1	0.9	1.7	4700000	-0.063	-0.071														
48	Pier 37N	Column A	48	4/29/2009	1800000	1	1200000	22	-0.053	-0.071				59.4			66.7				9.4			9.3

	FA	FB	FC	FD	FE	FF	FG	FH	FI	FJ	FK	FL	FM	FN	FO	FP	FQ	FR	FS	FT	FU
1											Pier 37N										
2		Resistivity Probes (Q)				Half-Cells (V)															
3		Probe 1		Probe 2		Probe 1		Relative Humidity (%)							Ambient Temp (deg C)						
4		1	2	1	2	1	2	1	2	3	4	5	6	7	1	2	3	4	5	6	7
5	Column B	0.3	0.5	0.8	0.5	-0.076	-0.048														
6	Column B	0.4	0.7	0.7	0.5	-0.08	-0.068	71.4			56.8			57.5	13.5			13.5			14.9
7	Column B	0.6	0.6	0.8	0.5	-0.088	-0.073	62.3			63.2			61.3	22.2			20.8			20.7
8	Column B	0.8	0.5	0.7	0.5	-0.089	-0.077	74.1			82.9			82.5	23.9			22.5			22.3
9	Column B	0.4	0.5	0.7	0.3	-0.085	-0.066	59.4			57.4			55.7	12.8			11.3			11
10	Column B	0.4	0.5	0.7	0.4	-0.077	-0.054	71.5			68.2			67.8	0.2			2.2			2.4
11	Column B	0.8	0.5	0.8	0.4	-0.087	-0.068	54.5			62.6			63.9	17.6			16.3			16.2
12	Column B	0.8	0.6	0.8	0.5	-0.092	-0.075	62.3			73.8			69.5	27.3			25.9			25.8
13	Column B	0.6	0.5	0.8	0.5	-0.082	-0.053	60.3			64.3			66	5.1			3.8			3.8
14	Column B	0.3	0.6	0.7	0.5	-0.082	-0.053														
15	Column B	0.4	0.5	0.5	0.8	-0.086	-0.054														
16	Column B	0.5	0.6	0.8	0.5	-0.074	-0.05	62.7			62.8			64.9	8.1			6.6			6.6
17	Column B	0.8	0.6	0.8	0.5	-0.084	-0.067	57.9			55.3			61.9	23.3			21.9			21.9
18	Column B	0.7	0.6	0.8	0.5	-0.086	-0.157	56.1			64.4			61.2	21.8			20.3			20.4
19	Column B	0.5	0.5	0.7	0.4	-0.079	-0.06														
20	Column B	0.2	0.5	0.6	0.4	-0.074	-0.056														
21	Column B	0.5	0.5	0.7	0.4	-0.076	-0.059														
22	Column B	0.8	0.6	0.7	0.5	-0.085	-0.062	53.8			61.5			57.9	10.5			9.1			9
23	Column B	0.8	0.6	0.8	0.5	-0.081	-0.061	65.4			71.6			67.2	20.6			19.1			19.1
24	Column B	0.5	0.6	0.7	0.5	-0.083	-0.054	51.3			57.3			51.8	9.9			8.8			8.9
25	Column B	0.5	0.5	0.7	0.4	-0.079	-0.046											-2.1			
26	Column B	0.5	0.5	0.7	0.4	-0.079	-0.046											-2.1			
27	Column B	0.4	0.6	0.7	0.5	-0.062	-0.042														
28	Column B	0.6	0.5	0.7	0.4	-0.077	-0.051	58.1			76			61.2	6.8			5.6			5.4
29	Column B	0.7	0.6	0.8	0.8	-0.076	-0.068	61.6			65.6			60.7	14.5			13.2			13.3
30	Column B	0.8	0.6	0.8	0.5	-0.076	-0.049	68.6			69.5			65.5	23.5			22			22
31	Column B	1.8	0.6	0.7	0.5	-0.07	-0.038	52.4	62.9	65.5					10.5	7.3	7.1				
32	Column B	0.5	0.5	0.7	0.4	-0.073	-0.041														
33	Column B	0.5	0.5	0.7	0.4	-0.075	-0.047														
34	Column B	0.6	0.6	0.7	0.6	-0.074	-0.045				45.8			40.1				10.2			10.2
35	Column B	0.8	0.6	0.7	0.4	-0.075	-0.054		66.1	52.8						25.6	25.6				
36	Column B	0.7	0.6	0.8	0.5	-0.069	-0.046				59.9			52.7				17.3			17.3
37	Column B	0.6	0.5	0.7	0.5	-0.066	-0.043														
38	Column B	0.3	0.6	0.6	0.5	-0.063	-0.042														
39	Column B	0.8	0.6	0.8	0.4	-0.064	-0.041				54.4			43.8				11.6			12
40	Column B	0.7	0.6	0.8	0.5	-0.068	-0.046				56			41.3				26.4			26.5
41	Column B	0.7	0.6	0.8	0.5	-0.063	-0.038				64.1			61.3				12.7			12.9
42	Column B	0.6	0.6	0.8	0.5	-0.061	-0.035				54.8			54.1				12.8			12.6
43	Column B	0	0	0	0	0	0														
44	Column B	0	0	0	0	0	0														
45	Column B	0.8	0.6	0.8	0.5	-0.061	-0.043				58.6			51.7				25.7			25.8
46	Column B	0.9	0.5	0	0.5	-0.012	-0.035				47.1			64.8				10			10
47	Column B	0.6	0.5	0.7	0.5	-0.062	-0.039														
48	Column B	1.4	0.5	0	0.4	0	0.033				61.6			45				9.6			9.4

	FV	FW	FX	FY	FZ	GA	GB	GC	GD	GE	GF	GG	GH	GI	GJ	GK	GL	GM	GN	GO	GP	GQ	GR	GS	GT	GU	GV
1																											
2		Resistivity Probes (Q)				Half-Cells (V)									Resistivity Probes (Q)				Half-Cells (V)			Resistivity Probes (Q)				Half-Cells (V)	
3		Probe 1		Probe 2		Probe 1		Relative Humidity (%)			Ambient Temp (deg C)				Probe 1		Probe 2		Probe 1			Probe 1		Probe 2		Probe 1	
4		1	2	1	2	1	2	1	2	3	1	2	3		1	2	1	2	1	2		1	2	1	2	1	2
5	Column C	1	1.1	1	1	-0.134	-0.084							West Face	0.6	95000	0.5	0.6	0.272	0.195	East Face	0.7	0.2	0.9	1.1	-0.117	-0.029
6	Column C	1.2	1.3	1.1	1.2	-0.223	-0.079	60.5	48	55.7	15.1	13.7	13.8	West Face	0.6	0.7	0.6	0.8	0.216	0.174	East Face	0.9	0.3	0.9	1.1	-0.124	-0.09
7	Column C	1.1	1.3	1.2	1.3	-0.221	-0.082	63.7	52.3	58.7	22.3	20.9	20.8	West Face	0.7	0.8	0.6	0.8	0.156	0.161	East Face	0.9	1.1	0.9	1.3	-0.122	-0.092
8	Column C	1.3	1.3	1.2	1.3	-0.164	-0.073	75.7	76.2	75.2	24.1	22.6	22.5	West Face	0.7	52000	0.6	0.8	0.312	0.169	East Face	0.8	1.1	0.9	1.3	-0.118	-0.064
9	Column C	1.1	1.1	1	1.2	-0.154	-0.071	43.6	50.1	56	13	11.4	11	West Face	0.6	69100	0.6	0.7	0.274	0.072	East Face	0.9	0.5	0.8	1.1	-0.114	-0.042
10	Column C	1	1.2	1	1	-0.124	-0.099	67.7	59.7	59.1	0.3	3	3	West Face	0.6	835000	0.6	0.6	0.316	0.249	East Face	0.9	0.4	0.9	1.3	-0.135	0.069
11	Column C	1.1	1.2	1.2	1.2	-0.12	-0.096	55.4	45.3	53.3	17.6	16	16	West Face	0.6	1100000	0.6	0.8	0.337	0.325	East Face	0.9	1.1	0.9	1.1	-0.132	0.123
12	Column C	1.3	1.3	1.3	1.3	-0.129	-0.076	61.3	62.7	63	27.4	25.8	25.8	West Face	0.7	600000	0.6	0.8	0.276	0.171	East Face	0.9	1.2	0.9	1.3	-0.125	-0.048
13	Column C	1.1	1.1	1.1	1.2	-0.111	-0.076	54.4	54	53	5.8	4.6	4.6	West Face	0.6	700000	0.6	0.6	0.252	0.133	East Face	0.9	0.8	0.9	1.1	-0.117	0.013
14	Column C	1.1	1.1	1	1.1	-0.109	-0.088							West Face	0.6	750000	0.6	0.6	0.234	0.115	East Face	0.8	0.9	0.9	1	-0.12	0.006
15	Column C	1	1.1	1	1.1	-0.113	-0.099							West Face	6	800000	0.6	0.6	0.243	0.169	East Face	0.8	1.1	0.9	1.1	-0.123	0.039
16	Column C	1	1.2	1.1	1.1	-0.112	-0.098	70.4	67.9	63.8	7.3	5.8	5.9	West Face	0.6	750000	0.6	0.6	0.184	0.198	East Face	0.9	0.8	0.9	1.1	-0.126	0.121
17	Column C	1.2	1.3	1.2	1.3	-0.118	-0.09	59.3	58	58.5	23.3	21.8	21.7	West Face	0.7	550000	0.7	1	0.081	0.206	East Face	0.9	1.1	0.9	1.2	-0.12	0.101
18	Column C	1.6	1.3	1.3	1.3	-0.115	-0.083	54.2	52.2	54.6	22	50.6	50.6	West Face	0.6	600000	0.6	0.8	0.068	0.099	East Face	0.9	1.1	0.9	1.2	-0.113	0.108
19	Column C	1	1	1	1.1	-0.109	-0.09							West Face	0.5	1200000	0.5	0.6	0.086	0.083	East Face	0.8	0.6	0.8	1	-0.102	0.1
20	Column C	0.9	1	0.9	1	-0.1	-0.094							West Face	0.5	1500000	0.6	0.6	0.016	0.202	East Face	0.9	0.6	0.9	1	0.103	0.038
21	Column C	1	1.1	1.1	1	-0.111	-0.117							West Face	0.6	1300000	0.5	0.7	0.005	0.172	East Face	0.8	0.7	0.8	1	-0.119	-0.037
22	Column C	1.1	1.1	1.1	1.1	-0.116	-0.109	55.4	55.4	58	10.9	9.5	9.3	West Face	0.6	0.6	600000	0.7	-0.037	0.187	East Face	0.9	0.9	0.9	1.2	-0.123	0.034
23	Column C	1.2	1.3	1.2	1.3	-0.112	-0.09	66	71.3	65.7	20.7	19	19.2	West Face	0.7	600000	0.6	0.7	-0.017	0.187	East Face	0.9	1.2	0.9	1.3	-0.116	0.091
24	Column C	1.1	1.2	1	1.2	-0.105	-0.083	49.8	63.3	53.6	10.2	8.7	8.7	West Face	0.6	800000	0.6	0.8	-0.007	0.169	East Face	0.9	0.9	0.9	1.9	-0.103	0.03
25	Column C	1	1.5	1	1.1	-0.105	-0.093							West Face	0.6	1200000	0.6	0.5	0.03	0.094	East Face	0.8	0.4	0.7	1	-0.101	0.021
26	Column C	1	1.5	1	1.1	-0.105	-0.093							West Face	0.6	1200000	0.6	0.5	0.03	0.094	East Face	0.8	0.4	0.7	1	-0.101	0.021
27	Column C	1.1	1.1	1	1.1	-0.118	-0.127							West Face	0.5	1200000	0.6	0.6	0.006	0.001	East Face	0.9	0.5	0.9	1.4	-0.118	0.027
28	Column C	1.1	1.1	1.1	1.1	-0.119	-0.123	76.4	72	70.4	7	5.6	5.6	West Face	0.9	600000	0.6	0.6	0.023	0.092	East Face	0.8	0.6	0.9	1.3	-0.126	0.02
29	Column C	1.2	1.2	1.3	1.2	-0.112	-0.104	59.4	62.9	63.2	15.9	13.5	13.7	West Face	0.6	600000	0.6	0.7	0	0.087	East Face	0.9	1.1	0.9	1.3	-0.115	-0.034
30	Column C	1.2	2.7	1.3	1.4	-0.117	-0.081	66.4	66	65.7	23.9	22.1	22	West Face	0.7	550000	0.6	0.8	-0.042	0.01	East Face	0.9	1.3	0.9	1.2	-0.107	0.053
31	Column C	1.1	1.2	1.1	1.3	-0.103	-0.078	62.8	69.7	65.6	9.3	7.5	7.3	West Face	0.7	900000	0.7	1	0.01	0.053	East Face	0.8	1	0.9	1.1	-0.102	0.05
32	Column C	1	1	1	1	-0.109	-0.1							West Face	0.6	1500000	0.6	0.6	-0.008	0.006	East Face	0.8	0.4	0.9	1	-0.119	0.045
33	Column C	1	1.1	1	1.1	-0.111	-0.112							West Face	0.6	1500000	0.6	0.7	-0.009	-0.049	East Face	0.8	0.7	0.9	1	-0.135	0.025
34	Column C	1.2	1.3	1.2	1.2	-0.112	-0.1		49.3	59		10.4	10.4	West Face	0.6	500000	0.6	0.7	-0.009	-0.054	East Face	0.9	0.9	0.9	1.2	-0.138	0.024
35	Column C	1.2	1.4	1.2	1.3	-0.105	-0.067		56.7	61.4		25.7	25.8	West Face	0.7	375000	0.6	0.8	-0.022	0.007	East Face	0.9	1.2	0.9	1.3	-0.106	0.012
36	Column C	1.2	1.3	1.2	1.2	-0.101	-0.064		56.9	61.8		16.6	16.5	West Face	0.6	4500000	0.6	0.7	-0.013	-0.057	East Face	0.9	1.1	0.9	1.2	-0.127	0.021
37	Column C	1	1	0.9	1	-0.109	-0.119							West Face	0.6	2100000	0.6	0.6	0.003	-0.098	East Face	0.7	0.8	0.8	0.9	-0.135	-0.004
38	Column C	1	1.1	1	1	-0.112	-0.105							West Face	0.5	1300000	0.6	0.6	0.007	-0.094	East Face	0.8	0.9	0.9	1.1	-0.142	-0.01
39	Column C	1.2	1.3	1.2	1.2	-0.109	-0.089		42.3	51.4		12.2	12.3	West Face	0.8	500000	0.6	0.7	-0.014	-0.076	East Face	0.9	1.1	0.9	1.1	-0.139	-0.006
40	Column C	1.3	1.3	1.3	1.4	-0.105	-0.06		33.7	53.9		26.7	26.5	West Face	0.7	450000	0.6	0.7	-0.01	-0.073	East Face	0.9	1.2	0.9	1.3	-0.132	-0.004
41	Column C	1.2	1.3	1.1	1.3	-0.099	-0.052		63.4	60.4		12.9	12.8	West Face	0.6	560000	0.6	0.7	-0.013	-0.078	East Face	0.9	1.1	0.9	1.2	-0.131	-0.011
42	Column C	1.5	1.3	1.2	1.2	-0.097	-0.05		47.5	53.3		13.7	13.3	West Face	0.6	600000	0.6	0.7	0.005	-0.083	East Face	0.9	1.1	0.9	1.1	-0.13	-0.01
43	Column C	0	0	0	0	0	0							West Face	0	0	0	0	0	0	East Face	0	0	0	0	0	0
44	Column C	0	0	0	0	0	0							West Face	0	0	0	0	0	0	East Face	0	0	0	0	0	0
45	Column C	1.2	1.3	1.3	1.4	-0.106	-0.073		53.6	54.6		25.5	25.6	West Face	0.7	280000	0.6	0.7	0.017	0.096	East Face	0.9	1.2	0.9	1.3	-0.142	0.036
46	Column C	1.2	1.3	1.2	1.2	-0.106	-0.079		47.4	63.2		10.2	10.1	West Face	0.6	700000	0.6	0.6	0.005	-0.124	East Face	0.9	1.1	0.9	1.1	-0.122	-0.001
47	Column C	1	1.1	1.1	1	-0.113	-0.105							West Face	0.6	1000000	0.6	0.6	0.004	-0.1	East Face	0.8	0.6	0.9	1.1	-0.147	-0.027
48	Column C	1.1	1.2	1.1	1.2	-0.118	-0.129		51.1	50.8		9.3	9.3	West Face	0.6	800000	0.6	0.7	0	-0.115	East Face	0.9	1	0.9	1.2	-0.13	-0.109

	GW	GX	GY	GZ	HA	HB	HC	HD	HE	HF	HG	HH	HI	HJ	HK	HL	HM	HN	HO	HP	HQ	HR	HS	HT	HU	HV	HW	HX	HY	HZ	IA	IB	IC	ID	IE
1	Pier 37S																																		
2					Resistivity Probes (Q)						Half-Cells (V)																		Resistivity Probes (Q)		Half-Cells (V)		Resistivity Probes (Q)		Half-Cells (V)
3	Pier 37S				Probe 1			Probe 2			Probe 1			Relative Humidity (%)							Ambient Temp (deg C)								Probe 1	Probe 2	Probe 1		Probe 1	Probe 2	Probe 1
4		Column	Temp. {°F	Date	1	2	3	1	2	3	1	2	3	1	2	3	4	5	6	7	1	2	3	4	5	6	7		1	1	1		1	1	1
5	Pier 37S	37D	30	1/4/2000	0.6	0.6	0.6	0.8	0.6	0.6	-0.419	-0.392	-0.383															West Face	0.8	0.8	-0.054	East Face	0.6	0.6	0.107
6	Pier 37S	37D	63	4/24/2000	0.7	0.6	0.6	0.8	0.6	0.6	-0.416	-0.371	-0.38	41.7			48.3			42.2	15.6			14.5			14.5	West Face	0.8	0.9	-0.091	East Face	0.5	0.6	0.054
7	Pier 37S	37D	70	6/26/2000	0.6	0.6	0.6	0.8	0.6	0.5	-0.38	-0.369	-0.376	59.5			62.2			61.7	21.8			20.3			20.2	West Face	0.8	0.9	-0.082	East Face	0.6	0.6	0.101
8	Pier 37S	37D	79	8/24/2000	0.7	0.6	0.6	0.8	0.6	0.6	-0.395	-0.386	-0.389	70.3			72.1			67.7	24.5			23.1			22.9	West Face	0.9	0.9	-0.073	East Face	0.6	0.6	-0.079
9	Pier 37S	37D	57	11/2/2000	0.6	0.5	0.6	0.8	0.6	0.6	-0.425	-0.408	-0.408	47.8			53..4			55.1	13.3			11.9			11.8	West Face	0.9	0.9	-0.065	East Face	0.5	0.6	-0.095
10	Pier 37S	37D	43	3/20/2001	0.6	0.6	0.6	0.8	0.5	0.6	-0.333	-0.342	-0.349	60.5			60.9			59	0.9			2.8			3.3	West Face	0.8	0.9	-0.053	East Face	0.6	0.6	-0.058
11	Pier 37S	37D	59	5/31/2001	0.6	0.6	0.7	0.8	0.6	0.6	-0.36	-0.374	-0.383	35.5			46			50.2	17.4			16			16	West Face	0.8	0.9	0.138	East Face	0.6	0.6	0.131
12	Pier 37S	37D	73	8/3/2001	0.7	0.7	0.7	0.8	0.7	0.6	-0.366	-0.372	-0.381	56.4			62.1			58.2	27.5			25.9			25.9	West Face	0.8	0.9	0.008	East Face	0.5	0.6	0.105
13	Pier 37S	37D	39	10/17/2001	0.6	0.6	0.6	0.8	0.6	0.5	-0.39	-0.384	-0.388	50.1			52.8			55.1	6.7			5.2			4.8	West Face	0.8	0.9	-0.031	East Face	0.5	0.6	0.095
14	Pier 37S	37D	30	12/20/2001	0.6	0.6	0.6	0.7	0.6	0.6	-0.395	-0.408	-0.423															West Face	0.8	0.8	-0.055	East Face	0.5	0.5	0.073
15	Pier 37S	37D	26	2/21/2002	0.8	0.6	0.5	0.7	0.5	0.6	-0.368	-0.38	-0.392															West Face	0.8	0.8	-0.05	East Face	0.5	0.5	0.084
16	Pier 37S	37D	46	4/29/2002	0.6	0.6	0.9	0.7	0.5	0.6	-0.373	-0.383	-0.396	68.7			62.4			58	7			5.6			5.5	West Face	0.9	1	-0.034	East Face	0.6	0.7	0.106
17	Pier 37S	37D	72	6/20/2002	0.7	0.6	0.6	0.8	0.6	0.6	-0.334	-0.335	-0.35	57			59.1			60.7	23.4			22			22.1	West Face	0.8	0.9	-0.001	East Face	0.6	0.6	0.111
18	Pier 37S	37D	69	9/3/2002	0.8	0.7	0.8	0.9	0.6	0.6	-0.385	-0.38	-0.391	50.3			56			56	22.4			21			21.1	West Face	0.8	0.9	-0.02	East Face	0.6	0.6	0.047
19	Pier 37S	37D	22	11/5/2002	0.6	0.6	0.6	0.7	0.5	0.6	-0.391	-0.374	-0.374															West Face	0.8	0.8	-0.014	East Face	0.7	0.7	0.055
20	Pier 37S	37D	25	1/28/2003	0.5	0.5	0.5	0.6	0.5	0.5	-0.362	-0.364	-0.365															West Face	0.7	0.8	-0.034	East Face	0.6	0.5	0.078
21	Pier 37S	37D	18	3/7/2003	0.5	0.6	0.6	0.7	0.6	0.5	-0.321	-0.346	-0.356															West Face	0.7	0.7	-0.054	East Face	0.5	0.5	0.101
22	Pier 37S	37D	44	5/20/2003	0.5	0.6	0.6	0.7	0.6	0.5	-0.321	-0.346	-0.356															West Face	0.7	0.7	-0.054	East Face	0.5	0.5	0.101
23	Pier 37S	37D	66	7/11/2003	0.8	0.6	0.7	0.8	0.6	0.6	-0.362	-0.382	-0.396	64.3			67.8			62.2	20.8			19.2			19.2	West Face	0.8	0.9	0.2	East Face	0.6	0.6	0.094
24	Pier 37S	37D	47	9/25/2003	0.7	0.6	0.6	0.7	0.6	0.6	-0.387	-0.389	-0.402	56.8			54.5			49.4	12.3			10			9.8	West Face	0.8	0.8	0.085	East Face	0.7	0.7	0.055
25	Pier 37S	37D	24	11/13/2003	0.6	0.5	0.6	0.7	0.6	0.6	-0.377	-0.379	-0.386									-2.1						West Face	0.8	0.8	0.113	East Face	0.7	0.6	0.03
26	Pier 37S	37D	24	11/13/2003	0.6	0.5	0.6	0.7	0.6	0.6	-0.377	-0.379	-0.386									-2.1						West Face	0.8	0.8	0.113	East Face	0.7	0.6	0.03
27	Pier 37S	37D	30	2/19/2004	0.6	0.6	0.6	0.8	0.6	0.6	-0.324	-0.334	-0.346															West Face	0.8	0.8	0.095	East Face	0.7	0.7	0.03
28	Pier 37S	37D	43	3/29/2004	0.6	0.6	0.5	0.7	0.6	0.6	-0.35	-0.377	-0.396	76.9			65			60.2	7			5.8			5.7	West Face	0.8	0.8	0.06	East Face	0.6	0.6	0.013
29	Pier 37S	37D	59	6/2/2004	0.6	0.7	0.6	0.8	0.7	0.6	-0.368	-0.382	-0.399	45.1			62.1			57.7	16.6			13.7			13.8	West Face	0.8	0.9	0.125	East Face	0.7	0.6	0.008
30	Pier 37S	37D	71	8/26/2004	0.6	0.6	0.6	0	0	0	0.8	0.6	0.7	63.5			63.6			62.2	23.5			22			22.1	West Face	0.9	0	0.9	East Face	0.9	0	0.6
31	Pier 37S	37D	50	10/20/2004	0.8	0.6	0.6	0.8	0.6	0.6	-0.348	-0.388	-0.398	68	64.4	65.9					8.7	7.1	7.1					West Face	0.9	0.9	0.131	East Face	0.9	0.6	0.002
32	Pier 37S	37D	0	12/15/2004	1.5	0.6	0.6	0.8	0.6	0.6	-0.368	-0.391	-0.409															West Face	0.7	0.8	0.088	East Face	3.9	0.5	-0.005
33	Pier 37S	37D	11	2/23/2005	0.6	0.6	0.6	0.7	0.5	0.6	-0.343	-0.37	-0.388															West Face	0.7	0.8	0.043	East Face	10.7	0.6	-0.005
34	Pier 37S	37D	48	4/22/2005	0.7	0.6	0.6	0.8	0.5	0.5	-0.348	-0.374	-0.403				67			49.5				10.6			10.5	West Face	0.8	0.9	0.198	East Face	0.6	0.5	-0.008
35	Pier 37S	37D	72	7/22/2005	0.8	0.7	0.6	0.8	0.6	0.6	-0.337	-0.35	-0.382		64.2	56						25.7	25.7					West Face	0.8	0.9	0.234	East Face	0.9	0.6	-0.002
36	Pier 37S	37D	64	9/16/2005	0.7	0.6	0.7	0.8	0.6	0.6	-0.37	-0.373	-0.401				61.2			55.8				16.5			16.5	West Face	0.8	0.9	0.194	East Face	0.9	0.6	-0.002
37	Pier 37S	37D	15	12/21/2005	0.7	0.6	0.6	0.7	0.6	0.5	-0.381	-0.399	-0.419															West Face	0.8	0.9	0.104	East Face	0.9	0.6	0.001
38	Pier 37S	37D	22	2/28/2006	0.8	0.6	0.6	0.7	0.6	0.6	-0.353	-0.391	-0.431															West Face	0.7	0.8	0.065	East Face	0.6	0.5	0.001
39	Pier 37S	37D	56	5/22/2006	0.7	0.6	0.6	0.8	0.6	0.6	-0.333	-0.358	-0.403				58			47.3				12.6			12.6	West Face	0.9	0.8	0.113	East Face	0.7	0.6	0.002
40	Pier 37S	37D	86	7/18/2006	0.7	0.7	0.7	0.9	0.6	0.6	-0.324	-0.34	-0.381				60.4			44.5				27.2			27.2	West Face	0.9	0.9	0.196	East Face	0.9	0.6	0
41	Pier 37S	37D	55	9/12/2006	0.9	0.6	7	0.8	0.6	0.6	-0.68	-0.377	-0.412				62.5			60.5				13.1			13.2	West Face	0.8	0.9	0.159	East Face	0.9	0.6	0.001
42	Pier 37S	37D	57	11/8/2006	0.6	0.7	0.6	0.8	0.6	0.6	-0.385	-0.395	-0.432				51.5			51.3				13.2			12.8	West Face	0.9	0.9	0.146	East Face	0.6	0.9	0
43	Pier 37S	37D	34	1/8/2007	0.6	0.6	0.6	0.8	0.6	0.6	-0.364	-0.4	-0.423															West Face	0.7	0.8	0.22	East Face	0.9	0.6	0.006
44	Pier 37S	37D	36	3/12/2007	0.6	0.6	0.6	0.7	0.5	0.6	-0.324	-0.347	-0.381															West Face	0.8	0.9	0.096	East Face	0.9	0.6	0
45	Pier 37S	37D	80	6/11/2007	1	0.6	0.7	0.8	0.6	0.6	-0.325	-0.347	-0.385				58.6			53.3				25.9			25.8	West Face	0.9	0.9	0.225	East Face	0.9	0.7	-0.001
46	Pier 37S	37D	54	10/19/2007	0.6	0.6	0.6	0.7	0.5	0.6	-0.387	-0.394	-0.42				46.3			64.6				10.5			10.4	West Face	0.8	0.8	0.292	East Face	0.9	0.6	0.004
47	Pier 37S	37D	30	12/3/2007	0.9	0.6	0.7	0.7	0.5	0.6	-0.385	-0.405	-0.436															West Face	1	0.8	0.126	East Face	0.9	0.6	0
48	Pier 37S	37D	48	4/29/2009	0.6	0.6	0.6	0.8	0.6	0.6	-0.025	-0.071	-0.105				53.6			42.4				9.1			9	West Face	0.8	0.8	0.235	East Face	1	0.5	0.006

	IF	IG	IH	II	IJ	IK	IL	IM	IN	IO	IP	IQ	IR	IS	IT	IU	IV	IW	IX	IY	IZ	JA	JB	JC	JD	JE	JF	JG	JH	JI	JJ	JK	JL	JM	JN	JO	JP	JQ	JR	JS	JT	JU	JV	JW	JX	JY	JZ	KA	KB
1	Pier 40N																																																
2					Resistivity Probes (Q)						Half-Cells (V)										Resistivity Probes (Q)		Half-Cells (V)								Resistivity Probes (Q)						Half-Cells (V)				Resistivity Probes (Q)						Half-Cells (V)		
3	Pier 40N				Probe 1			Probe 2			Probe 1			Relative Humidity (%)			Ambient Temp (deg C)				Probe 1	Probe 2	Probe 1	Relative Humidity (%)			Ambient Temp (deg C)				Probe 1			Probe 2			Probe 1				Probe 1			Probe 2			Probe 1		
4		Pier 40N	Temp. (°F	Date	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3		1	1	1	1	2	3	1	2	3		1	2	3	1	2	3	1	2	3		1	2	3	1	2	3	1	2	3
5	Pier 40N	Column A	30	1/4/2000	1.2	0.1	0.7	1.3	1.1	0.6	-0.136	-0.078	-0.116							Column C	1.1	0.9	-0.081							West Face	0.8	12300000	0.9	0.9	0.5	0.9	-0.225	-0.215	-0.103	East Face	1	1	0.9	1	0.9	0.9	-0.115	-0.211	-0.085
6	Pier 40N	Column A	67	4/24/2000	0.6	0.1	0.7	1.4	1.2	0.6	-0.134	-0.101	-0.158	78.5	52.3	71.4	13	12.9	14.2	Column C	1.1	1.1	-0.091	56.2	49.1	64.5	12.8	12.8	14.3	West Face	0.9	2600000	0.9	0.9	0.8	0.9	-0.255	-0.194	-0.135	East Face	1.1	1	0.9	1.2	1	0.9	-0.159	-0.197	-0.091
7	Pier 40N	Column A	69	6/26/2000	0.5	0.3	0.3	1.5	1.2	1.2	-0.137	-0.103	-0.164	64.6	63.8	86.3	21	19.4	19.2	Column C	1.2	1.1	-0.093	62.7	64	63.8	20.7	19	19	West Face	0.9	1400000	0.9	1	0.6	1.1	-0.306	-0.193	-0.135	East Face	1.2	1.2	0.9	1.2	1.1	0.9	-0.172	-0.193	-0.093
8	Pier 40N	Column A	79	8/24/2000	1.4	0.2	0.7	1.4	1.3	139	-0.134	-0.098	-0.159	76.7	71.1	93.9	23.8	22.3	22.3	Column C	1.2	1	-0.092	70.8	73	69.1	23.8	22.2	22.1	West Face	0.9	2000000	0.9	0.9	0.7	0.9	-0.292	-0.181	-0.131	East Face	1.2	1.2	0.9	1.2	1	0.9	-0.169	-0.18	-0.088
9	Pier 40N	Column A	57	11/2/2000	0.5	0.3	0.7	1.4	1.1	4.1	-0.135	-0.091	-0.146	55.3	51.6	66	11.9	10.4	10.4	Column C	1.1	1.1	-0.086	54.7	49.4	59.1	10.8	10.7	12.3	West Face	0.8	6000000	0.9	0.9	0.6	0.9	-0.277	-0.179	-0.12	East Face	1.1	1	0.9	1.1	0.9	0.9	-0.153	-0.178	-0.083
10	Pier 40N	Column A	43	3/20/2001	1.2	0.2	0.6	1.3	1.1	0.6	-0.141	-0.082	-0.127	74.3	73.9	79.1	-0.8	1.4	1.4	Column C	1.1	0.9	-0.085	62.4	57.2	57.5	0.5	3	2.5	West Face	0.9	2000000	0.9	0.9	0.6	0.8	-0.237	-0.218	-0.113	East Face	1	1	0.9	1.1	1	0.9	-0.132	-0.21	-0.082
11	Pier 40N	Column A	59	5/31/2001	1.5	0.3	0.8	1.5	1.3	0.8	-0.13	-0.09	-0.149	54.7	42.3	79.6	19.4	18.1	18.2	Column C	1.1	1	-0.087	65.1	40.4	44.2	18.5	16.9	16.7	West Face	0.9	1250000	0.9	0.9	0.7	0.9	-0.26	-0.215	-0.125	East Face	1.5	1.1	0.9	1.2	1	0.9	-0.157	-0.207	-0.081
12	Pier 40N	Column A	73	8/3/2001	1.4	0.3	0.8	1.5	1.3	500000	-0.131	-0.097	-0.158	62.2	62.6	67.3	26.6	252.2	25.4	Column C	1.1	1.1	-0.089	61.3	62.2	59.9	26.8	25.3	25.1	West Face	0.9	1000000	0.9	1	0.7	0.9	-0.314	-0.213	-0.128	East Face	1.2	1.2	1	1.3	1.1	1	-0.18	-0.2	-0.084
13	Pier 40N	Column A	39	10/17/2001	1	0.2	0.6	1.3	1.1	2	-0.133	-0.083	-0.131	61.5	48.1	70.2	7.4	6	6.1	Column C	1.1	1	-0.087	49.1	47.5	48.7	7.9	6.7	6.4	West Face	0.9	1950000	0.9	0.9	0.5	0.9	-0.247	-0.2	-0.109	East Face	1	1	0.9	1.1	0.9	0.9	-0.15	-0.184	-0.073
14	Pier 40N	Column A	30	12/20/2001	1.3	0.2	0.7	1.2	1.1	0.9	-0.133	-0.08	-0.144							Column C	0.9	0.9	-0.089							West Face	0.8	1300000	0.9	0.9	0.6	0.9	-0.222	-0.196	-0.118	East Face	1	1	0.9	0.9	0.9	0.9	-0.134	-0.185	-0.081
15	Pier 40N	Column A	26	2/21/2002	1.3	0.2	0.7	1.2	1.1	0.9	-0.133	-0.08	-0.144							Column C	0.9	0.9	-0.089							West Face	0.8	1300000	0.9	0.9	0.6	0.9	-0.222	-0.196	-0.118	East Face	1	1	0.9	0.9	0.9	0.9	-0.134	-0.185	-0.081
16	Pier 40N	Column A	46	4/29/2002	1	0.2	0.7	1.4	1.2	3.7	-0.131	-0.083	-0.129	59.9	50.9	53.7	9.6	9.2	8.6	Column C	1	1	-0.087	57.8	57.3	56	9.8	8	7.8	West Face	0.9	1000000	0.9	0.9	0.6	0.9	-0.233	-0.224	-0.12	East Face	0.9	1	0.9	1.1	1	0.9	-0.149	-0.209	-0.085
17	Pier 40N	Column A	72	6/20/2002	0.9	0.3	0.7	1.5	1.2	600000	-0.129	-0.085	-0.149	53.6	55.8	54.7	23.6	22.2	22	Column C	1.1	1.1	-0.087	53.8	56.1	83.5	23.7	22.4	22.6	West Face	0.9	1200000	0.9	1	0.6	1	-0.28	-0.22	-0.124	East Face	1.1.	1.1	0.9	1.2	1	1	-0.172	-0.202	-0.081
18	Pier 40N	Column A	69	9/3/2002	1	0.2	0.8	1.4	1.3	600000	-0.13	-0.085	-0.144	46.4	45.3	73.5	23.5	22.5	22.6	Column C	1.1	1.1	-0.084	44.8	51.9	52.8	23.4	21.7	21.7	West Face	0.9	2000000	0.9	0.9	0.6	0.9	-0.332	-0.213	-0.118	East Face	1.2	1.2	0.9	1.2	1.1	1	-0.175	-0.194	-0.077
19	Pier 40N	Column A	22	11/5/2002	1.5	0.1	0.6	1.2	1	1.5	-0.131	-0.073	-0.113							Column C	1	0.9	-0.081							West Face	0.8	2000000	0.8	0.9	0.4	0.8	-0.278	-0.2	-0.1	East Face	1	1	0.9	1.1	0.9	0.9	-0.129	-0.182	-0.07
20	Pier 40N	Column A	25	1/28/2003	1.1	0.2	0.6	1.2	1	2.5	-0.12	-0.069	-0.105							Column C	1	0.9	-0.077							West Face	0.8	3000000	0.9	0.9	0.5	0.9	-0.197	-0.237	-0.094	East Face	1	1	0.8	1	0.9	0.9	-0.108	-0.21	-0.072
21	Pier 40N	Column A	18	3/7/2003	0.2	0.2	0.6	1.2	1	1.3	-0.123	-0.07	-0.109							Column C	0.9	0.9	-0.081							West Face	0.8	1900000	0.9	0.8	0.4	0.9	-0.203	-0.227	-0.106	East Face	0.9	1	0.9	1	0.9	0.9	-0.117	-0.21	-0.081
22	Pier 40N	Column A	44	5/20/2003	0.2	0.2	0.6	1.2	1	1.3	-0.123	-0.07	-0.109							Column C	0.9	0.9	-0.081							West Face	0.8	1900000	0.9	0.8	0.4	0.9	-0.203	-0.227	-0.106	East Face	0.9	1	0.9	1	0.9	0.9	-0.117	-0.21	-0.081
23	Pier 40N	Column A	66	7/11/2003	1.4	0.3	0.7	1.5	1.2	500000	-0.126	-0.081	-0.143	65.7	66.4	71.6	21.5	20.2	20.3	Column C	1.1	1.1	-0.081	63.4	66.9	61	20.9	19.4	19.2	West Face	0.9	1200000	0.9	1	0.7	0.9	-0.309	-0.223	-0.11	East Face	1.2	1.1	0.9	1.2	1.1	0.9	-0.153	-0.201	-0.074
24	Pier 40N	Column A	47	9/25/2003	0.5	0.2	0.6	1.3	1.1	5.7	-0.135	-0.065	-0.129	46.1	49.4	55.2	10.9	9.1	9.3	Column C	1.1	0.9	-0.077	32.3	43.6	41	10.8	9.3	9.5	West Face	0.9	4000000	0.9	0.9	0.6	0.9	-0.338	-0.207	-0.102	East Face	1.1	1.2	0.9	1.1	1	0.9	-0.14	-0.183	-0.07
25	Pier 40N	Column A	24	11/13/2003	1.2	0.3	0.7	1.1	1	26.9	-0.13	-0.069	-0.115		-2					Column C	1	1	-0.073							West Face	0.8	2300000	0.9	0.9	0.6	0.9	-0.237	-0.2	-0.09	East Face	0.9	1	0.8	1	0.9	0.9	-0.119	-0.18	-0.068
26	Pier 40N	Column A	24	11/13/2003	1.2	0.3	0.7	1.1	1	26.9	-0.13	-0.069	-0.115		-2					Column C	1	1	-0.073							West Face	0.8	2300000	0.9	0.9	0.6	0.9	-0.237	-0.2	-0.09	East Face	0.9	1	0.8	1	0.9	0.9	-0.119	-0.18	-0.068
27	Pier 40N	Column A	30	2/19/2004	1.5	0.5	0.7	1.4	0.9	0.9	-0.111	-0.065	-0.113							Column C	1	0.9	-0.065							West Face	0.8	1200000	0.9	0.9	0.5	0.9	-0.187	0.248	-0.085	East Face	1	1	0.9	1	1.1	1	-0.121	-0.232	-0.071
28	Pier 40N	Column A	43	3/29/2004	1.2	0.2	0.7	1.4	1.1	3.5	-0.127	-0.068	-0.132	80.2	69	79	6.9	5.4	5.4	Column C	1.3	1	-0.078	76.2	63.1	74.8	6.4	4.9	4.9	West Face	0.9	1000000	0.9	0.9	0.6	0.9	-0.243	-0.231	-0.118	East Face	0.9	1	0.9	1.1	0.9	0.9	-0.146	-0.217	-0.09
29	Pier 40N	Column A	59	6/2/2004	1.6	0.2	0.7	1.3	1.1	700000	-0.131	-0.063	-0.127	75.7	59.9	80.1	16.1	14.7	14.8	Column C	1	1	-0.075	52.8	57.9	53	16.3	13.9	14	West Face	0.9	650000	0.9	0.9	0.7	0.9	-0.226	-0.22	-0.102	East Face	1.1	1.1	0.9	1.2	1.1	0.9	-0.142	-0.209	-0.075
30	Pier 40N	Column A	71	8/26/2004	2.1	0.3	0.7	1.6	1.3	600000	-0.121	-0.072	-0.128	57.2	59.7	76.9	22.1	22.6	22.6	Column C	1.1	1.2	-0.073	63.4	64.6	63.7	23.8	22.5	21.9	West Face	0.9	1000000	0.9	0.9	0.7	0.9	-0.269	-0.206	-0.097	East Face	1.2	1.1	0.9	1.2	1.1	0.9	-0.15	-0.2	-0.068
31	Pier 40N	Column A	50	10/20/2004	1.4	0.2	0.8	1.4	1.1	1200000	-0.127	-0.065	-0.106	63	70.7	67.2	9.3	7.5	7.4	Column C	0.9	1	-0.071	71.3	67.1	75.1	9.3	7.5	7.6	West Face	0.9	3700000	0.9	0.9	0.6	1.1	-0.221	-0.196	-0.084	East Face	1	1.1	0.9	1.1	1.2	0.9	-0.118	-0.192	-0.065
32	Pier 40N	Column A	0	12/15/2004	1.1	0.3	0.6	1.2	1	7.8	-0.119	-0.046	-0.094							Column C	0.9	0.9	-0.072							West Face	0.8	900000	0.9	0.9	0.6	0.8	-0.204	-0.21	-0.11	East Face	0.9	1	0.9	1	0.9	0.9	-0.106	-0.201	-0.082
33	Pier 40N	Column A	11	2/23/2005	0.7	0.3	0.7	1.3	1.1	7.3	-0.128	-0.051	-0.096							Column C	0.9	0.9	-0.071							West Face	0.9	1300000	1	0.9	0.4	0.9	-0.17	-0.234	-0.128	East Face	0.9	1	0.9	1.1	0.9	0.9	-0.107	-0.215	-0.086
34	Pier 40N	Column A	48	4/22/2005	0.5	1.2	0.7	1.4	1.1	600000	-0.128	-0.063	-0.126		49.7	50.2		10.1	10.1	Column C	1	1	-0.077		56.4	73		10.3	10.2	West Face	0.9	800000	0.9	0.9	0.6	0.9	-0.263	-0.221	-0.127	East Face	1.1	1.1	0.9	1.1	1	0.9	-0.142	-0.209	-0.084
35	Pier 40N	Column A	72	7/22/2005	1.9	0.3	0.8	1.5	1.3	1500000	-0.124	-0.079	-0.128		78.3	66.1		25.8	25.7	Column C	1.1	1.1	-0.074		56.2	53.9		25.4	25.5	West Face	1.6	900000	0.9	0.9	80.6	0.9	-0.263	-0.21	-0.118	East Face	1.5	1.1	0.9	1.3	1.1	0.9	-0.149	-0.199	-0.075
36	Pier 40N	Column A	64	9/16/2005	1.5	0.2	0.7	1.4	1.2	900000	-0.128	-0.071	-0.118		64.3	77.3		16.4	16.8	Column C	1	1.1	-0.075		58.9	55.7		16	16	West Face	0.9	1790000	0.9	0.9	900000	0.9	-0.245	-0.2	-0.1	East Face	1.1	1.1	0.9	1.1	1	1.1	-0.133	-0.19	-0.072
37	Pier 40N	Column A	15	12/21/2005	1.3	0.2	0.7	1.3	1	30	-0.123	-0.049	-0.081							Column C	1	1.1	-0.049							West Face	0.8	2000000	0.8	0.9	400000	0.8	-0.224	-0.208	-0.123	East Face	1	0.9	0.8	1	0.9	0.9	-0.095	-0.194	-0.084
38	Pier 40N	Column A	22	2/28/2006	1.7	0.2	0.7	1.4	1.1	22.6	-0.118	-0.055	-0.099							Column C	0.9	0.9	-0.068							West Face	0.9	1500000	0.8	0.9	200000	0.8	-0.269	-0.214	-0.131	East Face	0.9	0.9	0.9	0.9	0.9	0.9	-0.108	-0.205	-0.095
39	Pier 40N	Column A	56	5/22/2006	1.2	0.2	0.8	1.4	1.2	2.9	-0.128	-0.069	-0.115		43.5	71.3		10.5	10.6	Column C	1.1	1	-0.072		33.1	39.4		10.7	10.8	West Face	0.9	400000	0.9	0.9	93.2	0.9	-0.324	-0.213	-0.127	East Face	1.2	1	1	1.1	1	0.9	-0.134	-0.21	-0.09
40	Pier 40N	Column A	86	7/18/2006	1.5	0.3	0.8	1.5	1.3	600000	-0.123	-0.083	-0.128		56.7	72.4		27.2	27.4	Column C	1.1	1.2	-0.068		31.9	38.3		27	27	West Face	0.9	600000	0.9	1	200000	0.9	-0.361	-0.201	-0.128	East Face	1.1	1.2	1	1.2	1.1	1	-0.146	-0.196	-0.081
41	Pier 40N	Column A	55	9/12/2006	1.4	0.4	0.7	1.5	1.1	800000	-0.126	-0.075	-0.109		44.1	72.1		13.7	13.8	Column C	1	1	-0.069		54	54.8		13.2	13	West Face	0.9	1400000	0.9	0.9	1500000	0.9	-0.343	-0.19	-0.109	East Face	1.1	1.1	0.9	1.2	1	0.9	-0.127	-0.19	-0.074
42	Pier 40N	Column A	57	11/8/2006	1.4	0.4	0.8	1.4	1.2	750000	-0.102	-0.069	-0.108		54.4	68.6		13	13.2	Column C	1	1	-0.066		45.3	49.7		13.7	13.7	West Face	0.9	500000	0.9	0.9	49.4	0.9	-0.271	-0.188	-0.097	East Face	1.1	1.2	0.9	1.2	1	0.9	-0.118	-0.194	-0.072
43	Pier 40N	Column A	34	1/8/2007	1.2	0.5	0.9	1.3	1.1	1400000	-0.116	-0.066	-0.112							Column C	0.9	0.9	-0.058							West Face	0.8	500000	0.9	0.9	109.2	0.8	-0.219	-0.212	-0.103	East Face	1	1	0.7	1	0.9	0.9	-0.106	-0.205	-0.094
44	Pier 40N	Column A	36	3/12/2007	1	0.3	0.7	1.4	1.1	720000	-0.123	-0.095	-0.113							Column C	1	0.9	-0.069							West Face	0.9	0.65	0.9	0.9	300000	0.9	-0.27	-0.227	-0.1	East Face	1	1	0.9	1.1	1	0.9	-0.115	-0.224	-0.374
45	Pier 40N	Column A	80	6/11/2007	1.5	1.1	0.7	1.5	1.1	530000	-0.118	-0.088	-0.123		57.4	70.4				Column C	1.1	1.1	-0.065	47.7		49.1	21.4		21.2	West Face	0.9	350000	0.9	0.9	NR	0.9	-0.29	-0.209	-0.109	East Face	1.2	1.1	0.9	1.2	1.1	0.9	-0.13	-0.203	-0.073
46	Pier 40N	Column A	54	10/19/2007	0.6	0.9	0.9	1.5	1.1	800000	-0.124	-0.074	-0.11		47.4	68.6		10.3	10.2	Column C	1	1	-0.068	47.5		61.3	10.4		10.6	West Face	0.9	3E+08	0.8	0.9	800000	0.9	-0.229	-0.207	-0.096	East Face	1.1	1	0.9	1.1	1	0.9	-0.119	-0.195	-0.062
47	Pier 40N	Column A	30	12/3/2007	1.2	0.2	1	1.3	1.2	1.5	-0.119	-0.057	-0.099							Column C	1.3	1.1	-0.062							West Face	1	580000	0.9	0.9	350000	0.9	-0.197	-0.209	-0.097	East Face	0.9	1	0.9	1	0.9	0.9	-0.102	-0.207	-0.078
48	Pier 40N	Column A	48	4/29/2009	1.4	0.9	1.1	1.4	1.1	5.2	-0.098	-0.066	-0.118		55.2	67.4		9.6	9.4	Column C	1	1	-0.055		57.8	63.5		9.3	9.3	West Face	0.9	300000	0.9	0.9	15.2	0.9	-0.221	-0.206	-0.112	East Face	1	1.1	0.9	1.1	1	0.9	-0.115	-0.196	-0.073

APPENDIX C: CORROSION POTENTIAL DATA - 2018 STUDY

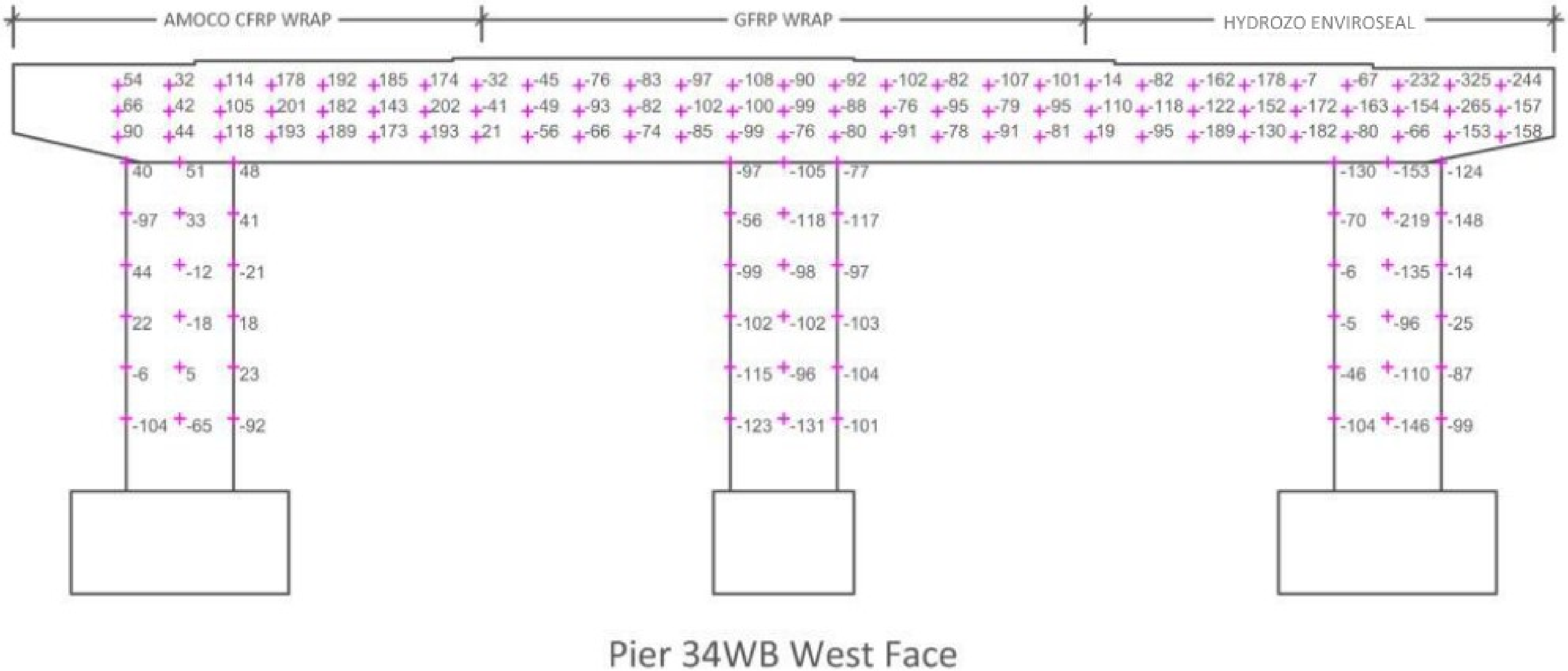

Figure C.1 Corrosion potential measurements (mV vs CSE) for west face of Pier 34WB [ECE].

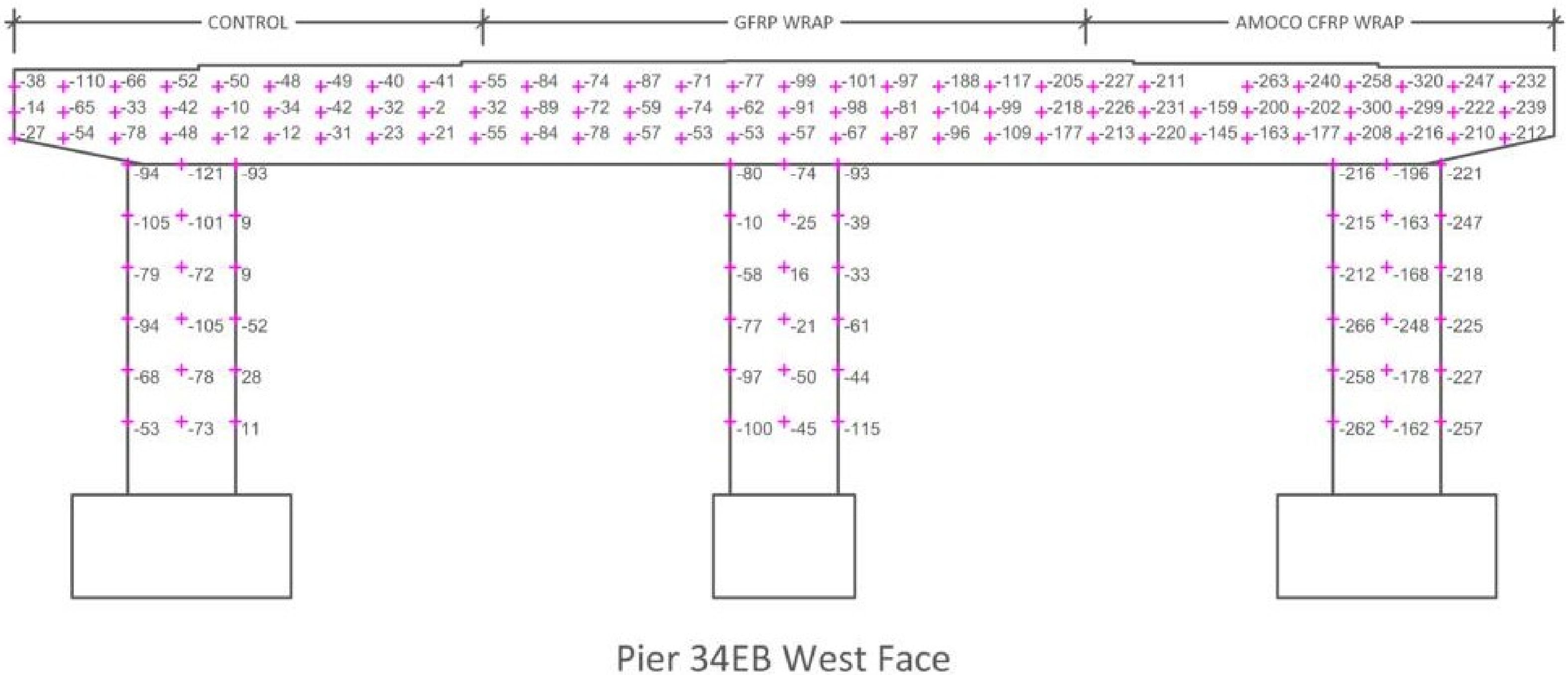

Figure C.2 Corrosion potential measurements (mV vs CSE) for west face of Pier 34EB [non-ECE].

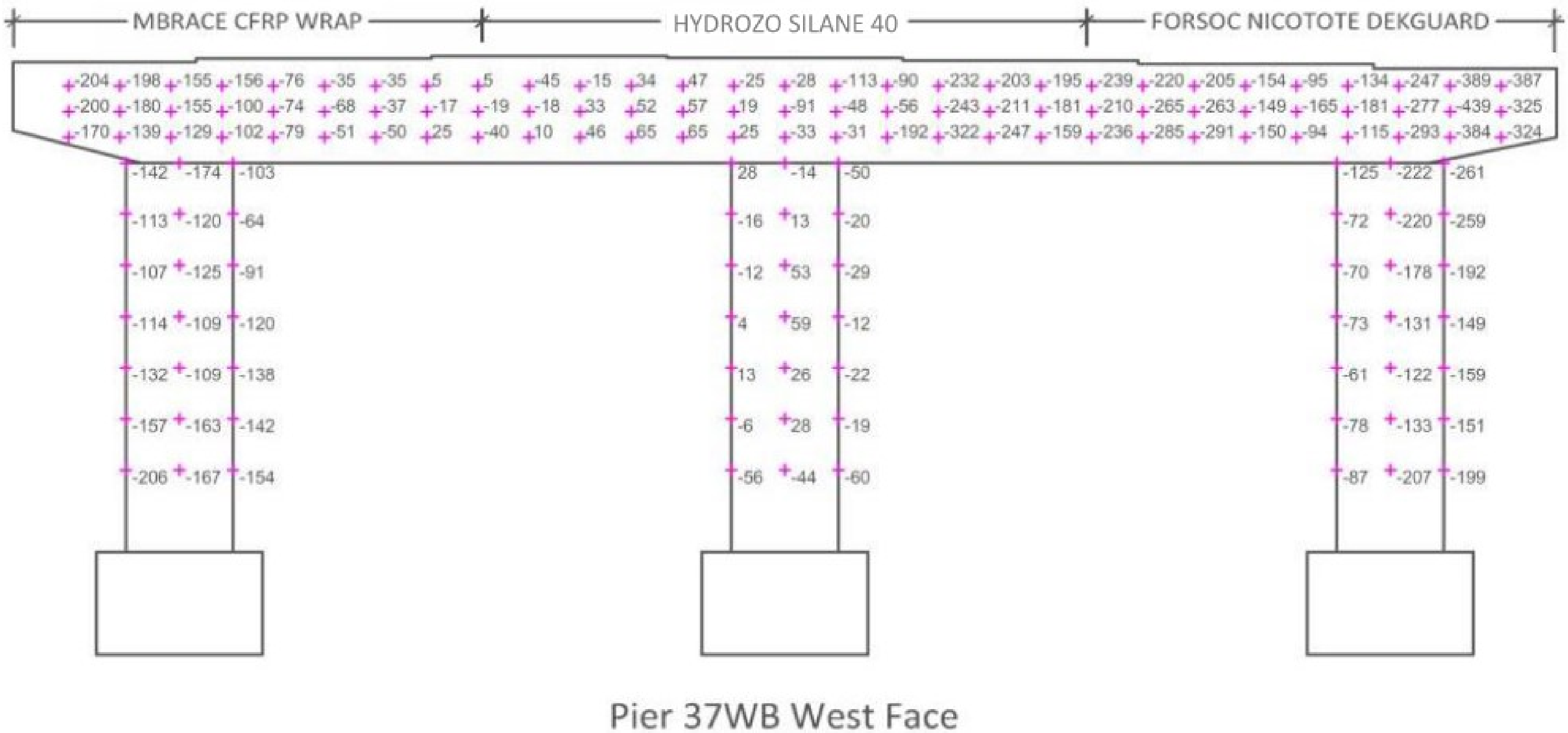

Figure C.3 Corrosion potential measurements (mV vs CSE) for west face of Pier 37WB [ECE].

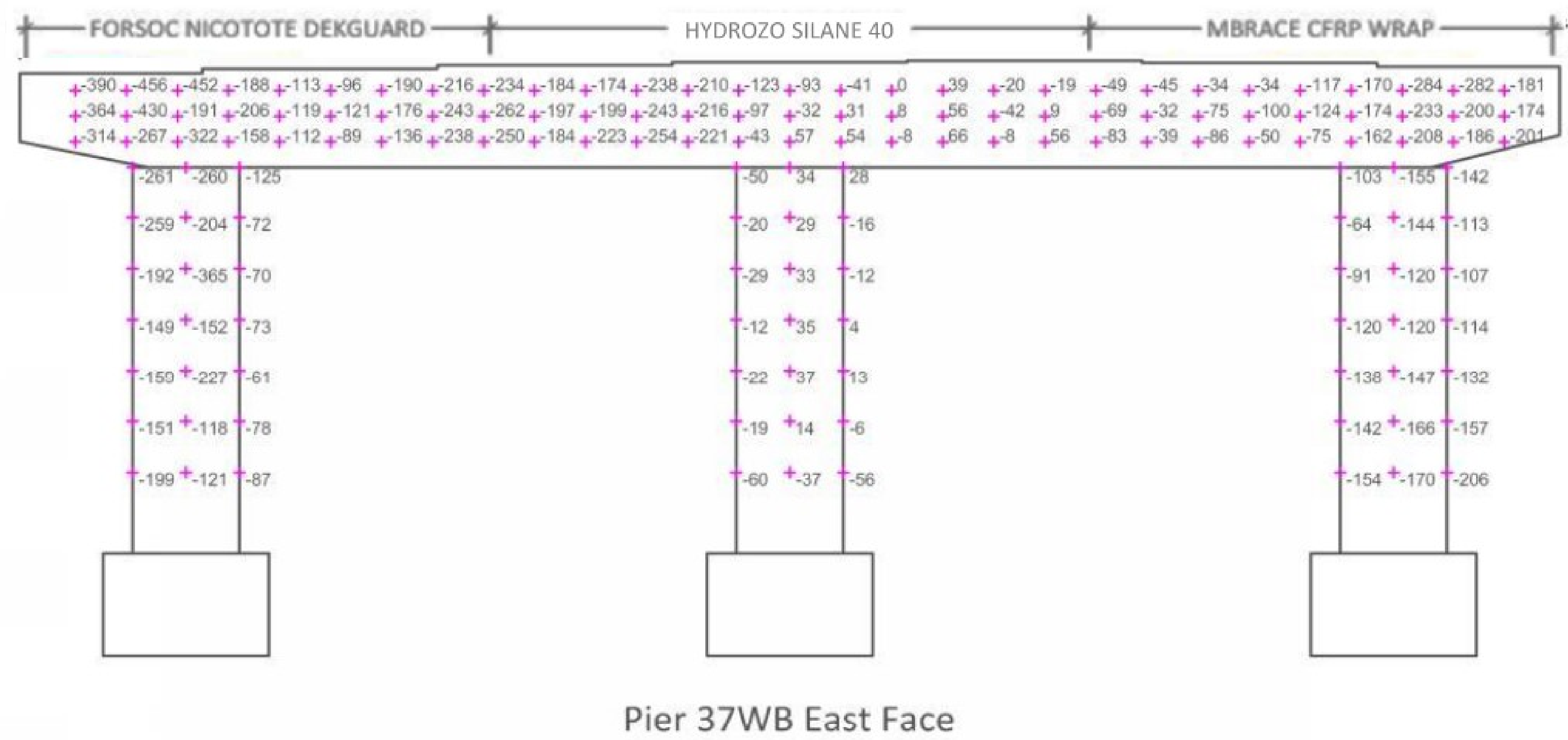

Figure C.4 Corrosion potential measurements (mV vs CSE) for east face of Pier 37WB [ECE].

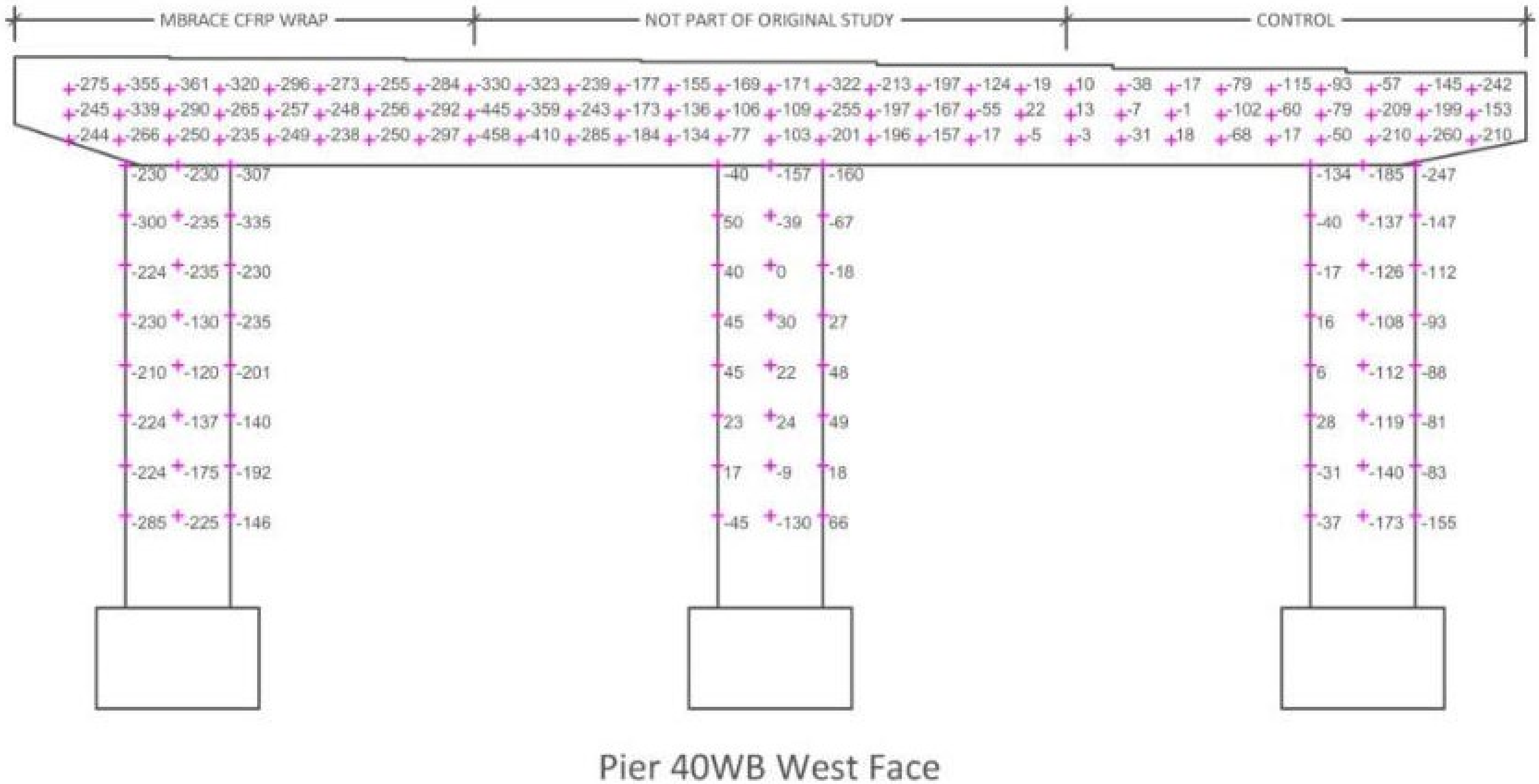

Figure C.5 Corrosion potential measurements (mV vs CSE) for west face of Pier 40WB {non-ECE].