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USE OF FRP FOR CORROSION MITIGATION APPLICATIONS IN A MARINE ENVIRONMENT

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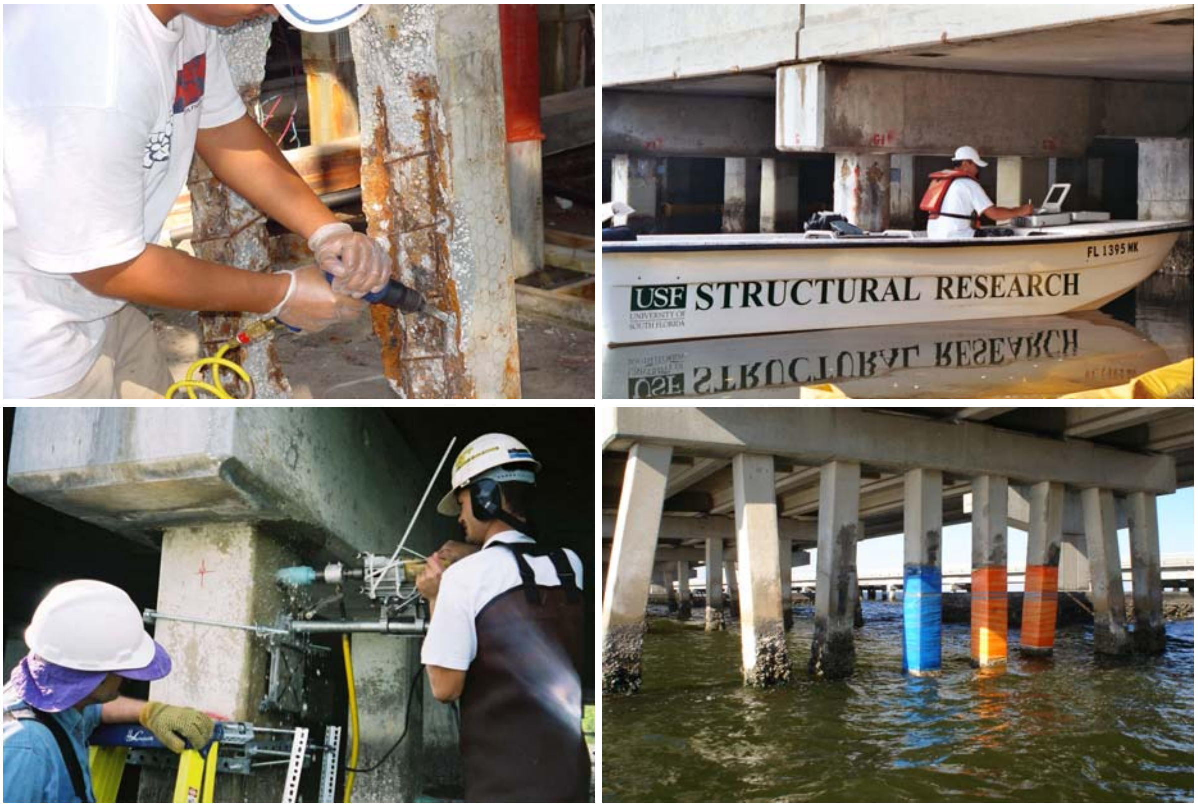

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16. Abstract

This report presents results of laboratory and field studies to evaluate the role of fiber reinforced polymers in repairing corrosion damaged piles in a marine environment. Both CFRP and GFRP were evaluated in laboratory testing and in field demonstration studies using instrumented piles.

The focal point of the laboratory study was to determine whether a FRP wrap reduced corrosion in prestressed specimens when the chloride threshold level was exceeded. Results from exposure tests under ambient and accelerated conditions convincingly showed that the FRP was effective in slowing down the corrosion rate. In both exposures, identical wrapped and unwrapped specimens were subjected to simulated tidal cycles under ambient or nominal 140F temperature. For ambient exposure, the reduction in strand weight loss in newly fabricated CFRP and GFRP wrapped specimens was only 50% that in unwrapped controls. In the hot water exposure the increase in strand metal loss in unwrapped specimens initially pre-corroded to a target 25% and then exposed for nearly 2 years was 64.1% compared to 12.1% in the worst performing FRP wrapped specimen. Gravimetric and ultimate strength tests from this study also showed that the effect of the type of repair prior to wrapping was relatively unimportant. Results for full and epoxy repair were comparable.

Two field demonstration projects were conducted using both dry (coffer dam) and wet wrap systems. In both studies, a significant number of piles were instrumented to allow corrosion assessment. Corrosion rate in the wrapped piles was found to be lower in comparison to unwrapped controls and CFRP had lower rates than GFRP. On-site bond tests carried out showed that the bond from one of the two wet wrap systems evaluated was comparable to that from the dry system using cofferdam construction. Overall, findings suggest that FRP wrap is a viable method for repairing corrosion-damaged piles.

17. Key Word

corrosion, marine, prestressed piles, fiber reinforced polymers, repair, rehabilitation, under water, coffer dam, design, wrapping, simulated tides, accelerated, bond, linear polarization

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USE OF FRP FOR CORROSION MITIGATION APPLICATIONS IN A MARINE ENVIRONMENT

Final Report on a Research Project
Sponsored by

Florida and US Department of Transportation

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October 2005

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Multiply	***by***	***to obtain***
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound (lb)	4.448	newtons
kip (1000 lb)	4.448	kilo newton (kN)
newton	0.2248	pound
kip/ft	14.59	kN/meter
pound/in^2	0.0069	MPa
kip/in^2	6.895	MPa
MPa	0.145	ksi
ft-kip	1.356	kN-m
in-kip	0.113	kN-m
kN-m	0.7375	ft-kip

PREFACE

The investigation reported was funded by a contract awarded to the University of South Florida, Tampa by the Florida Department of Transportation. The assistance and guidance of Mr. Jose Garcia and Mr. Steve Womble from the FDOT throughout the project is gratefully acknowledged.

We are indebted to Mr. John Hooks, Federal Highway Administration, Office of Infrastructure RD & T for loaning equipment that was used throughout the project for corrosion monitoring. We also thank Mr. Alan Hyman, Mr. Ben McKinney , Mr. Ron Meade, Mr. Todd Hammerle, Mr. Michael Kenis, Mr. Seta Koroitamudu (District 5), Mr. Jose Rodriguez, Dr. Jose Danon (D7) for their interest and support. We thank Mr. Ivan Lasa, State Materials Office, for his contribution to this study. We also thank Dr. Alberto Sagues for his guidance on the field instrumentation scheme adopted in this project.

Specimens were cast at Henderson Prestress, Tarpon Springs. We are very grateful to Mr. Dirk Henderson for accommodating us yet again and for assisting us in every manner to ensure success. Fiber reinforced polymer materials used in the study were donated by Air Logistics Inc. and Fyfe and Co. We thank Ed Fyfe, President, Fyfe for his generosity. We also thank Ms Sarah Witt, Mr. Zach Smith, Fyfe, Mr. Paul Pinet, Premier Corrosion Protection Services, and Mr. Carlos Farabell for their assistance. We are indebted to Air Logistics, Pasadena, CA for their technical assistance and help with the under water FRP wrap. We especially thank Mr. Franz Worth and Mr. Steve Bazinet who participated in the laboratory demonstration, in the field wrap and provided equipment required for pullout testing.

Full-scale pile segments used for field instrumentation mock-up instrumentation were provided by Mr. John Robertson, Standard Concrete Products with assistance from Mr. Matias, Linder Rental Company and Pro-Cutting Concrete Cutters, Inc who cut the pile into 6 ft. segments that were shipped to USF. Their assistance is gratefully acknowledged.

This study could not have been completed without the enthusiastic support and assistance of undergraduate and graduate students not supported by the project. The following undergraduate students helped in the two concrete pours: Rebecca Ambrose, Jimmy Daniels, Amber Drew, Michele Eceiza, Balter Exceus, Bryan Fast, Bill Gore, Jadon Hull, Jessica Schenning, Cheron Thornton and Jeff Vogel. Graduate student Kadir Uslu worked tirelessly on the project. We really appreciate his contribution. We also thank graduate students Mike Stokes, Van Wagner, Kevin Johnson, Ed Garbin and Ivan Gualtero and undergraduate student Julio Aguilar for their help.

EXECUTIVE SUMMARY

This report presents findings from a 44 month laboratory and field study to evaluate the effectiveness of fiber reinforced polymers (FRP) in strengthening corrosion-damaged piles in a marine environment. In the study, both carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) systems were evaluated.

The laboratory studies investigated: (1) expansion due to corrosion, (2) the effect of the fiber type and number of FRP layers on the corrosion rate, (3) the effect of exposure on the FRP-concrete bond, (4) the role of surface preparation on FRP performance, and (5) strengthening effectiveness of underwater wrap using a newly developed water-activated resin. The demonstration studies evaluated two disparate FRP systems – two wet wrap and a dry wrap system using coffer dam construction (undertaken by SDR Engineering and described in Appendix C). Piles were instrumented to allow determination of the corrosion rate and on-site pullout tests conducted to evaluate the FRP-concrete bond.

The focal point of the laboratory study was to determine if FRP wrapping slowed down the corrosion rate in specimens where corrosion was about to initiate. For this purpose, identical one-third scale specimens were fabricated in a prestressing yard with cast-in chlorides over 22 in. length corresponding to the splash zone in the prototype pile. Twenty two of these specimens were used in an ambient exposure study. Sixteen of these were wrapped using 1, 2, 3 or 4 CFRP or GFRP layers. Of the remaining six, four were outdoor controls and two were indoor controls. The twenty outdoor specimens were placed upright in a salt water tank and subjected to two daily simulated tidal cycles in which the water level inside the tank was changed every six hours. Throughout the nearly three-year exposure period, corrosion measurements were regularly taken. At the end of the exposure period, the specimens were removed from the tank and individual strands and ties carefully removed and cleaned. The metal loss over the 22 in. chloride contaminated region was determined by gravimetric testing in which the cleaned strands and ties were accurately weighed. The results of the tests showed that the weight loss in both the CFRP and GFRP wrapped strands was only 50% of that in the unwrapped controls. On an average, three of the seven wires in a strand in the unwrapped controls were broken because of corrosion. In contrast, there was only one break in a wire among the 16 wrapped specimens, i.e. out of 64 strands. This test conducted under normal outdoor conditions on newly fabricated specimens convincingly demonstrated that FRP can slow can down the corrosion rate in specimens that were about to corrode.

In addition to the ambient exposure study, accelerated exposure tests were conducted to evaluate the performance of the wrap in repair situations where the specimen had already undergone significant corrosion. In this study a total of 26 specimens were first corroded to a targeted 25% metal loss verified by gravimetric testing. 22 of these specimens were wrapped using different CFRP and sealing schemes. The remaining 4 served as controls for gravimetric and strength testing. Prior to wrapping, specimens were either fully repaired in which deteriorated concrete was removed, strands cleaned, coated with corrosion inhibitor and the section re-formed or simply epoxy repaired in which only cracks were sealed with epoxy. These specimens were then placed upright in an outdoor covered tank where they were exposed to the same simulated tidal regime as the ambient specimens excepting that the temperature of the salt water was maintained at a nominal 140F throughout. At the end of the nearly 2-year exposure period, specimens were removed and the effect of the exposure determined from gravimetric and ultimate strength test in which the specimens were loaded to failure under eccentric axial loads. The results of the gravimetric test showed that the increase in the metal loss in the strands in unwrapped controls was 64.1% compared 12.1% in the worst performing wrapped specimens.

The gravimetric results also showed that the performance of the full and epoxy repair were comparable. These findings were confirmed by strength tests where the ultimate capacities were found to be similar.

Pull out tests were carried out to evaluate the FRP-concrete bond for selected specimens from the ambient exposure study. These specimens had all been wrapped under dry conditions. Bond was evaluated at three different levels – the constantly dry region, the constantly wet submerged region and the region in the middle that was subjected to wet/dry cycles. The results of the tests showed that the average bond stress was similar for all three regions for both the carbon and glass systems. The average bond stresses for carbon in these three regions varied between 264 psi to 284 psi. The corresponding values for the glass system varied between 260-300 psi. Several of the failures occurred at the FRP/concrete interface and not in the concrete. While the GFRP gave higher average values, the number of concrete failures from the CFRP was lower.

In the field demonstration studies, piles were FRP wrapped at two contrasting sites – in the shallow waters of Allen Creek and the much deeper and more turbulent waters of Tampa Bay. The first demonstration project was conducted on the piles supporting Allen Creek Bridge, Clearwater (#150036) in 2002-2003. The second demonstration project conducted on piles supporting the Gandy Bridge (#100300) was completed at the end of 2004. These different sites posed different sets of instrumentation and access problems that required different strategies for their solution. In the first case, piles could be accessed by ladders but instrumentation had to unobtrusive so that it would not attract the unwelcome attention of vandals in the urban environment. In the second case, an innovative scaffolding system had to be developed for access but vandalism was not as great a consideration in the middle of Tampa Bay.

In the first demonstration study (Allen Creek Bridge), two disparate systems – a dry wrap and a wet wrap using a water-activated resin system – were tested. The wet wrap system eliminated the need for coffer dam construction. Prior to the application of the wet wrap system, a laboratory demonstration was conducted in which specimens pre-corroded to a targeted 25% metal loss were wrapped underwater in an outdoor tank. Subsequently, ultimate strength tests were undertaken immediately after the wrap had cured and after additional corrosion in a constant current accelerated system. Laboratory tests showed that despite the low bond values (120-130 psi), underwater strengthening was effective. The ultimate capacity of the underwater strengthened specimens exceeded their original capacity.

In the Allen Creek Bridge, a total of eight 14 in x 14 in prestressed piles were wrapped over a 5 ft length – four using a dry system and four using the water-activated wet wrap system. Two other piles in a similar state of disrepair were instrumented and served as controls. The dry system was the same used for wrapping carbon in the ambient and accelerated exposure studies. This is a wet lay-up. In the wet system, two piles were wrapped using carbon and two others using fiberglass. The wet system is a pre-preg were the FRP material is saturated with epoxy by the supplier and sent to the site in hermetically sealed pouches so that it does not cure in contact with atmospheric moisture.

Four of the eight wrapped piles were instrumented in addition to the two controls. Instrumentation consisted of 42 in. long 3/16 in. diameter 316 stainless steel bars that were embedded in each of the four pile faces. The ends of these bars had a 90 degree hook at the top that protruded from the pile surface. This system eliminated the need for wiring and junction boxes since connections needed could be made using alligator clips. This system allowed the corrosion rate in the piles to be measured. The system has proven robust and has been in use for more than 2 ½ years. Results from the measurements show that the corrosion rates in the wrapped piles, whether carbon or glass, were lower than that in the unwrapped controls. The performance of the piles wrapped using carbon from the dry and wet systems was comparable. However, corrosion rates were higher in the fiberglass wrapped pile using the wet system though the rates are still very low.

On-site bond tests were conducted on two piles using the dry and the wet systems at two different levels after the wrap had been in place for more than 26 months. The average bond at the top and bottom for the dry system was 260 and 105 psi respectively. The corresponding values for the wet system were 130 psi and 14 psi for carbon and 50 psi and 29 psi for glass.

In the second demonstration project on the Gandy Bridge, two alternate wet systems were evaluated. These were the water-activated pre-preg system used previously and a new wet lay up system by Fyfe. Three 20 in x 20 in prestressed piles were wrapped over a 6 ft length. Two were wrapped using carbon and the water activated system and the third using the wet lay up system. The three wrapped piles and a fourth control were instrumented using two different probes – a rebar probe developed by the State Materials Office and a commercial probe developed by Concorr Inc. The piles were wrapped at the end of last year. The system has not yet fully stabilized though corrosion measurements have been taken regularly.

On-site bond tests were carried out six months after wrapping. These pullout tests were conducted at three different levels – at the top where it is always dry and at two levels in the tidal region that are subjected to wet / dry cycles. The average bond for the water activated pre-preg resin system at the top, bottom and middle was respectively 130 psi, 50 psi and 72 psi (these were higher than those measured in the Allen Creek Bridge). The corresponding values for the wet lay-up system were 36 psi, 188 psi and 145 psi. The latter values in the wet region exceeded the 105 psi bond strength that was obtained using the dry system in the Allen Creek Bridge.

Overall, the laboratory study clearly demonstrates that FRP slows down the corrosion rate. The field measurements for the Allen Creek Bridge show that the corrosion rates are lower in wrapped specimens irrespective of whether carbon or glass is used. The bond values are on the low side though laboratory tests showed that this did not adversely affect strengthening. However, a wet wrap system gave better bond values than a dry wrap system that used costly cofferdam construction. This is especially encouraging. Further improvements in bond and innovative installation are likely to make FRP wrap more competitive in the future.

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1. INTRODUCTION

1.1 Introduction

Florida's long coastline and sub-tropical climate have led to the rapid deterioration of reinforced or prestressed concrete piles exposed to a marine environment. Generally, cracking and corrosion of the reinforcement lead to concrete spalling and further acceleration of the deterioration process. The resulting concrete spalling is usually repaired by patching which is only a sacrificial cover masking the existing problem. These patches are ineffective and most often require repair and replacement in a very short time.

Fiber reinforced polymers (FRPs) have been widely used in seismic regions for enhancing column ductility. In such applications, the FRP material is wrapped around the column with the fibers aligned at right angles to the column axis. The high circumferential tensile stresses resulting from the large earthquake-induced cyclic deformations are effectively resisted by the high-strength FRP material. The role of the FRP is essentially to limit the circumferential strains, referred to as *confinement*. As a result of this (external) confinement both strength and ductility of the column is increased with the FRP wrap providing passive restraint, i.e. until an earthquake event it is unstressed.

A similar potential exists for FRP’s successful use in repairing corrosion-damaged piles. In this case, circumferential tensile stresses are induced by the increase in volume due to corrosion of the reinforcement. Proper use of FRP material can effectively resist the resulting hoop stresses. Moreover, it has been suggested that increased confinement can decrease post-repair corrosion rates by (1) serving as a diffusion barrier to inhibit the ingress of chlorides, oxygen and moisture through the cover, and (2) by compressing the corrosion products and changing its electro-chemistry.

Previous research studies have largely focused on the application of FRP for corrosion repair typically caused by deicing salts. Here, the concrete surface is dry and application conditions are quite different from those for piles. As such, these findings may not fully apply for pile repair.

This research project presents the results from a multi-year study to evaluate the effect of FRP wrapping on the corrosion performance of square prestressed members exposed to a marine environment. The goal of the project was to conduct laboratory tests and field trials to assess the effectiveness of FRP wrap in restoring the strength of deteriorated prestressed concrete piles and in reducing the corrosion rate.

1.2 Objectives

The principal objective of the study was to conduct laboratory and field research to evaluate the effectiveness of alternate FRP systems in reducing the post-repair corrosion rate in prestressed piles. Laboratory studies were intended to quantify the effectiveness of the FRP through strength and gravimetric testing. The field studies were demonstration projects that evaluated this new repair system through appropriate instrumentation and monitoring.

1.3 Organization of Report

This report is organized into eleven chapters and three appendices that describe various aspects of the study. For convenience, all references cited are listed at the end of the respective chapters.

Chapter 2 provides an overview of the entire project. Detailed information on all materials used is summarized in Chapter 3 for easy reference. The fabrication of the prestressed concrete test specimens is described in Chapter 4. The design of FRP requires information on the expansion caused by corrosion. Chapter 5 presents results from an experimental study to measure this value. Two separate experimental studies were undertaken to determine the effectiveness of FRP in slowing down the corrosion rate. Chapter 6 contains details of a study in which the performance of two different systems using carbon fiber and glass were evaluated under simulated tidal cycles in ambient conditions. Chapter 7 describes a study in which role of surface preparation on the effectiveness of FRP was investigated. Here, specimens were wrapped in carbon fiber but were subjected to simulated tidal conditions using hot salt water.

Two field demonstration projects were conducted. In the first of these studies both dry-wrap (coffer dam construction) and wet-wrap were evaluated. Since a new type of water-activated resin was being used, laboratory testing was conducted to evaluate its effectiveness in strengthening. This is reported in Chapter 8. The field demonstration study using this new wet-wrap system is described in Chapter 9. The dry-wrap was conducted by SDR Engineering who served as consultant for the project. Their report is contained in Appendix C. The second demonstration study evaluated two different wet-wrap systems. This is described in Chapter 10. The conclusions and recommendations from the study are summarized in Chapter 11.

Three appendices contain supplementary information relating to the Surface Preparation Study (Appendix A), Column Study and Ambient Exposure Study (Appendix B) and SDR’s Report (Appendix C).

2. PROJECT OVERVIEW

2.1 Introduction

This chapter provides an overview of the entire project. The goal of the project was to assess the effectiveness of FRP for repairing corroding prestressed specimens in a marine environment. Information on the laboratory studies is summarized in Section 2.2 while Section 2.3 describes the field studies.

2.2 Laboratory Study

A number of laboratory studies were initiated to obtain information for assessing the effectiveness of FRP in slowing down the rate of corrosion and also for designing the wrap. Laboratory tests used one-third scale models of 18 in. square prestressed piles that had been found to be representative of piles observed to corrode in a marine environment [2.1-2.2]. An overview of specimen geometry, FRP materials and corrosion simulation schemes is included in this section.

2.2.1 Geometry

All specimens were prestressed by four 5/16 in. low relaxation Grade 270 strands. The 6 in. x 6 in. cross-section was a 1/3rd scale model of 18 in. prestressed piles. A fifth unstressed strand was provided at the center of the cross-section to serve as an internal cathode for an impressed current accelerated corrosion scheme used (see Section 2.2.3). A 22 in. segment at the center of the specimen was cast with 3% chloride ions to model the “splash zone”. Class V special concrete was used (Chapter 3) and the concrete cover was 1 inch. #5 gage spirals spaced 4.5 in. on centers were provided in the chloride contaminated region. The cross-section of the specimens is shown in Fig. 2.1.

Table 2.1 Specimen Breakdown

Test	5 ft	6 ft	Comments
Corrosion Expansion	8		Chapter 5
Ambient Exposure	22		Chapter 6 – simulated ambient tidal cycles
Surface Preparation	16	10	Chapter 7 - effect of surface preparation
Strengthening Columns	1	10	Chapter 8 – evaluation of underwater epoxy

Specimens were either 5 ft or 6 ft long. The 5 ft specimens were used for assessment by gravimetric testing in which corroded strands are retrieved from a specimen and actual metal loss measured. The 6 ft specimens were used where assessment was carried out by eccentric column testing. Table 2.1 shows a breakdown on the distribution of specimens in the various laboratory studies. The largest number of specimens was used in the two exposure studies (Chapters 6 and 7).

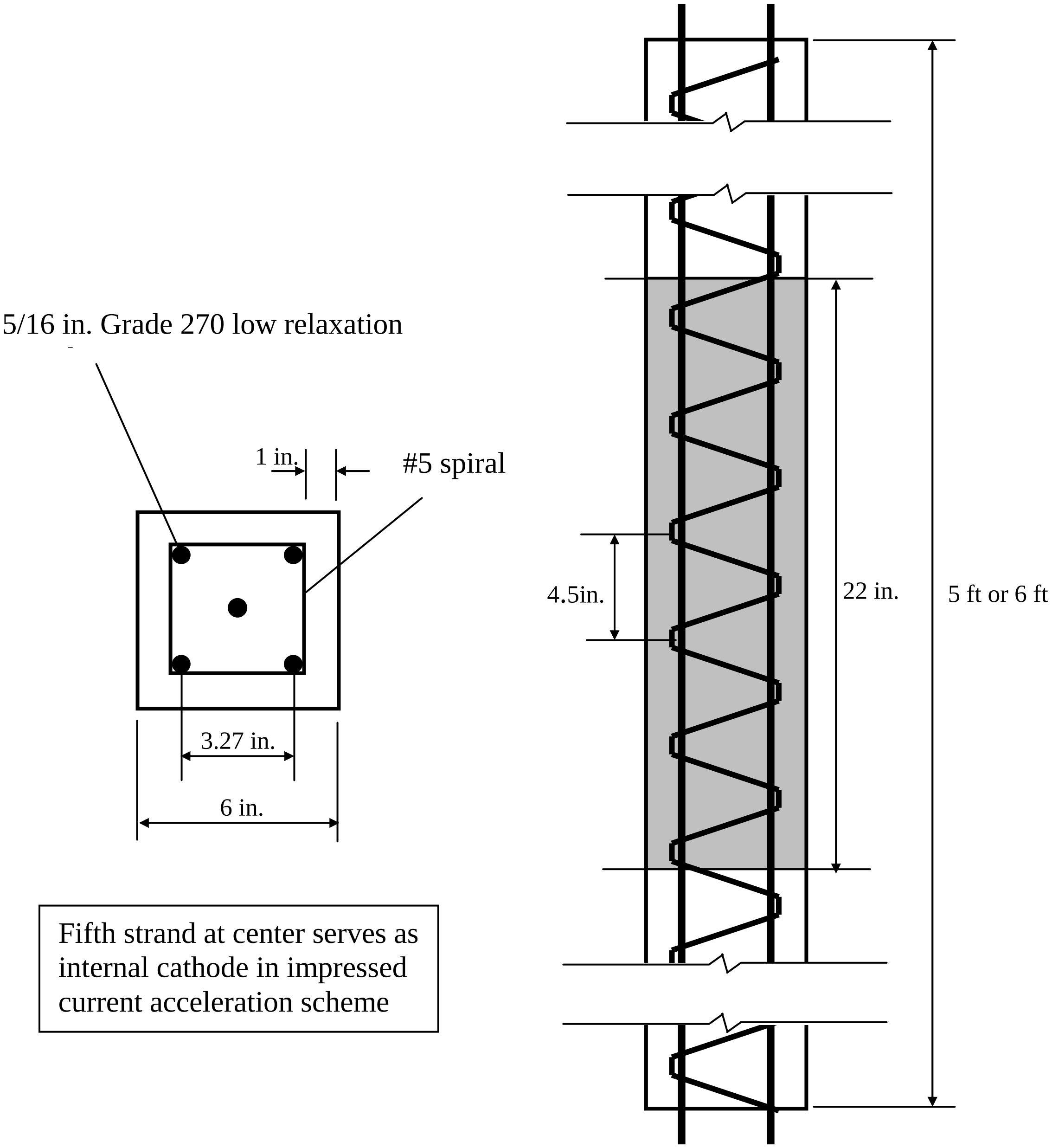

Figure 2.1 Specimen Geometry

2.2.2 FRP Material

Three different FRP systems were evaluated. These were systems developed by Fyfe, MAS and Air Logistics. Both glass and carbon fiber were tested. All fibers were bi-directional. The number of FRP layers was varied from 1 to 4. Other variables considered were (1) wrap length (2) seal vs no seal (3) exposure to simulated tidal cycles in ambient conditions (4) exposure to hot water simulated tidal cycles. Details on the important parameters investigated are summarized in Table 2.2.

Table 2.2 FRP Material

Test	Wrap Material	Variables	Comments
Ambient Exposure	Glass fiber (Fyfe)	Number of FRP layers, Wet/dry cycle, effect of variable temperature, and humidity	Chapter 6 – evaluation by gravimetric testing – Bond evaluated
	Carbon fiber (MAS)		
Surface Preparation	Carbon fiber (MAS)	Number of FRP layers, wet/dry cycles in hot salt water, effect of sealing, effect of wrap length	Chapter 7 – evaluation by gravimetric and eccentric column test
Column Strengthening	Carbon (Air Logistics)	Wet surface wrap, ultimate strength, bond	Chapter 8 – Underwater wrap

2.2.3 Corrosion Simulation

Three different systems were used to corrode specimens including two accelerated schemes. The accelerated systems were used to corrode specimens to targeted steel loss levels and to evaluate the post-wrap performance. A third non-accelerated system using simulated tidal cycles was used to evaluate corrosion instrumentation and to provide a measure of the validity of the two accelerated schemes.

Impressed current systems are widely used for accelerating corrosion of steel in laboratory testing. Two types of impressed current using constant voltage and constant current are used [2.3]. The constant voltage system is easy to use since all it requires is a connection to a battery but its performance may be erratic. A constant current system is more difficult since circuitry has to be designed to maintain the same current despite changes in the resistance of the steel and the concrete as the specimen corrodes.

In this study, a constant current system from a previous investigation [2.4] was used. This required an impressed current of 110 mA to corrode specimens shown in Fig.

2.1. This current corresponded to a current density of 100 μA/cm^2 - the upper limit in naturally corroding specimens exposed to a marine environment. In the previous laboratory study performed at USF using similar prestressed specimens, this impressed current induced the 10% steel loss after 50 days.

All four strands and ties were assumed to be anodes as they were electrically connected in the concrete, and the central strand was used as a cathode (Fig. 2.2). To ensure that there was electrical continuity between strands, all four strands protruding from the end of the specimen were connected. Both the anode and the cathode were electrically connected to a 40V battery and special circuitry was designed so that the current was constant in each specimen. To lower the resistivity of the concrete in the chloride contaminated region, sponges and soaker hoses were used to keep this region moist.

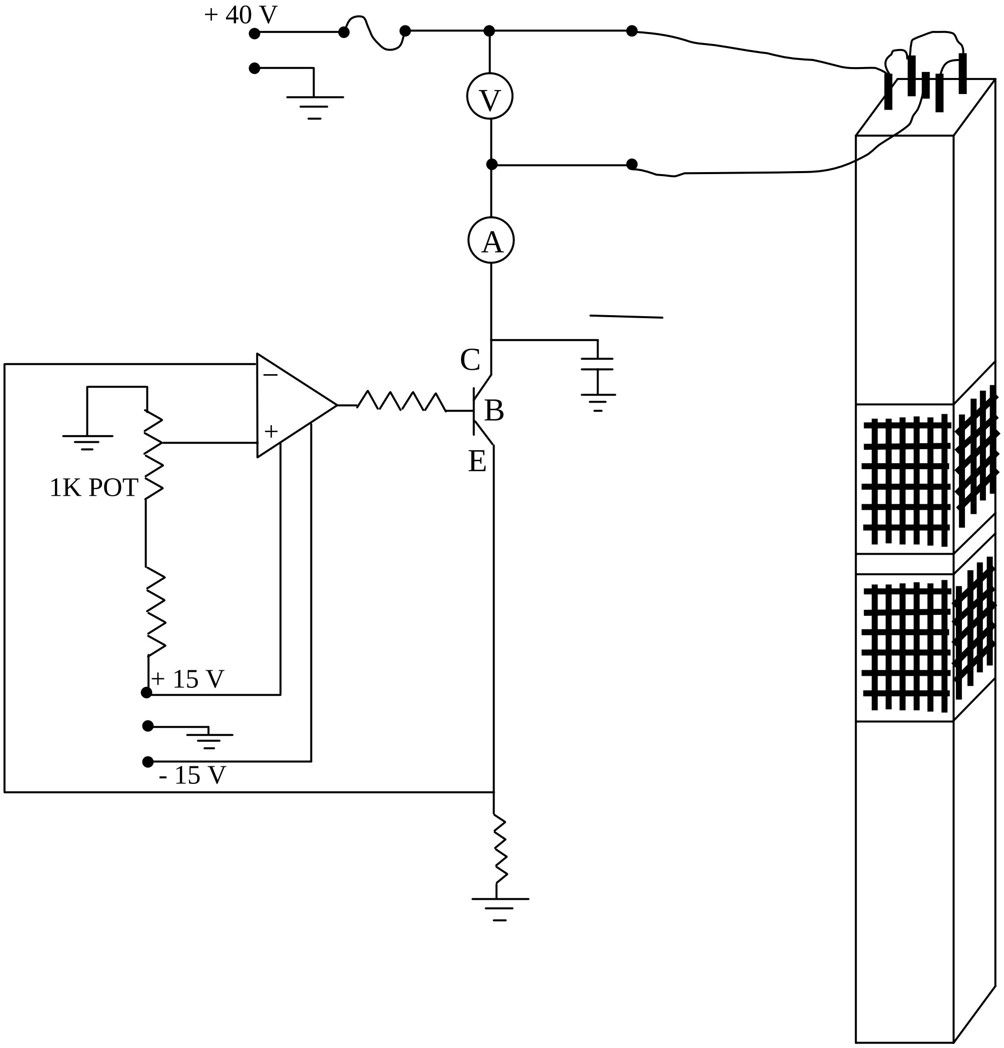

Figure 2.2 Schematic Drawing of Circuit

In addition to the impressed current system, two separate simulated tidal cycles were developed. In these tidal cycles, the water level was varied between 32 in. (high tide) and 14 in. (low tide) with the 'tide' changed every six hours. This was used in the ambient exposure described in Chapter 6 that used 5 ft long specimens. A similar scheme was used to evaluate the post-wrap performance but in this case the water was heated to accelerate corrosion. The targeted temperature of the hot water was 60C. Details are provided in Chapter 7. Table 2.3 summarizes details on the various corrosion acceleration systems used in the laboratory studies.

Table 2.3 Corrosion Acceleration Schemes Used

Test	Acceleration Method	Comments
Expansion	Constant current (110 mA) No water	Chapter 5
Ambient Exposure	Wet/dry cycles Salt water	Chapter 6
Surface Preparation	Pre-Wrap Constant current (110 mA) Water-sponge system Post-Wrap Wet/dry cycles (Hot) Salt Water (Hot)	Chapter 7
Column Strengthening	Constant current (110 mA) Water-sponge system	Chapter 8

2.3 Field Study

Two field demonstration studies were conducted to evaluate the effectiveness of two alternate systems (1) a "dry" wrap requiring cofferdam construction, and (2) a "wet" wrap that did not require cofferdam construction. In the first study both dry and wet wrap systems were evaluated at the same site. In the second study, two alternate wet wrap systems were evaluated at a different site. In both cases, piles were instrumented to allow measurement of the corrosion rate through linear polarization.

The site for the first field study was the Allen Creek Bridge, Clearwater (#150036) suggested by the Florida Department of Transportation. It met critical access requirements, e.g. shallow waters, proximity to the university, yet provided an aggressive environment with a long history of severe substructure corrosion problems in piles.

A total of eight of 14 in. x 14 in. prestressed piles were used in the FRP wrap. Four of the piles were repaired by the “dry” wrap system using carbon fiber (MAS) supplied by SDR Engineering. Three of the piles were wrapped by two layers and the other using four layers. The report submitted by SDR is contained as a self standing document in Appendix C. The under water system (“wet” wrap) was performed using another four piles. Two of the piles were repaired using 2 layers of carbon fiber and the other two were wrapped using four layers of glass fiber. The strengthening provided by the wraps in the two systems was comparable. Two other piles at the same site were used as unwrapped controls.

To allow the post-wrap performance of the piles to be used, six piles were instrumented including two controls, two piles repaired by the “dry” system, and two by the under water system. An innovative system was installed that eliminated the need for junction boxes and wiring. The system has worked well throughout the duration of this project. Details of the field study may be found in Chapter 9 and in Appendix C.

The second field study was conducted on piles supporting the Gandy Bridge (#100300). In this study, three 20 in. x 20 in. prestressed piles were wrapped using two different wet wrap systems, the pre-preg system used earlier in the Allen Creek Bridge and a wet-lay up system developed by Fyfe. Two of the piles were wrapped using carbon and the other by glass (Fyfe). The fiber layout was identical to that used in the earlier bridge. All wrapped piles and an unwrapped control were instrumented using rebar probes developed by the State Materials Office and a probe developed by Concorr Inc. In this case, both wiring and junction boxes were necessary. Details of this field study may be found in Chapter 10.

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3. MATERIAL PROPERTIES

3.1 Introduction

This chapter presents information on the properties of all the materials used for the fabrication of the prestressed test specimens, their subsequent repair using FRP and conventional materials. Details of the Class V concrete mix design used are summarized in Section 3.2. Modifications required to make the chloride-contaminated concrete are discussed in Section 3.3. Properties of the spiral ties and prestressing steel are included in Section 3.4 and Section 3.5, respectively. Information on the FRP materials used for wrapping the corroding specimens under dry conditions is contained in Section 3.6. Section 3.7 summarizes information on the properties of the materials used for the under water FRP repair. Other information on materials used for the surface preparation study is included in Section 3.8.

3.2 Concrete Mix Design

Two types of concrete were required because a 22 in. segment of the specimen was chloride contaminated. Both used the same Class V Special concrete. Because the chloride admixture used was also a water reducing agent, its mix design was slightly different.

Table 3.1 summarizes the design requirements for FDOT's Class V Special concrete. An approved mix provided by Florida Rock Industries was used in the study [3.1]. Information on this mix is summarized in Tables 3.2 and 3.3. The mix used 852 lb of cementitious material and 3/8 in. aggregates.

Table 3.1 Class V Special Design Requirements

Criteria	Requirement
Compressive Strength	6,000 psi
Cement Content	8.5 cwt/yd^3
Water to Cement Ratio	0.33
Slump	6.5 (+/- 1.5) in.
Air	2 %
Fine Aggregate Volume	42.3 %
Unit Weight	142.3 lb/ft^3

Table 3.2 Approved Mix Details

Materials	Quantities (SSD Basis)	Volume (ft^3)
Type II Cement	702	3.57
Fly Ash Class F	150	1.09
Silica Sand	1198	7.30
#89 Cr. Limestone	1510	9.96
Water	283	4.54
Darex AEA	0.5 oz.	0.54
WRDA-64	34.0 oz.	-----
Adva Flow	30.0 oz.	-----

Table 3.3 Aggregate Data

Aggregate	S.G.	Unit Weight	Gradation (% Passing U.S. Sieve)							
			½"	3/8"	#4	#8	#16	#30	#50	#100
Fine	2.63	105	-----	-----	100	100	94	62	18	2
Coarse	2.43	89	100	97	42	9	3	-----	-----	-----

3.3 Chloride-Contaminated Concrete

To make the chloride contaminated concrete, Daraccel was used as an admixture. Daraccel is manufactured and supplied by W.R. Grace & Co. It is both a water reducing and an accelerating admixture that uses calcium chloride. As a result, the water reducing admixture, WRDA-64 used for the regular concrete (Table 3.2) was not required.

Table 3.4 presents the mix design for the chloride contaminated concrete. It is identical to the regular concrete mix excepting for the replacement of 34 oz. of the WRDA-64 admixture by 1408 oz. of Daraccel. Each ounce of Daraccel provides approximately 0.0182 lb of chloride ion so that 1408 ounces provide 1408 x 0.0182 = 25.6 lbs/cy that corresponds to the targeted 3% by weight of cementitious material.

Daraccel was directly mixed on site since it is an accelerating agent. Details are given in Ch. 4.

Table 3.4 Chloride Contaminated FDOT Class V Special Mix

Materials	Quantities (SSD Basis)	Volume (ft^3)
Type II Cement	702	3.57
Fly Ash Class F	150	1.09
Silica Sand	1198	7.30
#89 Cr. Limestone	1510	9.96
Water	283	4.54
Darex AEA	0.5 oz.	0.54
Daraccel	1408 oz.	-----
Adva Flow	30.0 oz.	-----

3.4 Spiral Ties

The spiral reinforcement used for the fabrication of the prestressed specimens was fabricated using #5 gage steel. The ties were specially fabricated by Wire Products Incorporated of Florida. Material properties of the steel provided by the manufacturer [3.2] are summarized in Table 3.5.

Table 3.5 Spiral Ties Specifications

Basic Wire 4 in. × 4 in. Spirals	
Diameter	0.208 in.
Area	0.034 $in.^2$
Tensile Strength	99.7 ksi
Yield Strength	92.6 ksi
Area Reduction	62 %

3.5 Prestressing Steel

The size of the prestressing steel used was decided from modeling considerations for the one-third scale laboratory specimens used in this project. This led to the choice of 5/16 in. strands. Low relaxation, seven wire, Grade 270, steel strands were used. The material properties provided by the manufacturer [3.3], Strand Tech Martin, Inc. are summarized in Table 3.6.

Table 3.6 Prestressing Strands Specifications

Properties	Value
Tensile Strength	270 ksi
Breaking Load	16,000 lbs.
Load @ 1 % Ext.	14,400 lbs.
Nominal Area	0.059
Minimum Elongation	3.5%

3.6 CFRP and Tyfo® WEB Composite

Two different types of materials - carbon fiber reinforced polymer (CFRP) provide by SDR Engineering [3.4] and Tyfo® WEB glass fiber reinforced polymer manufactured by Fyfo Co. LLC. - were used for wrapping prestressed specimens in this study. CFRP is a 0°/90° bi-directional weave carbon fabric. The material properties of the fiber and the cured laminate are listed in Tables 3.7-3.8.

The Tyfo® WEB Composite [3.5] is composed of Tyfo® WEB reinforcing fabric and Tyfo® S Epoxy. Tyfo® WEB is a 0°/90° bi-directional weave glass fabric and its material properties provided by the manufacturer, Fyfe Co. LLC, are summarized in Table 3.9. Details on the Tyfo® S Epoxy are given in Table 3.10.

Table 3.7 Carbon Fiber Properties

Properties	Quantities
Tensile Strength	530,000 psi
Tensile Modulus	33,500,000 psi
Elongation	1.4%
Weight per Square Yard	12 oz.
Thickness	0.0048 in.

Table 3.8 Cured Laminate Properties of CFRP

Property	**Value**
Tensile Strength	90,000 psi
Modulus Of Elasticity	10.6×10^6 psi
Elongation At Break	1.2%
Thickness	0.020 in.
Strength per inch width	1,800 lbs/layer

Table 3.9 Composite Laminate Properties of Tyfo® WEB

Property	**Value**
Ultimate Tensile Strength	44,800 psi
Modulus Of Elasticity	2.8×10^6 psi
Elongation At Break	1.6%
Thickness	0.01 in.

Table 3.10 Properties of Tyfo® S Epoxy

Properties	**Value**
Tensile Strength	10,500 psi
Tensile Modulus	461,000 psi
Elongation	5.0 %
Tg	180F (typical)
Flexural Strength	11,500 psi
Flexural Modulus	400,000 psi

3.7 Materials for Under Water Repair

Two different systems – Air Logistics and Fyfe - were used for the underwater wrap used in the demonstration projects. The Air Logistics system is a pre-preg. All materials in this system were manufactured and provided by them. Details of the carbon fiber material used in the Air Logistics system are summarized in Table 3.11-3.12.

Table 3.11 Properties of Aquawrap® Fabrics [3.6]

Fibers	Tensile Strength (ksi)	Tensile Modulus (ksi)	Load per Ply (lb/in)
Uni-directional Glass Fiber	85	5,200	2,400
Bi-directional Glass Fiber	47	3,000	1,200
Uni-directional Carbon Fiber	120	11,000	3,400
Bi-directional Carbon Fiber	85	3,200	2,400

Table 3.12 Properties of Aquawrap® Base Primer #4 [3.6]

Properties	Quantities
Compressive Strength	10 ksi
Tensile Strength	4.8 ksi
Elongation at Break	40%
Flexural Strength	6.6 ksi
Shore Hardness	91

For the Fyfe wrap, only fiberglass was used. Tyfo® SEH-51A, a custom weave, uni-directional glass fabric is normally used with Tyfo-S Epoxy. However, for the underwater application in Gandy Bridge, Tyfo® SW-1 underwater epoxy was used. As this is not a pre-preg, it has to be mixed at the site and the FRP fabric impregnated just prior to use. Properties of materials as provided by the manufacturer are summarized in Table 3.13 – 14 [3.5].

Table 3.13 Properties of Tyfo® SEH-51 Composite [3.5]

Properties	**Quantities**
Tensile Strength	3.3 k/in
Tensile Modulus	3030 ksi
Ultimate Elongation	2.2 %
Laminate Thickness	0.05 in
Dry fiber weight per sq. yd.	27 oz.
Dry fiber thickness	0.014 in.

Table 3.14 Properties of Tyfo® SW-1 Epoxy [3.5]

Properties	**Quantities**
Mixing ratio, by wt	100:56
Specific Gravity	1.6
Viscosity A&B mixed, cps	14,000-18,000
Gel Time, 65F, hours	2.5-3.5
7 day compressive strength	7000-8000 psi

Tyfo® PUWECC manufactured by FYFE Co. LLC. Tyfo® PUWECC is a cement-based patching material designed to be worked in water. This was used to patch the lost section of the severely corroded pile in Gandy Bridge (# 100300) prior to the application of FRP wrap. Material properties are summarized in Table 3.15.

Table 3.15 Properties of Tyfo® PUWECC [3.5]

Properties	**Value**
Compressive Strength (ASTM C-109)	1 days: 3,500 psi 7 days: 5,460 psi 28 days: 6,015 psi
Set Time (ASTM C-266)	Initial: 3 –5 min Final: 20 min
Tensile Strength (ASTM C-190)	7 days: 315 psi 28 days: 410 psi
Flexural Strength (ASTM C-78)	28 days: 964 psi
Shear Bond Strength (ASTM C-1042)	1 day: 921 psi 7 days: 1,268 psi
Shrinkage & Expansion (ASTM C-157)	7 days: –0.053% 28 days: –0.160% 7 days: 0.047% 28 days: 0.104%
Chlorides (ASTM D-1411)	< 0.01%

3.8 Materials for Surface Preparation Study

Properties of Amercoat® 385 used for the concrete sealing is summarized in Table 3.16. Sika MonoTop® 611 used for patching material and Sika Armatec® 110 EpoCem used as a corrosion inhibitor were manufactured by Sika Corporation. Their properties are summarized in Table 3.17 and 3.18.

Table 3.16 Properties of Amercoat® 385

Tests	**Results**
Abrasion (ASTM D4060) 1 kg load/1000 cycles CS-17 wheel	108 mg weight loss
Adhesion, Elcometer (ASTM D4541)	> 1000 psi
Salt Spray – 1coat at 6mils 5000 hours exposure Face corrosion (ASTM B117) Face blistering (ASTM B117)	 None None
Humidity (condensation, ASTM D4585) Face corrosion at 3000 hours exposure	 None

Table 3.17 Properties of Sika Armatec® 110 EpoCem [3.7]

Properties	Value
Compressive Strength (ASTM C-109)	3 days 4500 psi 7days 6500 psi 28 days 8500 psi
Flexural Strength (ASTM C-348)	28 days 1250 psi
Splitting Tensile Strength (ASTM C-496)	28 days 600 psi
Water Permeability at 10 bar (145psi)	8.92 x 10^{-15} ft/sec
Water Vapor Diffusion Coefficient µ H_2O	110
Carbon Dioxide Diffusion Coefficient µ CO_2	14000
Bond of Steel Reinforcement to Concrete (pull out test)	625 psi
14 days moist cure, plastic concrete to hardened concrete (ASTM C882)	
Wet on Wet	2800 psi
24 hr. open time	2600 psi

Table 3.18 Properties of Sika MonoTop® 611 [3.7]

Properties	Value
Flexural Strength (ASTM C-293)	28 days 720 psi
Splitting Tensile Strength (ASTM C-496)	28 days 500 psi
Bond Strength (ASTM C-882 modified)	28 days 2200 psi
Compressive Strength (ASTM C-109)	1 day 3000 psi 7 days 5500 psi 28 days 6500 psi
Chloride Ion Permeability (AASHTO T-277)	< 600 coloumbs

Sikadur® 32, Hi-Mod epoxy (Table 3.19) was used in the Allen Creek Bridge (#150036) to attach the dollies to the FRP in the initial bond tests conducted on the witness panels. In the second test on the same bridge and the only test on the Gandy (# 100300) Bridge, Power-Fast+ epoxy was used (Table 3. 20).

Table 3.19 Properties of Sikadur® 32, Hi-Mod [3.7]

Properties	**Value**
Flexural Strength (ASTM D-790)	14 days 7000 psi
Tensile Strength	6900 psi
Elongation at Break	1.9 %
Modulus of Elasticity	540 ksi
Shear Strength	6200 psi
Bonding Strength (2 days, moisture) Plastic concrete to hardened concrete Plastic concrete to Steel Hardened Concrete to Hardened Concrete	 1700 psi 2000 psi 1900 psi
Bonding Strength (14 days, moisture) Plastic concrete to hardened concrete Plastic concrete to Steel Hardened Concrete to Hardened Concrete	 2200 psi 2000 psi 2000 psi

Table 3.20 Properties of Power-Fast+ Epoxy [3.8]

Properties	Quantities
Set Life	2 years for components
Color	Component A – White Component B – Dark Gray Mixed Epoxy – Uniform Gray
Mixed Ratio	1:1 by volume
Compressive Strength (ASTM D 695)	11,125 psi (1 day) 14,740 psi (7 days)
Tensile Strength (ASTM D 638)	7,250 psi (Fast Set) 7,400 (Standard Set)
Flexural Strength (ASTM D 790)	6,200 psi (Fast Set) 6,700 psi (Standard Set)
Bond Strength (Dry Cure)	3,000 psi
Water Absorption (ASTM D 570)	Less than 1% (0.59%)

References

3.1 Florida Rock Industries, Inc, Tampa, FL, Class V Special (07-0002).

3.2 Wire Products Inc., Longwood, FL.

3.3 Strand-Tech Martin, Summerville, SC.

3.4 Application Instruction Note sent from SDR Engineering (Apr. 15, 2002)

3.5 Fyfo Co. LLC, http://www.fyfeco.com/

3.6 Air Logistics Corporation. Aquawrap Repair System, Pasadena, CA

3.7 Sika Corporation, www.sika.com

3.8 Powers Fasteners, Inc., www.powers.com

4. FABRICATION OF SPECIMENS

4.1 Introduction

The pile specimens used in the study were fabricated at a commercial prestressing facility, Henderson Prestress, in Tarpon Springs about 30 miles from the USF campus. This chapter provides an outline of some of the considerations that went into the fabrication of the specimens. Details relating to the facilities utilized in casting the specimens are contained in Section 4.2. The forming operations are described in Section 4.3. The placement of the concrete is presented in Section 4.4. An assessment of the initial prestressing force is included in Section 4.5.

4.2 Prestressing Facilities

Mr. Dirk Henderson, president of Henderson Prestress, permitted USF to cast the test specimens on an existing bed used for double-T sections. These steel forms provided an excellent base for welding six inch angle irons which were used as forms. Reinforced abutments located at each end of the bed were well suited for our stressing operations. In addition this bed offered a safe work platform to facilitate the fabrication of our specimens (Fig. 4.1).

The available facilities permitted all the test specimens to be cast in a single pour. However, this would be make it exceedingly difficult to instrument and wrap the large number of specimens after 28 days. In view of this, the specimens were cast in two pours. In the first pour completed on February 26 2002, a total of 37 specimens - twenty six ft specimens and seventeen 5 ft - were cast. In the second pour, completed on March 12, the remaining 41 specimens - all 5 ft were cast.

Figure 4.1 Prestressing Bed.

4.3 Customizing Bed

The form for the test specimens was fabricated over the three foot wide flat region of the double-T bed. A single line was formed by using two sets of 4 in. x 6 in. steel angles. The correct width was maintained by welding headers at intervals corresponding to the different member lengths for the two pours. The basic steps involved in the operation were (1) weld first angle (2) position headers (3) weld second angle (4) clamp header in position and tack weld it at the top (See Fig. 4.2).

Welded Head

Bed Ready for Stressing

Figure 4.2 Customizing Bed.

4.4 Fabricating Specimens

As the edges of the specimens would need to be ground during wrapping, wood trims having a radius of ½ in were inserted at the two bottom corners of the form. Spiral ties were positioned between the headers and five steel strands pulled through the slots in the abutments and through the spirals (Fig. 4.3). The fifth strand located at the middle was intended to serve as the counter electrode for accelerated corrosion. After the four corner strands had all been pulled, load cells were placed at each end of each strand which was then secured to the abutments using a prestressing chuck (see Fig. 4.4**).** The strands were pre-tightened by using a prestressing jack and a hand actuated hydraulic pump (Fig. 4.5). The force placed on each strand was monitored using load cells. The target force in each strand was 11.5 kips.

The spiral ties were then stretched and spaced as required. They were secured in place using reinforcing bar tie wire. Sheets of galvanized steel were then placed in the beds to delineate the simulated damage zone of each member and provide a barrier between the chloride-contaminated concrete and the regular concrete. Finally after the spirals had been secured and the steel sheets were in place, an oil-based releasing agent was applied to the formwork. Care was taken not to spray any of the strands or spirals so that there was no danger of de-bonding (Fig. 4.6).

Figure 4.3 Wood Trims and Strands Positioning.

Figure 4.4 Load Cells and Prestressing Strands at Abutment.

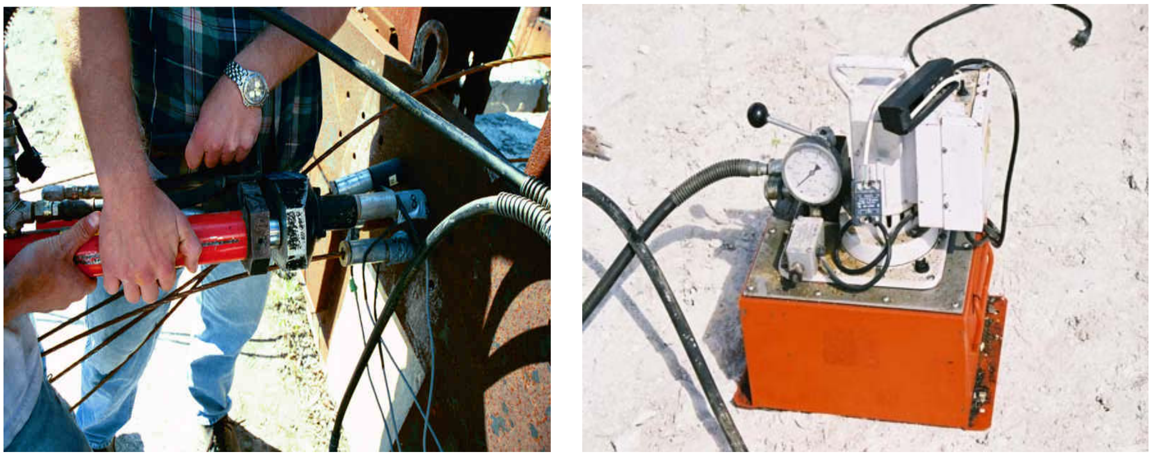


Figure 4.5 Prestressing Jack and Hydraulic Pump.

4.5 Placement of Concrete

As stated earlier, the fabrication of the all the test specimens required two separate pours. On each occasion, the regular FDOT Class V special mix was first placed followed by a second batch in which the chloride-contaminated FDOT Class V Special mix was installed in the 22 in. zone between galvanized barriers (Fig. 4.6). For each pour, a total of fifty, 6 in. x 12 in. cylinders were made. Twenty of these were made using normal concrete and thirty with the chloride contaminated concrete.

Stretched Spiral Ties with Barriers

Application of Releasing Agent

Figure 4.6 Finished Beds Ready for Concrete

The first pour was on February 26, 2002. The first truck arrived with the regular FDOT Class V Special concrete that was directly transferred into each of the two adjacent beds (Fig. 4.7). As the truck moved along the bed, concrete was vibrated to eliminate voids. The second concrete truck used for casting the chloride contaminated concrete arrived about an hour later.

Chloride contaminated concrete was made using Daraccel chloride admixture. This was stored in three plastic buckets and was directly added to the concrete inside the drum (Fig. 4.8). The drum was then rotated to mix the Daraccel and the concrete was placed from the truck into the designated areas. Because chlorides are accelerating agents, this concrete had to be placed in very short time to prevent it from setting. After the concrete had been placed, the galvanized steel sheets forming the barrier between the chloride contaminated region and the rest of the pile, were removed using pliers. The concrete was then vibrated. The exposed surface of the concrete was then finished using 3 in wide edging trowel so that the edges were curved. The entire bed covered by a plastic sheet to retain the moisture and help the concrete cure properly (Fig. 4.9). This entire procedure was repeated for the second pour completed on March 12, 2002.

Regular Concrete Pour

Daraccel Added Concrete Pour

Figure 4.7 Concrete Placement.

Figure 4.8 Daraccel Admixture.

Figure 4.9 Surface Finishing and Curing.

4.6 Marking and Removal of Specimens

Specimens were labeled inside the bed two to three days following each pour to allow identification of its type, pour date and location within the bed. Color coding was used to differentiate the two pours. Red and Blue colors identified the fist and the second pour, respectively. All specimens were numbered consecutively from 1-37 for the first pour (starting from the live end) and from 38-78 on the second day.

The prestressing force was released after sufficient time had been allowed for the concrete to gain strength. For the first pour, the force was released on March 4, six days after the pour. For the second pour, a somewhat greater interval was necessitated because of the need to set up experimental studies for specimens cast earlier. As a result, the force was released after 11 days on April 2. On each occasion, four cylinders – two regular and two chloride contaminated – were broken and compressive strength determined. The compressive strength was 3,700 psi for both types of concrete for the first pour. Compressive strengths were higher for the second because of the greater time and also warming trends. The average compressive strength for the regular concrete was about 6,050 psi and that for the chloride contaminated concrete, 4,975 psi.

The prestressing force was released by frame cutting the strands (Fig. 4.10). The weld connecting the steel angle to the bed was cut to free it from the specimens and the bed. The relatively small size of the specimens allowed them to be removed from the bed without the need for heavy lifting equipment. The specimens were returned to the University of South Florida campus during March 2002.

Figure 4.10 Cutting Specimens.

4.7 Prestressing Force Analysis

As mentioned earlier, eight load cells were used to measure the prestressing force in each of the four strands at both the dead and live ends. In addition, the elongation in each strand was monitored to provide a check on the load cell readings. Table 4.1 is a summary of the measured forces. Taking the initial prestress as 0.9 x the jacking force, to allow for relaxation and elastic shortening losses, the average initial prestress was 1,005 psi for specimens cast on February 26 and 1,060 psi for those cast on March 21.

Table 4.1 Jacking and Initial Prestress Forces

	Load Cell	**Day 1**		**Day 2**	
		P_j (lbs)	**P_i (lbs)**	**P_j (lbs)**	**P_i (lbs)**
Live End (East)	TSE	9,400	8,460	10,468	9,421
	BSE	11,900	10,710	12,241	11,017
	TNE	11,200	10,080	10,505	9,455
	BNE	9,600	8,640	10,544	9,490
Dead End (West)	TSW	8,720	7,848	11,203	10,083
	BSW	7,330*	6,597	10,346	9,311
	TNW	9,660	8,694	10,680	9,612
	BNW	9,900	8,910	10,555	9,500

* Load cells did not function properly

Table 4.2 Summary of Average Force and Prestress

	Day 1		**Day 2**	
	P_j	**P_i**	**P_j**	**P_i**
Average Force (lbs)	10,054	9,049	10,614	9,552
Average Prestress (psi)	1,117	1,005	1,179	1,061

5. EXPANSION STUDY

5.1 Introduction

This chapter provides details of an experimental study conducted to measure transverse strain developed due to corrosion of the embedded prestressed steel. A complete description of the experimental program is summarized in Section 5.2. Instrumentation and data acquisition system used for measuring strain data is presented in Section 5.3 and test results including crack survey, steel loss, strain changes are described in Section 5.4. Section 5.5 lists the main findings from the study.

5.2 Test Program

Fiber reinforced polymers are costly materials and economical FRP repairs require them to be engineered so that they can withstand expansion caused by corrosion and also provide strengthening to compensate for the metal loss. This study was directed towards measurement of expansion due to corrosion of steel.

A total of eight 5 ft specimens were utilized in the study. Four of these were instrumented to measure strains while the remaining four were used for gravimetric testing to allow correlation of strain to metal loss. Of the four instrumented specimens, two were instrumented using strain wire and two others by a combination of strain wires and crack gages. Details of the test program are summarized in Table 5.1. Specimen details are shown in Chapter 2. Information on materials used and their fabrication is included in Chapters 3 and 4 respectively.

Specimens were corroded using a constant current accelerated setup similar to that used earlier [1] excepting for two changes. First, no external cathode was required since specimens had been cast with an additional internal central strand that served as a cathode (Fig. 2.1). Second, the "sponge-egg" crate assembly for keeping the chloride contaminated region moist was installed (Fig. 5.1) but not used. Instead, a "dry" system was used because it did not compromise the integrity of the instrumentation. This meant higher voltages since concrete resistivity was higher. Moreover, cracking occurred away from the chloride contaminated zone following extended exposure. As before, 110 mA of impressed current was incrementally reached over 6 days. The applied current and voltage were manually monitored. The current dropped at the end of the study (Fig. 5.2) due to a system problem with the circuit. As a consequence, about 80 mA of impressed current was applied for 30 days. Fig. 5.3 shows the variation of voltage over 125 days. The voltage gradually increased with time. Comparing to other studies in this project (Fig. 7.2, Fig. 8.3), the voltage increase was about four times greater.

Table 5.1 Specimen Details

Specimen Number	Type	Size (ft)	Instrumentation	Targeted Steel Loss	Test Method
#1	Gravimetric Control	5	-	10%	Gravimetric
#9	Gravimetric Control	5	-	15%	Gravimetric
#3	Gravimetric Control	5	-	20%	Gravimetric
#4	Gravimetric Control	5	-	25%	Gravimetric
#5	Instrumented	5	3 strain wire	30%	Gravimetric
#6	Instrumented	5	3 strain wire	30%	Gravimetric
#7	Instrumented	5	2 strain wire 24 crack gages	30%	Gravimetric
#8	Instrumented	5	2 strain wire 24 crack gages	30%	Gravimetric

Figure 5.1 "Dry" Accelerated Corrosion Set Up.

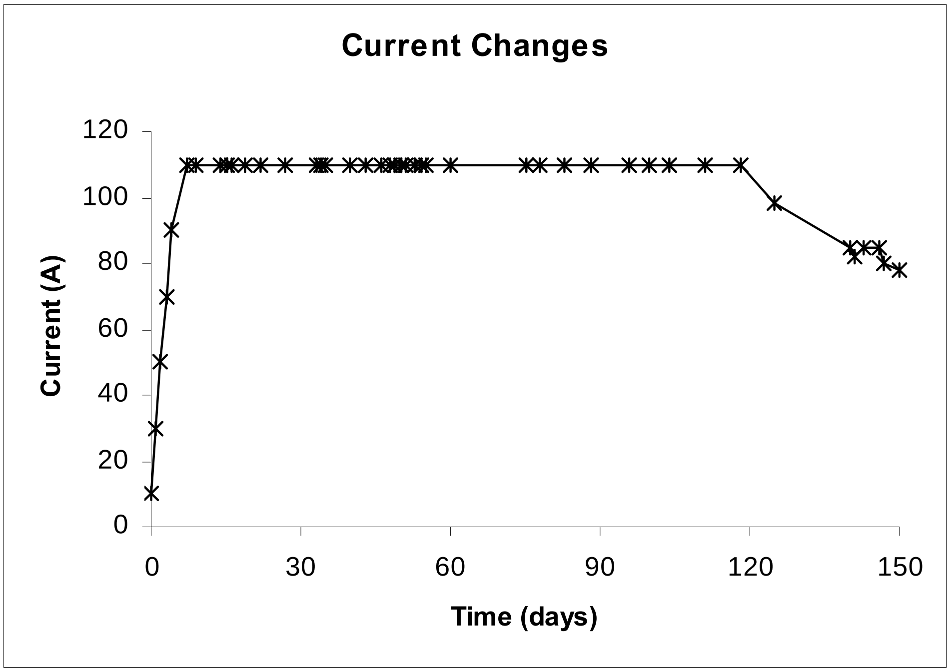


Figure 5.2 Applied Current vs Time

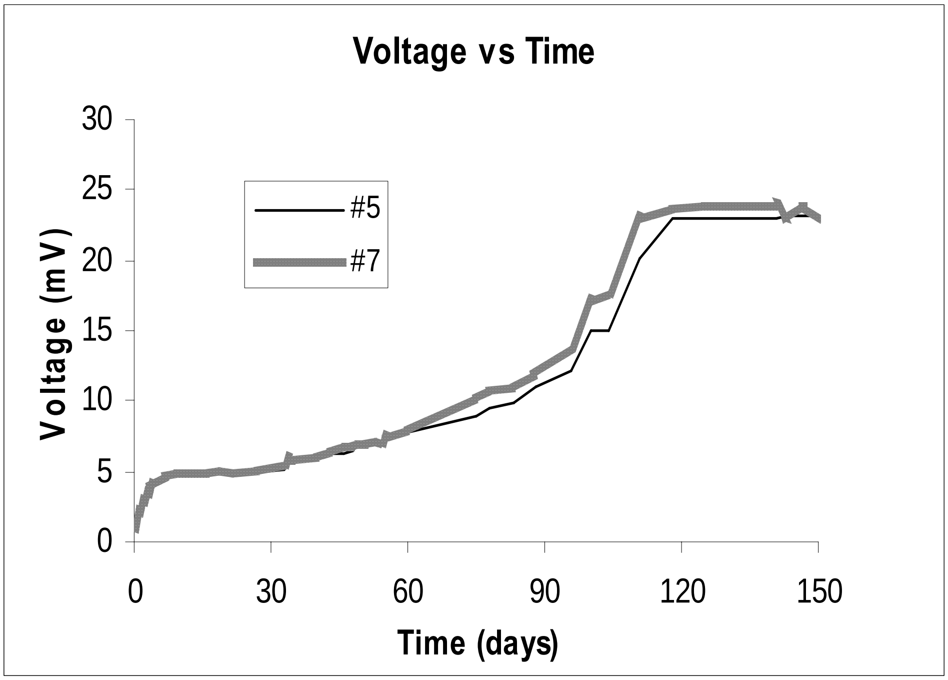
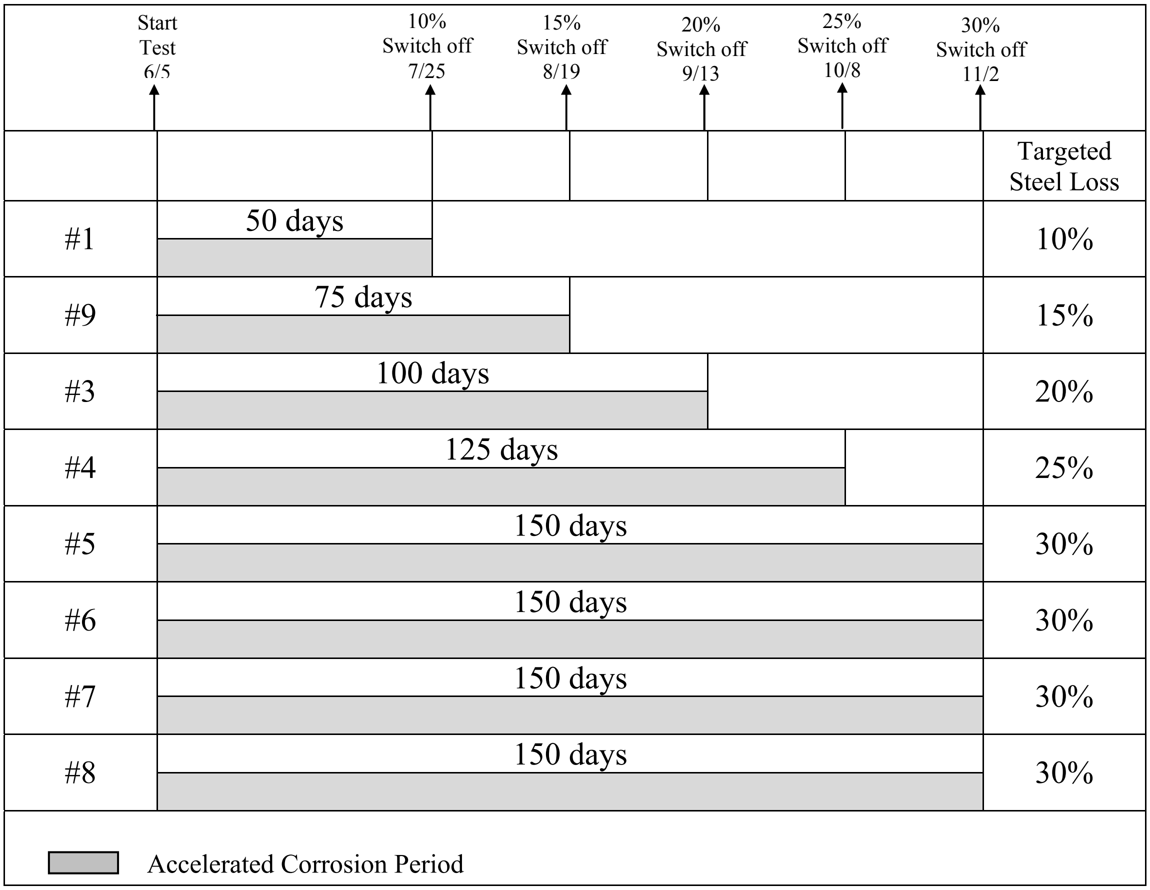


Figure 5.3 Internal Voltage vs Time

In a previous study [1] it was found that 50 days exposure to the constant current accelerated regime resulted in a 10% metal loss. Despite the changes to the set up, similar corrosion loss was also expected in this study since the current remained the same. As a result, specimens earmarked for gravimetric testing were removed after 50 days (#1 targeted loss 10%), 75 days (#9 targeted loss 15%), 100 days (#3 targeted loss 20%) and 125 days (#4 targeted loss 25%). In the gravimetric testing, strands and ties were retrieved from the corroded concrete specimen and the actual weight loss measured. Following completion of study, the gravimetric testing was conducted on the four instrumented specimens (#5-#8). The results from these tests are presented in Section 5.4.3. Table 5.2 summarizes the timeline for the study.

Table 5.2 Time Line of Study



		Targeted Steel Loss
#1	50 days	10%
#9	75 days	15%
#3	100 days	20%
#4	125 days	25%
#5	150 days	30%
#6	150 days	30%
#7	150 days	30%
#8	150 days	30%

5.3 Instrumentation and Data Acquisition

As stated earlier, four specimens were instrumented to allow measurement of transverse expansion strain. The targeted metal loss for all the instrumented specimens was 30%. However, as gravimetric tests were conducted on identically corroded specimens after 10%, 15%, 20% and 25% metal loss, it allowed the relationship between metal loss and strain to be established.

Two specimens (#7, #8) were fully instrumented using 24 crack gages and 2 strain wires. Six crack gages were attached along the line of the strands (cracking was observed to occur in this manner in a previous study), on each face. These were symmetrically positioned at three levels (20 in, 30 in and 40 in from the top) so that they straddled the 22 in. chloride contaminated depth. For the two specimens, a total of 48 crack gages were used. In addition, two strain wires were placed 1 in. below the bottom crack gage and 1 in. above the top crack gage. Another two specimens (#5, #6) were instrumented only with strain wires. For these specimens, the three strain wires matched the location of the crack gages, i.e. they were positioned 20 in., 30 in. and 40in. from the top of each specimen. Figs. 5.4 and 5.5 show the installed crack gages and strain wires.

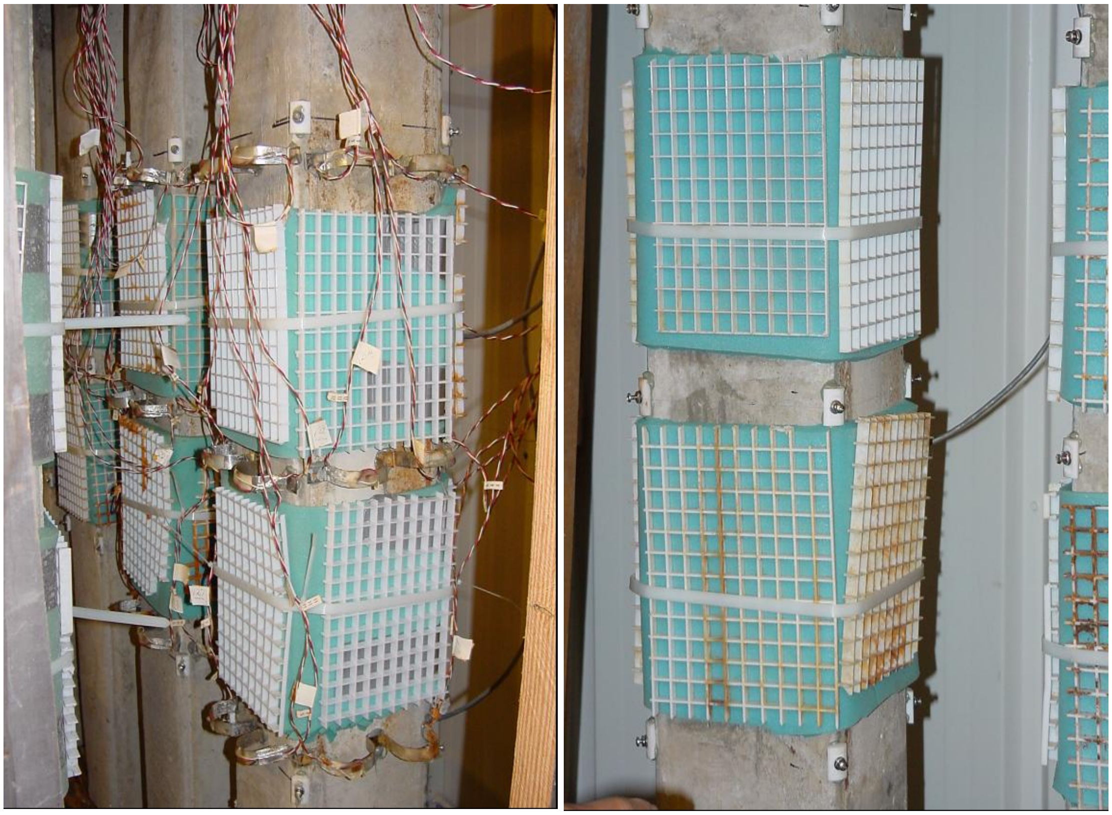

(a) (b)

Figure 5.4 (a) 24 Crack Gages and 2 Strain Wires at #7 and #8 Specimens; (b) Three Strain Wires at #5 and #6 Specimens

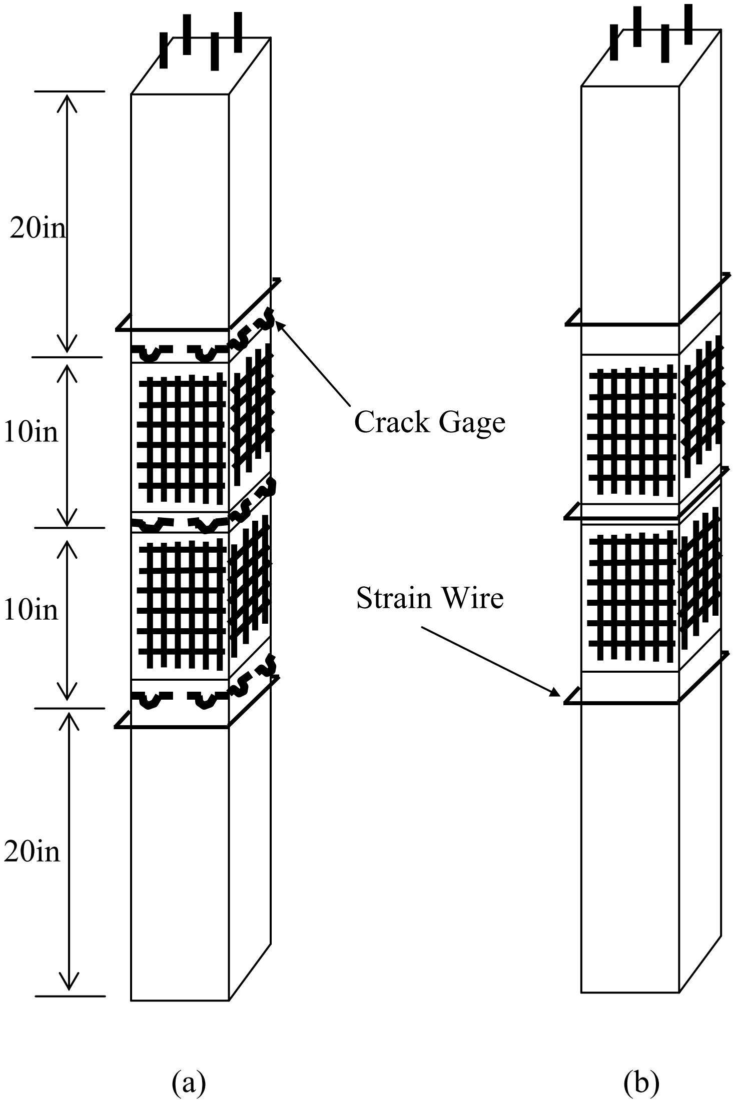
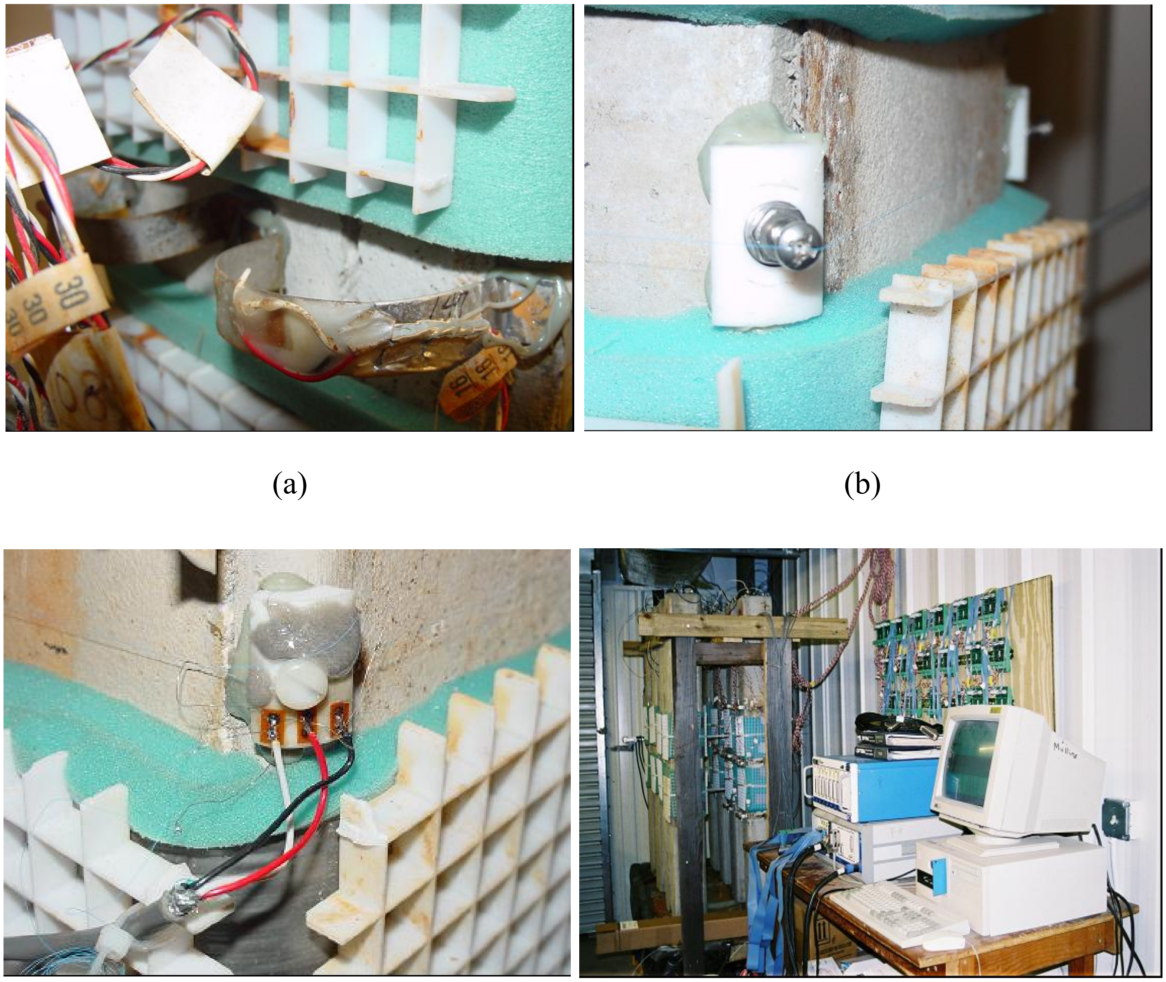


Figure 5.5 Schematic Drawing of (a)#7 and #8 Specimens; (b)#5 and #6 Specimens

Crack gages were made from stainless steel strips to which two strain gages were bonded. They were in half bridge configuration to allow for temperature compensation and were calibrated using a LVDT to measure a maximum movement of 2 mm. All crack gages were designed and fabricated at USF [5.1]. Details on the fabrication and calibration may be found elsewhere [5.2].

TP44-0 TFE ENAMEL BLUE wire manufactured by Pelican Wire Co., Inc. was used for the strain wire. Twenty-six inch length of strain wire was wound around a pile and placed on four bolts bonded at the four corners of specimen (Fig. 5.6).

Crack gages and strain wires were hooked to a MEGADAC data acquisition system. Strain changes in strain wires due to the lateral expansion of corroded specimen were directly monitored by a computer hooked to a MEGADAC system. However, crack gages bonded to the concrete surface measured the displacement between two legs in mm. The gage length of the crack gages varied between 40 to 50mm. The ratio of the measured displacement over the gage length provided the strain value.



(a) (b)

(c) (d)

Figure 5.6 (a) Installed Crack Gage; (b) Installed Strain Wire; (c) Strain Wire Connection; (d) MEGADAC Data Acquisition System

5.4 Test Results

Detailed inspection was carried out on all specimens upon attainment of the targeted steel loss levels of 10%, 15%, 20%, 25% and 30%. This included crack surveys (Section 5.4.1) and gravimetric tests (Section 5.4.2).

5.4.1 Crack Survey

Crack surveys were performed on all eight specimens using a magnifying glass and a crack comparator. Cracks were first traced on a transparent plastic sheet (Fig. 5.7) and later transferred to paper. To allow identification of the four faces of the specimen they are referred to with respect to their relative position in the casting bed. The top side where the specimen number is marked is referred to as “A”. The remaining three faces are referred to as “B”, “C” or “D”, clockwise with respect to A, i.e. B is to the right of A, C was the bottom face in the casting bed and D is to the left of A.

Figs. 5.8-5.12 show the observed crack pattern. They are plotted on a 2 in. grid. The 22 in. chloride contaminated region extends from grid 9.5 to grid 21.5. No cracking developed on face A before a 20% metal loss was reached. While the extent of the cracking is largely limited to the chloride contaminated for smaller steel losses (Fig. 5.8, 5.9) it extends over the entire section at higher losses (Fig. 5.10-5.12) when transverse cracks occur. Much of the cracking is concentrated on sides B and D. Information on the cracks in each specimen is summarized in Tables 5.3-5.7.

(a) (b)

Figure 5.7 (a) Conducting Crack Survey; (b) Cracked Specimens

10% of Targeted Steel Loss

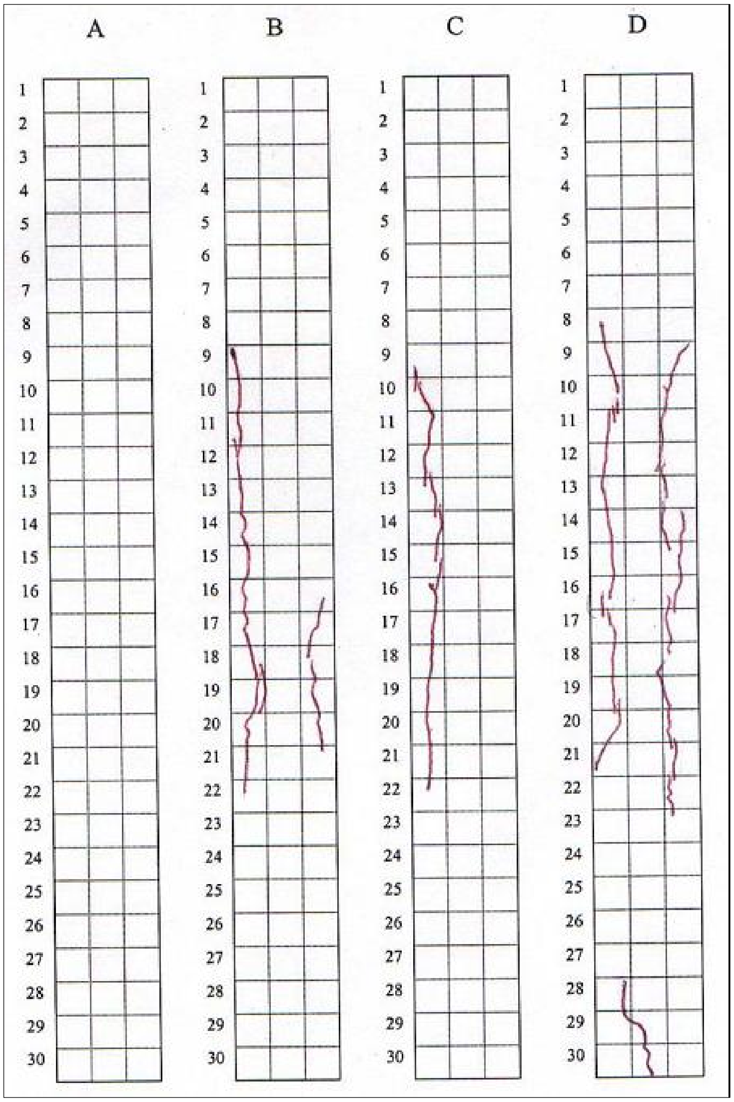


Figure 5.8 Crack Pattern of #1 at 10% of Targeted Steel Loss

Table 5.3 Crack Information at 10% of Targeted Steel Loss

	Number	**Maximum Length (in.)**	**Maximum Width (mm)**
# 1	32	28.5	0.8

15% of Targeted Steel Loss

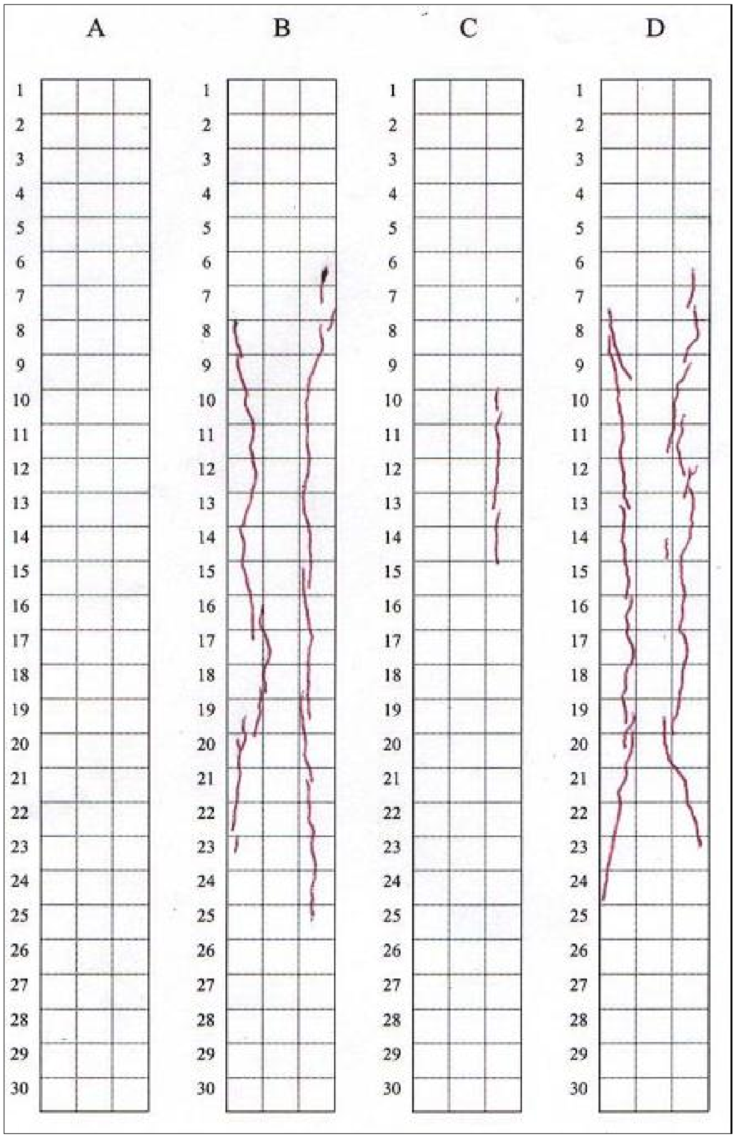


Figure 5.9 Crack Pattern of #9 at 15% of Targeted Steel Loss

Table 5.4 Crack Information at 15% of Targeted Steel Loss

	Number	**Maximum Length (in.)**	**Maximum Width (mm)**
# 9	30	38.5	1.5

20% of Targeted Steel Loss

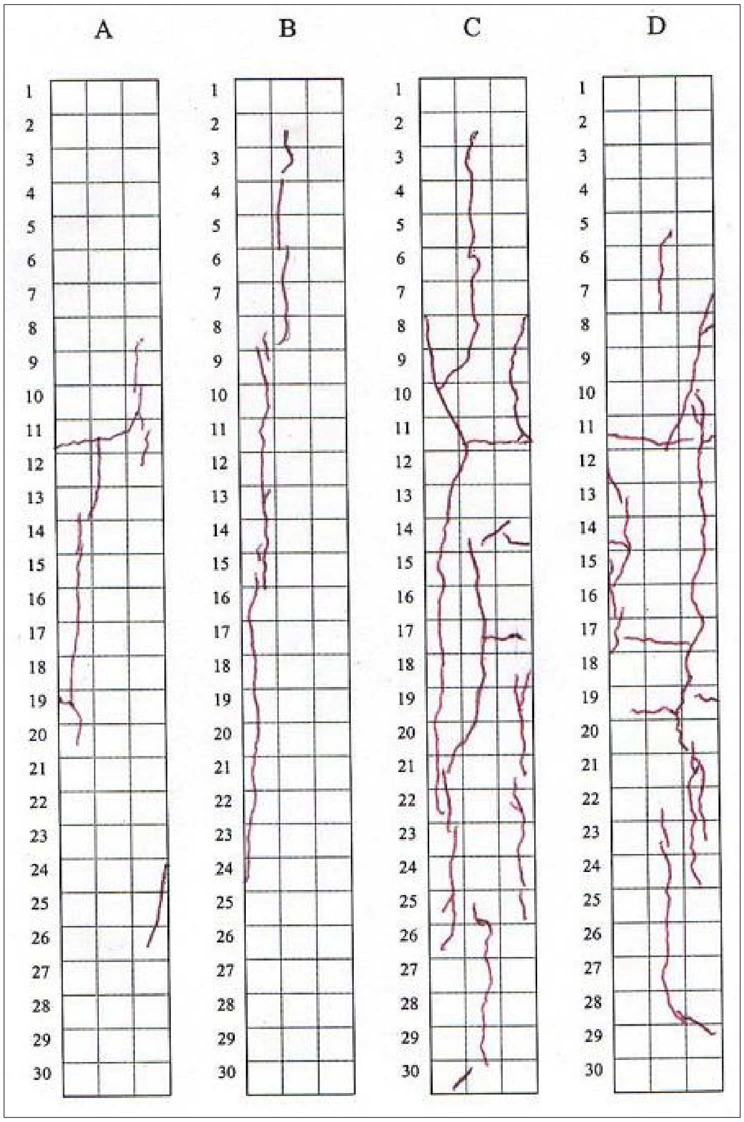


Figure 5.10 Crack Pattern of #3 at 20% of Targeted Steel Loss

Table 5.5 Crack Information at 20% of Targeted Steel Loss

	Number	**Maximum Length (in.)**	**Maximum Width (mm)**
#3	58	48	3

25% of Targeted Steel Loss

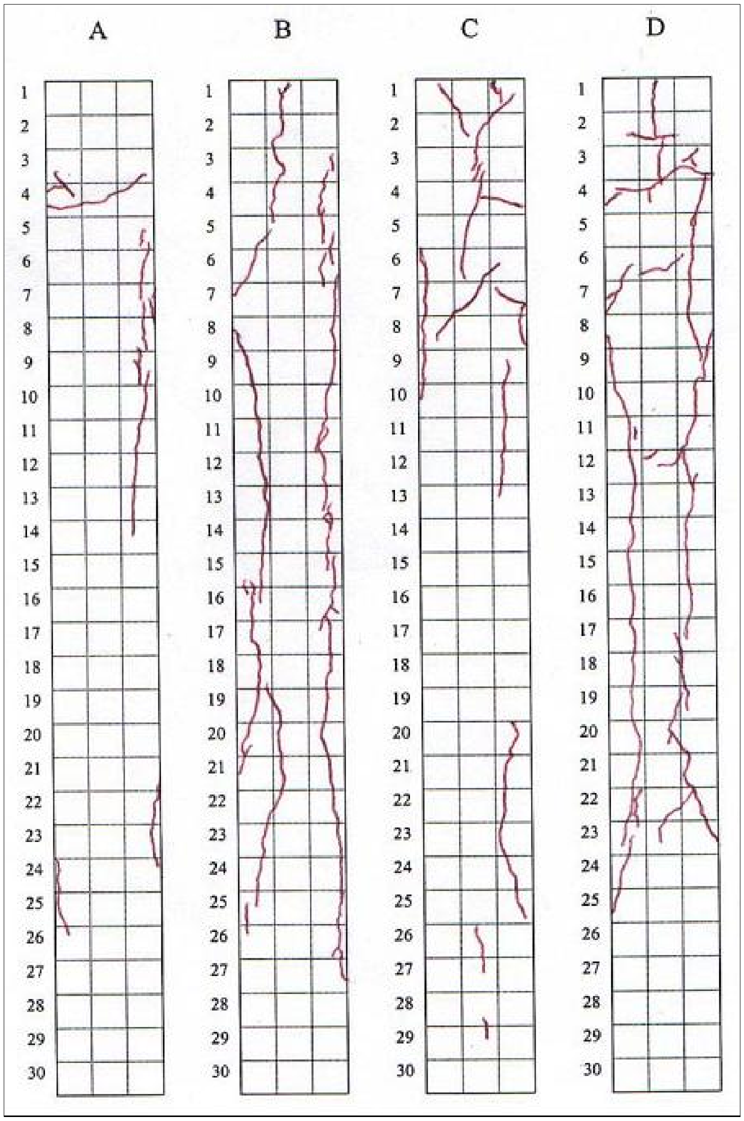
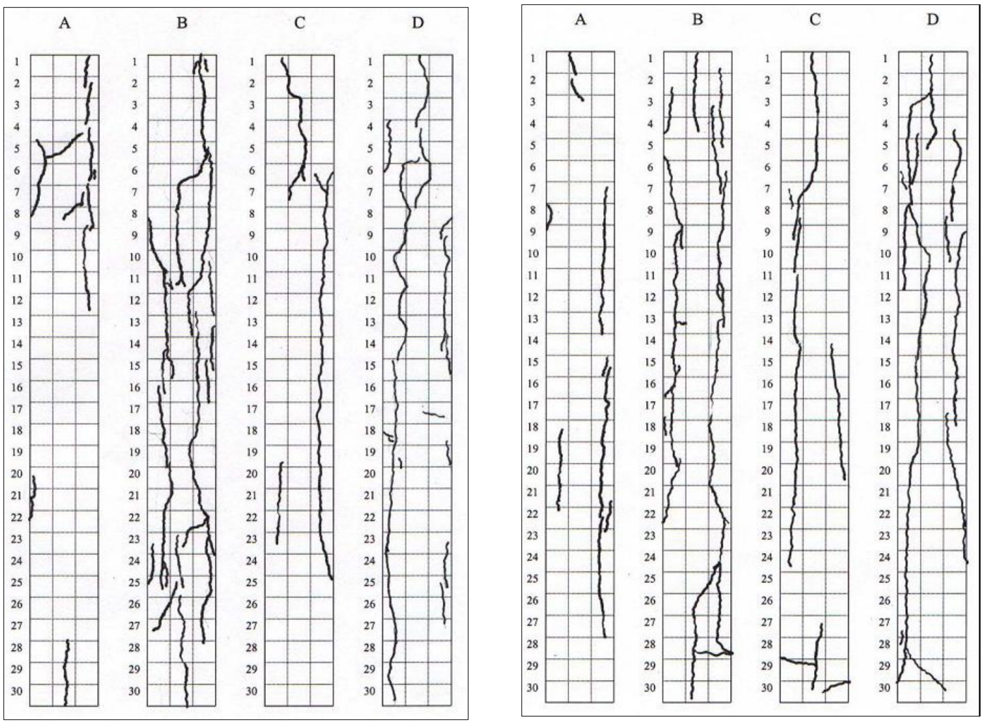


Figure 5.11 Crack Pattern of #4 at 25% of Targeted Steel Loss

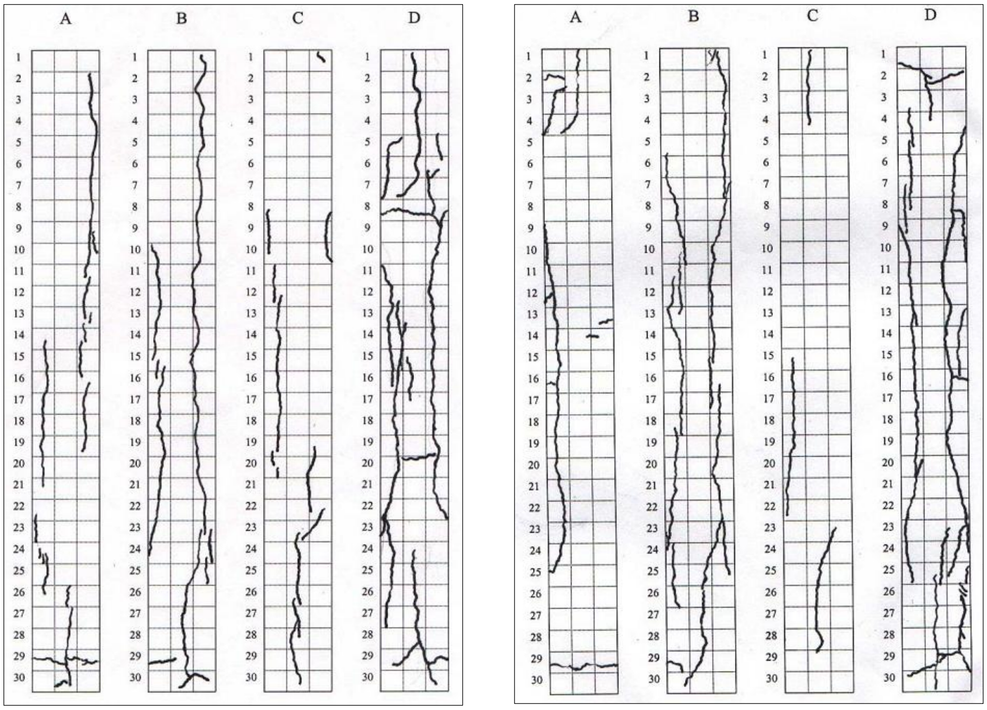
Table 5.6 Crack Information at 25% of Targeted Steel Loss

	Number	**Maximum Length (in.)**	**Maximum Width (mm)**
#4	77	50	2.1

30% of Targeted Steel Loss



(a) #5 specimen (b) #6 Specimen



(c) #7 specimen (d) #8 specimen

Figure 5.12 Crack Pattern of at 30% of Targeted Steel Loss

Table 5.7 Crack Information at 30% of Targeted Steel Loss

	Number	Maximum Length (in.)	Maximum Width (mm)
#5	57	60	3.5
#6	50	58	4
#7	57	59	2.5
#8	53	59	6
Average	54.3	59	4

5.4.2 Steel Loss

Following the crack survey, gravimetric testing was carried out on every specimen to verify the actual steel loss. For convenience, each strand was labeled according to the faces adjacent to it. For example, a strand positioned near face A and B is referred to as 'strand AB' (see Section 5.4.1 for definition of face).

After removing the surface concrete, four 22 in. length of strand and 5 turns of ties in the chloride-contaminated region were retrieved (Fig. 5.13). Retrieved strands and ties were cleaned mechanically and chemically following the cleaning procedures recommended by ASTM G1-90 [5.3]. The difference between the original and final weight after cleaning gave the metal loss.

The actual steel loss in the eight specimens are summarized in Table 5.8. Average losses were 12.85%, 15.66%, 19.54%, 20.65% and 25.72% [average value for specimens #5-#8] at 50 days, 75 days, 100 days, 125 days, and 150 days, respectively. A plot of the targeted vs actual loss appears in Fig. 5.14. A straight line shows the targeted steel loss. The actual steel loss was higher than the target steel loss before 100 days but became lower after 100 days because of a drop in the impressed current (Fig. 5.2). There is no pattern in the relative loss in the ties compared to strands. The latter could be expected to be larger because they are located closer to the central cathode; however this is only valid for specimens #5-#8.


(a) (b)

Figure 5.13 (a) Cutting Surface Concrete; (b) Retrieved Strands

Table 5.8 Actual Mass Loss of All Specimens

		#1		**#9**		**#3**		**#4**	
	Initial Weight (g)	**Final Weight (g)**	**Mass Loss (%)**	**Final Weight (g)**	**Mass Loss (%)**	**Final Weight (g)**	**Mass Loss (%)**	**Final Weight (g)**	**Mass Loss (%)**
AB	168.78	147.30	12.73	144.50	14.39	135.10	19.96	136.70	19.01
BC	168.78	148.20	12.19	143.30	15.10	140.30	16.88	136.50	19.13
CD	168.78	149.80	11.25	144.20	14.56	136.80	18.95	138.70	17.82
DA	168.78	147.30	12.73	143.00	15.28	132.60	21.44	131.90	21.85
strands	675.13	592.60	**12.22**	575.00	**14.83**	544.80	**19.30**	543.80	**19.45**
tie	332.55	285.60	**14.12**	274.90	**17.34**	266.00	**20.01**	255.80	**23.08**
total	1007.68	878.20	**12.85**	849.90	**15.66**	810.80	**19.54**	799.60	**20.65**
		#5		**#6**		**#7**		**#8**	
	Initial Weight (g)	**Final Weight (g)**	**Mass Loss (%)**	**Final Weight (g)**	**Mass Loss (%)**	**Final Weight (g)**	**Mass Loss (%)**	**Final Weight (g)**	**Mass Loss (%)**
AB	168.78	129.50	23.27	121.70	27.90	126.90	24.81	117.00	30.68
BC	168.78	125.30	25.76	121.90	27.78	130.80	22.50	127.20	24.64
CD	168.78	124.60	26.18	126.40	25.11	124.10	26.47	124.30	26.35
DA	168.78	116.00	31.27	126.30	25.17	117.40	30.44	114.20	32.34
strands	675.13	495.40	**26.62**	496.30	**26.49**	499.20	**26.06**	482.70	**28.50**
tie	332.55	249.70	**24.91**	252.50	**24.07**	261.60	**21.34**	256.80	**22.78**
total	1007.68	745.10	**26.06**	748.80	**25.69**	760.80	**24.50**	739.50	**26.61**

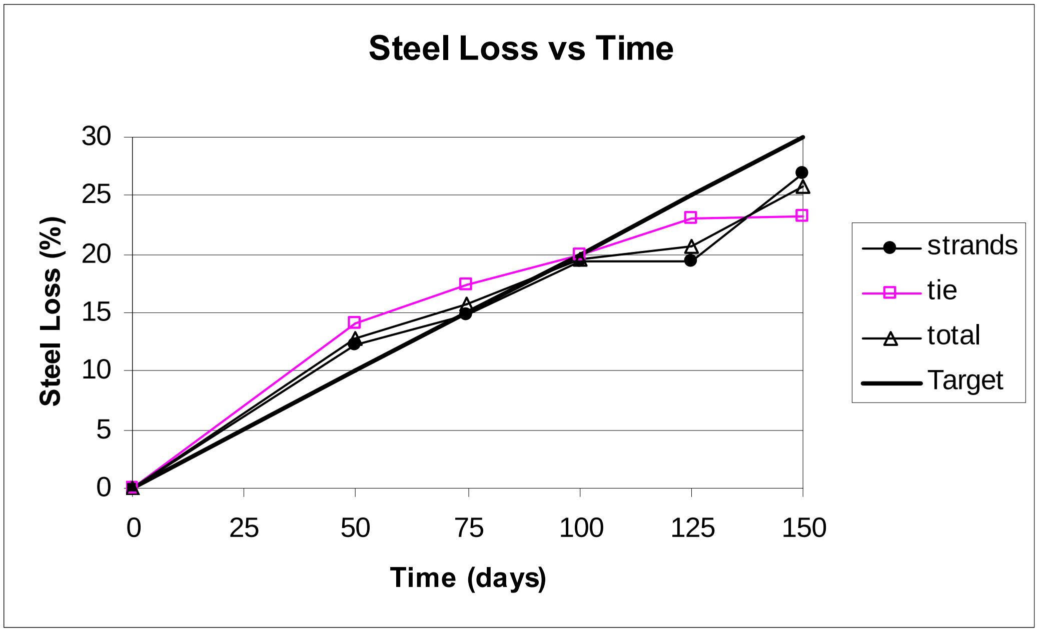


Figure 5.14 Actual Steel Loss and Targeted Steel Loss

5.4.3 Strain Change

5.4.3.1 Strain from Crack Gages

Two specimens (#7, #8) were each instrumented by 24 crack gages, six per face attached along the centerline of four strands at 20 in, 30 in, and 40 in from the end (Fig. 5.5). Both tensile and compressive (negative) movements were recorded Tensile movement was developed in the cracked region and compressive movement in the uncracked region.

Since transverse expansion is due to tensile stresses, negative values were neglected in estimating the transvers (lateral) strain. To calculate the lateral strain at each level, the sum of crack width values measured from the eight crack gages positioned at the same level was divided by the circumferential length (6 in x 4 faces x 25.4). Table 5.9 shows the calculated strain values from the crack width data at the three instrumented positions corresponding to 50 days, 75 days, 100 days, 125 days and 150 days. As expected, the maximum strains occurred at the middle (30 in). The average strains were higher at the bottom (40 in) than at the top (20 in). The average mid-height strains corresponding to12.85%, 15.66%, 19.54%, 20.65% and 25.72% metal loss were 4433με, 8460με, 12768με, 17310με, and 21331με respectively. The average strains over the 20 in. depth were 3712με, 7333με, 11121με, 14874 με and 17748με.

Table 5.9 Calculated Strain from Crack Gages (με)

	Position	**50 days (12,85%)**	**75 days (15.66%)**	**100 days (19.54%)**	**125 days (20.65%)**	**150 days (25.72%)**
#7	Top (20in)	2663	4607	6238	8075	9710
	Middle (30in)	4474	8589	12255	16111	19653
	Bottom (40in)	3703	7976	11693	15433	18831
#8	Top (20in)	3731	7642	12282	15928	16436
	Middle (30in)	4391	8331	13282	18509	23010
	Bottom (40in)	3310	6851	10975	15190	18850
Average	Top (20in)	3197	6124	9260	12002	13073
	Middle (30in)	4433	8460	12768	17310	21331
	Bottom (40in)	3507	7413	11334	15311	18840
Average		**3712**	**7333**	**11121**	**14874**	**17748**

5.4.3.2 Strain from Strain Wires

Two specimens (#5, #6) were each instrumented by three strain wires (20in, 30in, 40in) and another two (#7, #8) were instrumented by two strain wires (20in, 40in) (Fig. 5.5). The strain change due to corrosion expansion of the specimen was directly saved in a computer.

Strain values obtained from strain wire measurements are summarized in Table 5.10. The average mid-height strains corresponding to 12.85%, 15.66%, 19.54%, 20.65% and 25.72% metal loss were respectively 6273με, 10611με, 14278με, 18074με, and 25357με. The corresponding average strain over the 20 in. depth was respectively 4072με, 7605με, 11177με, 14374 με and 19645με.

Fig. 5.15 compares the average strain values from crack gages and strain wires for different steel loss values. The values are near identical.

Table 5.10 Measured Strain by Strain Wires (με)

	Position	**50 days (12.85%)**	**75 days (15.66%)**	**100 days (19.54%)**	**125 days (20.65%)**	**150 days (25.72%)**
#5	Top (20in)	1848	6157	11373	13382	18960
	Middle (30in)	6403	12540	18081	25080	34757
	Bottom (40in)	3913	5549	8223	11839	11839
#6	Top (20in)	3891	7280	9896	13549	18791
	Middle (30in)	6144	8681	10476	11069	15956
	Bottom (40in)	2075	5461	10988	12777	14624
#7	Top (20in)	3718	8119	11257	12174	17752
	Middle (30in)	NA	NA	NA	NA	NA
	Bottom (40in)	NA	NA	NA	NA	NA
#8	Top (20in)	2270	2686	6985	14335	18894
	Middle (30in)	NA	NA	NA	NA	NA
	Bottom (40in)	3045	7418	8915	10442	18472
Average	Top (20in)	2932	6061	9878	13360	18599
	Middle (30in)	6273	10611	14278	18074	25357
	Bottom (40in)	3011	6142	9375	11686	14979
Average		**4072**	**7605**	**11177**	**14374**	**19645**

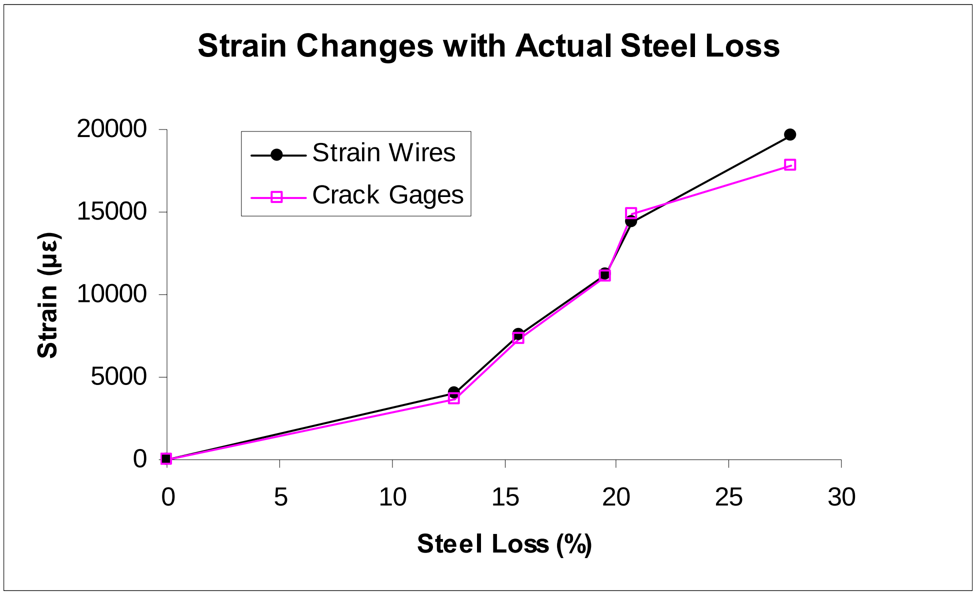


Figure 5.15 Crack Gages vs Strain Wire Data

5.5 Conclusions

The following conclusions may be drawn from the information presented in this chapter.

1. Actual steel loss values were reasonably close to targeted values as long as the impressed current was constant, e.g. 15.66% vs 15%, 19.54% vs 20%. However, when currents reduced (Fig. 5.2) results differed (see Table 5.8).

2. The readings from strain wire and crack gages were in very good agreement. Maximum strains occurred at the middle of the chloride contaminated region. The measured strain was 0.85% for 15.66% metal loss and 2.1% for 25.72% loss (Table 5.9). Since a "dry" system was used, these strains may be on the high side.

3. The "dry" constant current corrosion acceleration system worked well. However, it leads to increased voltage that compensates for the higher concrete resistivity.

References

5.1 Mullins, G., Sen, R, Torres-Acosta, Goulish, A., Suh, K., Pai, N. and Mehrani, A. (2001). "Load Capacity of Corroded Pile Bents". Final Report submitted to the Florida Department of Transportation, September.

5.2 Suh, KS (2002). "Effectiveness of CFRP in Reducing Corrosion Rate of Prestressed Members," MSCE Thesis submitted to Department of Civil and Environmental Engineering, University of South Florida, Tampa, FL , May.

5.3 ASTM G1-90 (1990), :Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens," American Society for Testing and Materials, Philadelphia, pp. 25-31.

6. EFFECTIVENESS OF FRP UNDER AMBIENT CONDITIONS

6.1 Introduction

This chapter presents details on the experimental study intended to evaluate the effectiveness of CFRP and GFRP wrap in reducing corrosion in chloride contaminated specimens under ambient conditions. The test program is summarized in Section 6.2. Instrumentation and data acquisition used for corrosion monitoring are discussed in Section 6.3. Specimen preparation and wrapping procedures are described in Section 6.4 and details on the exposure set up are presented in Section 6.5. Findings from the study are summarized in Section 6.6 with the conclusions contained in Section 6.7.

6.2 Test Program

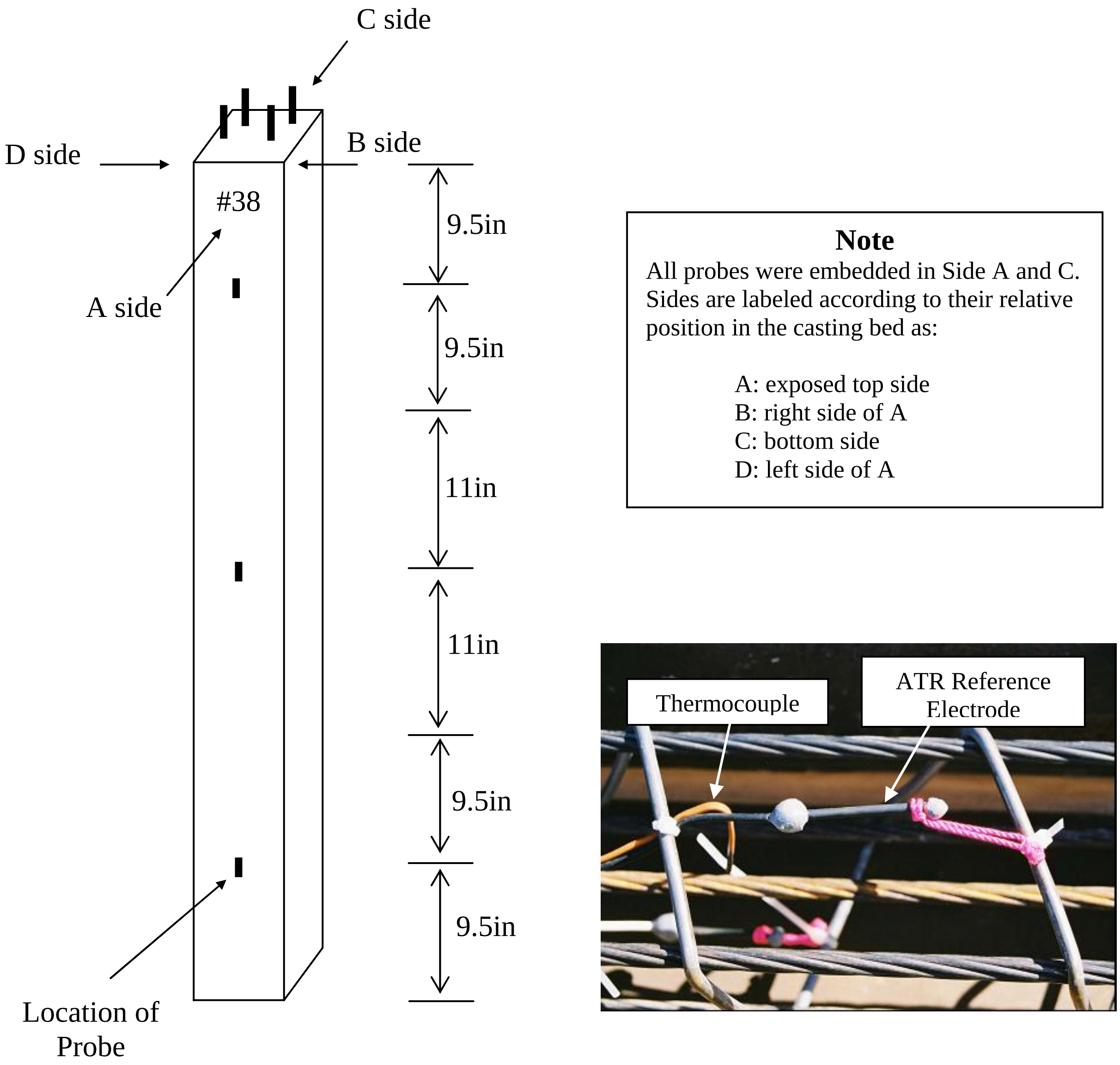
The primary goal of this study was to evaluate the effectiveness of FRP in slowing down the corrosion rate in chloride-contaminated concrete in a simulated marine environment under ambient conditions. Variables investigated include (1) fiber type (2) number of FRP layers (3) wet/dry exposure and (4) environment. Experimental parameters were selected to reflect Florida conditions and also to corroborate findings from a recently concluded study funded by Texas DOT [6.1].

A total of 22, five ft specimens were used in the study (Fig. 2.1). Sixteen of these were wrapped and remaining six unwrapped specimens were used as controls. A 22 in. length in the central region of all specimens had 3% chloride ion that was introduced during fabrication (see Chapter 4). Two different environments were investigated – an outdoor environment subjected to diurnal and seasonal fluctuations in temperature and humidity and a laboratory environment where specimens were under more uniform conditions. In both environments, specimens were exposed to wet/dry cycles in salt water.

Two different fiber types – carbon and glass – were evaluated. Consequently, half the specimens were wrapped using bi-directional CFRP and the other half using bi-directional GFRP. The number of FRP layers varied from 1 to 4. Details are summarized in Table 6.1. To allow corrosion performance to be monitored, each specimen was instrumented using reference electrodes and thermocouples. Reference electrodes allow measurement of the corrosion potential and the corrosion rate using linear polarization. Thermo-couples allow temperature inside the concrete to be measured. Activated titanium reference electrodes (ATR) were used; their number varied from 2 to 6 (see Fig. 6.1).

Table 6.1 Specimen Details

Specimen Number	Type	Number of Installed Probes	Wrap	Number of Wrap Layers
#38	Outdoor Control	6	No	0
#44	Outdoor Control	2	No	0
#45	Outdoor Control	2	No	0
#46	Outdoor Control	2	No	0
#39	Indoor Control	6	No	0
#49	Indoor Control	2	No	0
#54	CFRP wrap	2	Carbon	1
#55	CFRP wrap	2	Carbon	2
#56	CFRP wrap	2	Carbon	3
#57	CFRP wrap	2	Carbon	4
#58	CFRP wrap	2	Carbon	1
#42	CFRP wrap	6	Carbon	2
#59	CFRP wrap	2	Carbon	3
#43	CFRP wrap	6	Carbon	4
#48	GFRP wrap	2	Glass	1
#47	GFRP wrap	2	Glass	2
#50	GFRP wrap	2	Glass	3
#51	GFRP wrap	2	Glass	4
#52	GFRP wrap	2	Glass	1
#40	GFRP wrap	6	Glass	2
#53	GFRP wrap	2	Glass	3
#41	GFRP wrap	6	Glass	4



Note

All probes were embedded in Side A and C. Sides are labeled according to their relative position in the casting bed as:

A: exposed top side
B: right side of A
C: bottom side
D: left side of A

Figure 6.1 Position of ATR Probes and Thermocouple

6.3 Instrumentation and Data Measurements

Corrosion potential provides a measure of whether a specimen is corroding or not. The ATR reference electrodes used were calibrated against standard copper-copper sulfate reference electrodes. Generally, reference electrodes are placed on the surface of concrete to allow measurement of the corrosion potential. However, environmental factors such as concrete resistance, humidity, and junction contamination can affect the potential reading. For this reason, embedded reference electrodes such as those used in this study are recommended for long term measurement of corrosion potential.

A schematic drawing of the ATR reference electrode is shown in Fig. 6.2. These reference electrodes were fabricated at USF from titanium rods. The procedure for making these ATR reference electrodes is as follows [6.2]:

1. Cut titanium rod in 5 cm long pieces
2. Drill 0.06 in. (1.5 mm) diameter hole at one end of the titanium segment to a depth of 0.24 in. (6 mm)
3. Insert a stripped wire into the hole drilled in the titanium rod and crimp
4. Coat both ends of the titanium rod with EP-308 epoxy leaving an exposed titanium length of about 1.57 in. (40 mm)

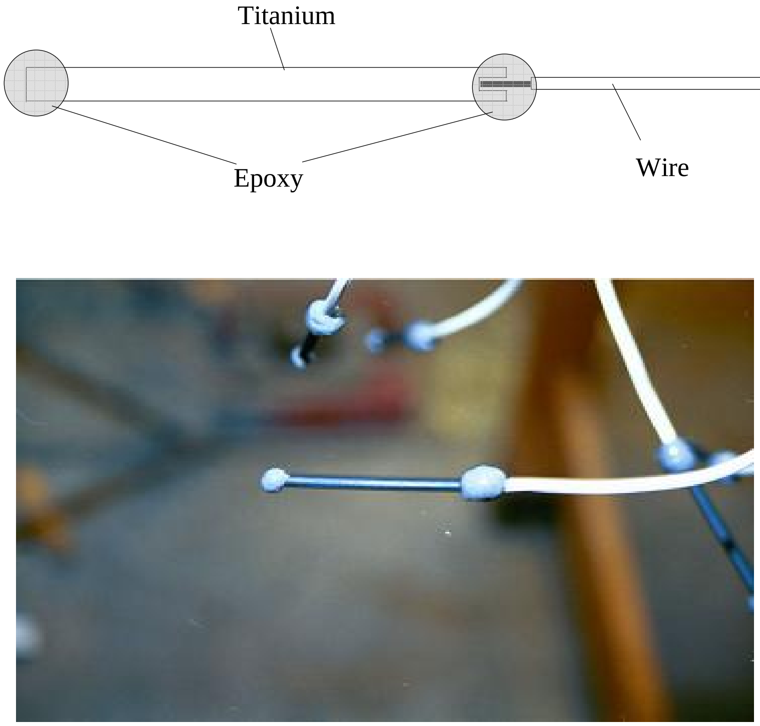


Figure 6.2 ATR Reference Electrode

Half cell potentials of all the specimens were measured using a high impedance voltmeter (MCM LC-1). To calibrate the titanium reference electrodes, a copper-copper sulfate reference electrode (CSE) was used. In the calibration, the negative terminal of the voltmeter was connected to a copper-copper sulfate reference electrode while the positive terminal was connected to the titanium reference electrode. The voltage measured is the calibration constant. Potential measurements (Fig. 6.3) were usually performed once a week initially with the first potential reading taken 24 days after the specimens were cast.

Figure 6.3   Measuring Potential

Linear polarization measurements were made using a PR Monitor manufactured by Cortest Instrument Systems. This has a three-electrode probe comprising a reference, working, and counter electrode (Fig. 6.4). A PR monitor measures the polarization resistance of the electrochemical system in a specimen. The polarization resistance is inversely proportional to the corrosion rate and allows the corrosion rate of the steel to be estimated. Concrete resistivity was measured using a soil resistance meter (Nilson 400). The polarized area was assumed to be the same as the chloride-contaminated area (22 in. length).

Figure 6.4 Linear Polarization Measurement Using PR Monitor

The ATR reference electrodes were soldered to a pre-made channel box to allow measurements to be made accurately and quickly. Thermocouples embedded in the concrete to measure temperature were hooked to a starlogger data acquisition system (Fig. 6.5). Temperature data was measured and recorded every hour.

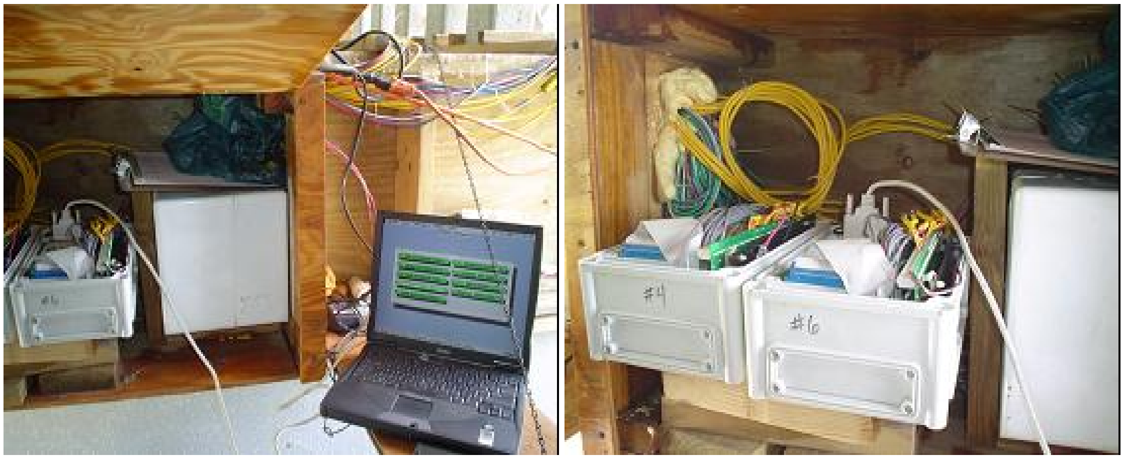


Figure 6.5 Temperature Data Measurement

6.4 Wrapping

Sixteen specimens were wrapped using FRP exactly 28 days after casting. Wrapping was applied over a 36 in. length in the central region of the specimen. This meant that the FRP extended 7 in. above and below the boundary of the 22 in. chloride contaminated region (Fig. 2.1). Eight specimens were wrapped with bi-directional carbon fiber reinforced polymer (CFRP) provided by SDR Engineering, Inc. And the other eight were wrapped using Tyfo® Web Composite and Tyfo® S epoxy donated by Fyfe, Co.

Tyfo® Web Composite system is a bi-directional glass fiber reinforced polymer (GFRP) manufactured by Fyfe, Co. LLC. Material properties of FRP and epoxies are presented in Chapter 3. The number of layers was varied from 1-4 (two specimens for each different layer) using the recommended lap lengths. For the CFRP this was 2 in., whereas for the GFRP it was 6 in.

The recommended procedure for wrapping was followed. Fyfe provided assistance for wrapping their specimens. The CFRP material had been used before and directions provided by the supplier were followed. Prior to wrapping all specimens were cleaned and the surfaces and edges made smooth using a grinder. Dust and concrete particles produced due to grinding were removed using acetone (Fig. 6.6).

Figure 6.6 Specimen Preparation

Resin and hardener were proportioned by volume and poured in a clean dry bucket. For the Fyfe system, the proportion was 100:42 while for the carbon system the corresponding volume ratio was 3:1. The two components were thoroughly mixed (5 minutes at 400-600 rpm for Fyfe and 3 minutes at 400 rpm for carbon) using a stirrer that was attached to a drill. The epoxy was uniformly coated on the concrete surface using roller brush and the precut FRP sheets were wrapped around the concrete using a roller to remove the air bubbles. To make sure there was complete bond between layers, another epoxy coat was applied over the installed sheets before successive FRP layers were wrapped. To protect the FRP wrap from UV, two coats of external latex paint having a grey color were applied over the wrapping area. Photos showing the various steps in wrapping the CFRP and GFRP material are shown in Figs 6.7 and 6.8, respectively.

(a) (b)

(c) (d)

Figure 6.7 (a) Cutting CFRP; (b) Resin and Hardener; (c) Applying CFRP; (d) Wrapped Specimens

(a) (b)

(c) (d)

Figure 6.8 (a) Cutting GFRP; (b) Mixing Epoxy; (c) Applying GFRP; (d) Wrapped and UV Protected Specimens

6.5 Exposure Set Up

Templates with cutouts were used to position the twenty specimens (16 wrapped and 4 unwrapped controls) vertically inside a 6 ft × 10 ft × 4 ft tank that was placed outdoors (Fig.6.9). In addition, two unwrapped specimens were placed in an indoor tank in a controlled environment (Fig. 6.10).

Figure 6.9 Setting for Outdoor Specimens

Figure 6.10 Setting for Indoor Specimens

All outdoor and indoor specimens were subjected to simulated tidal cycles in 3.5% salt water. The water level at high tide was 32 in. from the bottom and at low tide it was 14 in. This meant that a 2 in. length of the wrap was always submerged in water. This type of exposure had the maximum chance of trapping moisture within the wrap. Such entrapment of moisture had been found to be problematic in a previous study conducted by the University of Texas at Austin [6.1]. The water level was changed every 6 hours, and was controlled by a water pump and floating switch (Fig. 6.11). Thus, specimens were subjected to two tidal cycles daily.

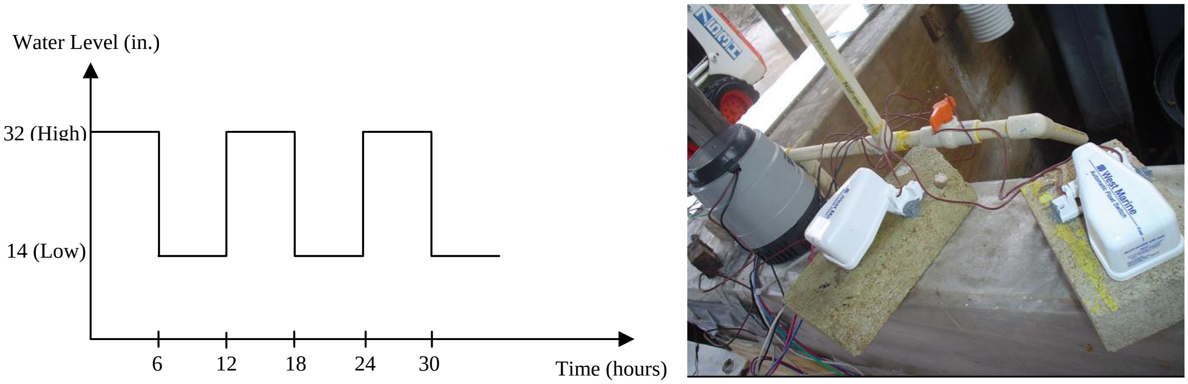


Figure 6.11 Simulated Tidal Cycle (L), and Water Pump & Floating Switches

Figure 6.12 Installed Outdoor Specimens

6.6 Test Results

When the exposure time reached about 300 days, cracks started to appear along the strand and corrosion products formed around cracks on the surface of the unwrapped specimens. After 1160 days exposure to the wet/dry cycles, all specimens were taken out from the tank and tested gravimetrically (Fig. 6.13).

Figure 6.13 Cracks on the Unwrapped Specimen (L) and Removal of Specimens After Targeted Exposure

6.6.1 Result of Corrosion Monitoring

6.6.1.1 Half-Cell Potential Result

The first potential reading was taken 24 days after pouring concrete. Wrapping was conducted on the 28th day and wet/dry cycles started on the 111th day after the concrete was cast.

Fig. 6.14 shows the change in the averaged half cell potential measured at the middle in four different types of specimens. All readings were more negative than 350mV indicating that there was a 90% probability of corrosion. The readings showed a big drop right after the start of wet/dry cycles possibly because of the availability of water. Potential changes were similar for the first 300 days. However, after 350 days the potential of the unwrapped control specimens became more negative while potential values of the FRP wrapped specimens showed a tendency to become less negative. The potential values of the wrapped specimens became more negative after 1000 days showing that the electrochemical environment around steel in wrapped specimens had changed.

There was little difference between the readings in the CFRP and GFRP specimens. The unwrapped specimens, whether indoor or outdoors, showed similar variation. Fig. 6.15 and 6.16 compare the effect of the number of CFRP and GFRP layers on the potential variation at the middle. For both CFRP and GFRP, the effect was minor.

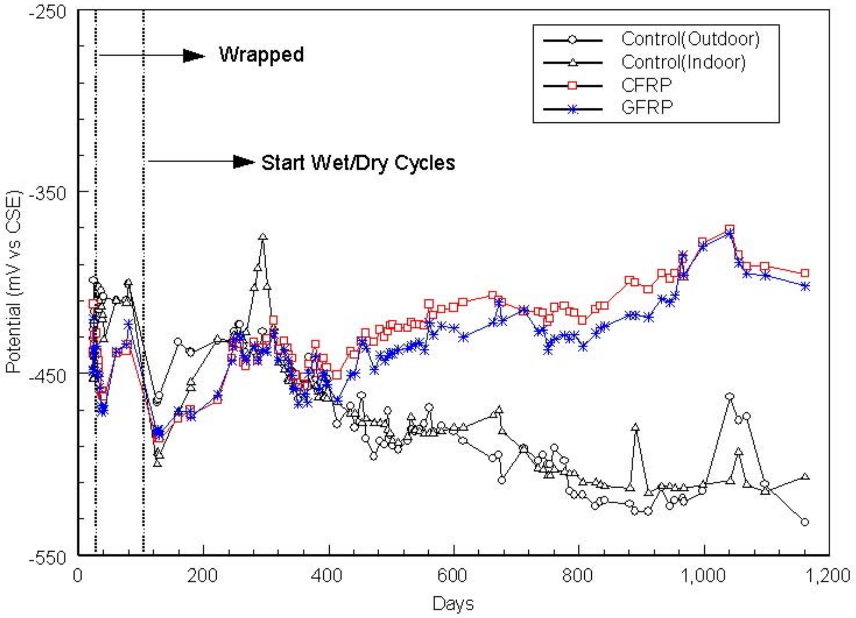


Figure 6.14 Overview of Averaged Potential Data At Middle

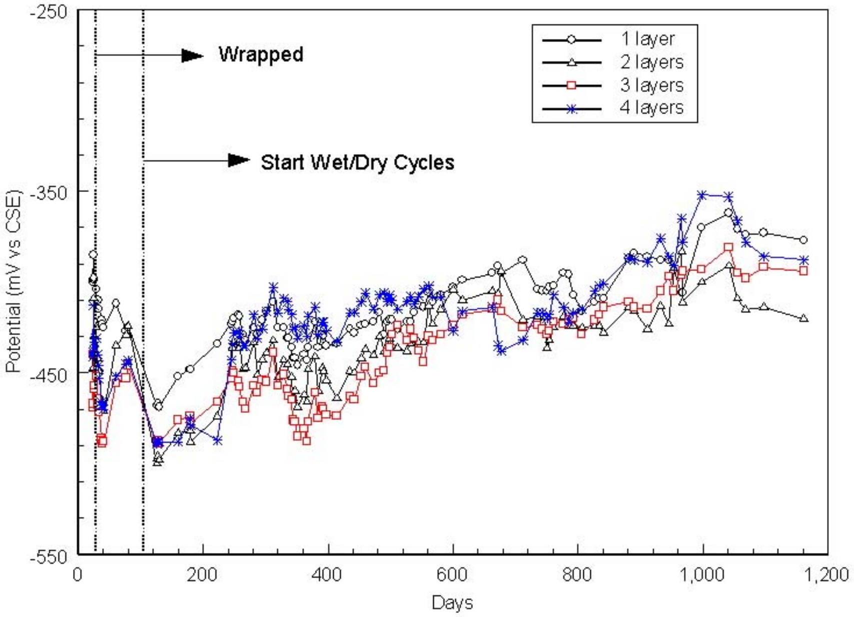


Figure 6.15 Effect of CFRP Layers on Potential At Middle

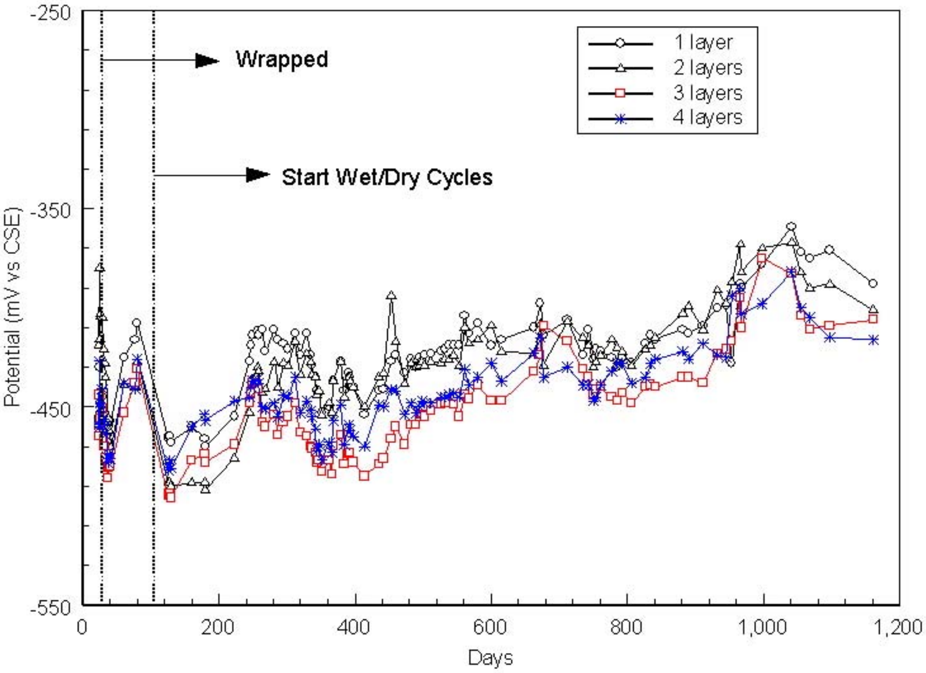


Figure 6.16 Effect of GFRP Layers on Potential at Middle

As mentioned in Section 6.2, to monitor the potential changes at three different levels in the same specimen, six references electrodes were provided in selected specimens. These were one outdoor control, one indoor control, two of CFRP wrapped specimens (2 & 4 layers), and two of GFRP wrapped specimens (2 & 4 layers). Three electrodes were installed on the A and C sides at three different levels (Fig. 6.1) - the constant dry area (Top), the tidal zone (Middle), and the constant wet area (Bottom).

Fig. 6.17–22 provides an overview of the potential change at the six locations in these selected specimens. More graphs comparing the potential values in different specimens are presented Appendix B. In the constantly dry area (Top), potential values were between –200 mV to –300 mV meaning there was little corrosion activity in any of the specimens.

The potential in the constantly wet region (Bottom) became more negative from the beginning of the wet/dry cycles and its variation was between –300mV to –500mV. Interestingly, its value was more negative in the indoor control than the outdoor control at the bottom (side C) (see Fig. B.34 in Appendix B). It was also more negative for the two-layer carbon wrapped specimens than the four-layer carbon wrapped ones (Fig. B.33 in Appendix B).

In the tidal area (Middle), the potential values of unwrapped specimens were most negative and showed a tendency to become more negative while those of wrapped specimens were similar or less negative than the potential in the bottom area.

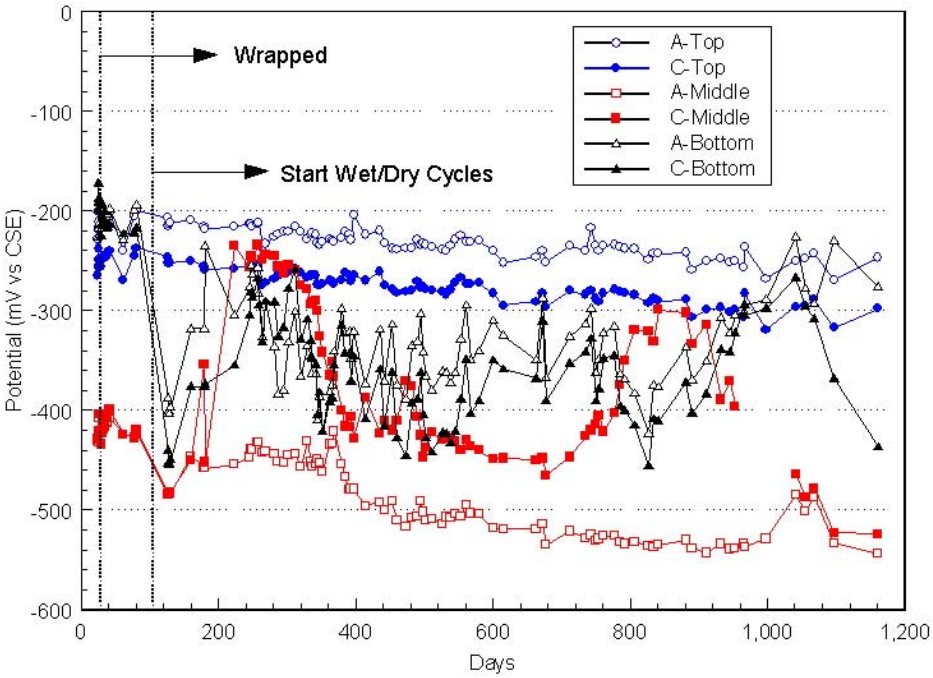


Figure 6.17 Potential Change at Three Levels in Outdoor Control Specimen

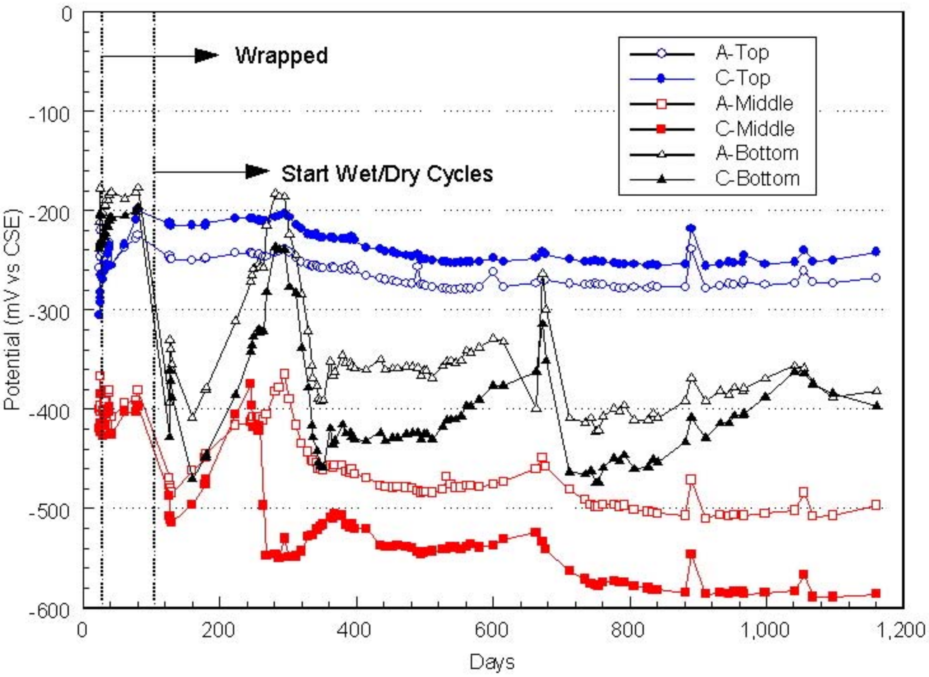


Figure 6.18 Potential Change at Three Levels in Indoor Control Specimen

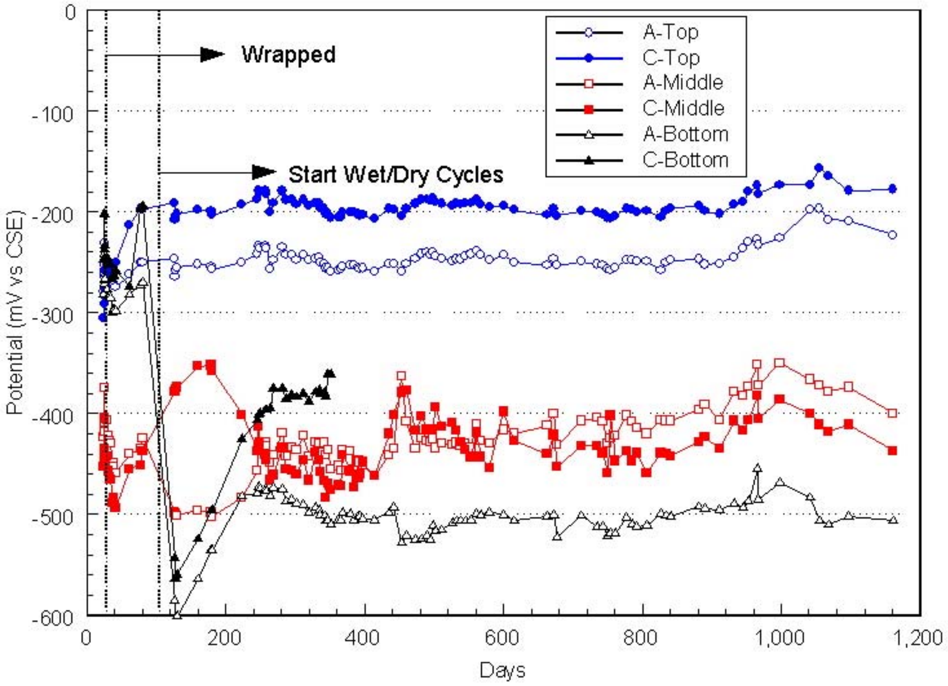


Figure 6.19 Potential Change at Three Levels in 2 Layer GFRP Wrapped Specimen

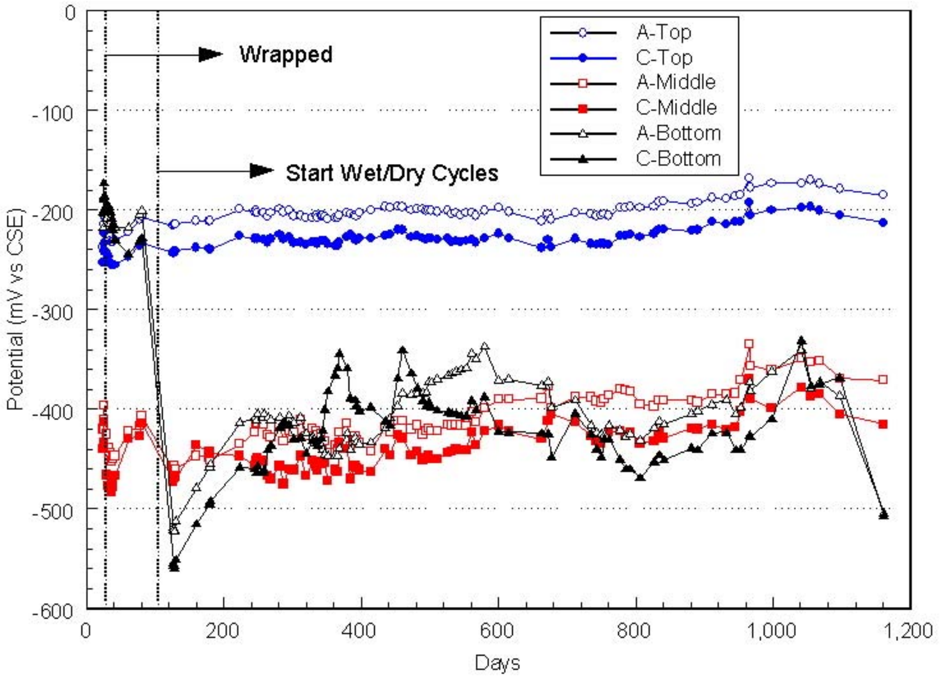


Figure 6.20 Potential Change at Three Levels in 4 Layer GFRP Wrapped Specimen

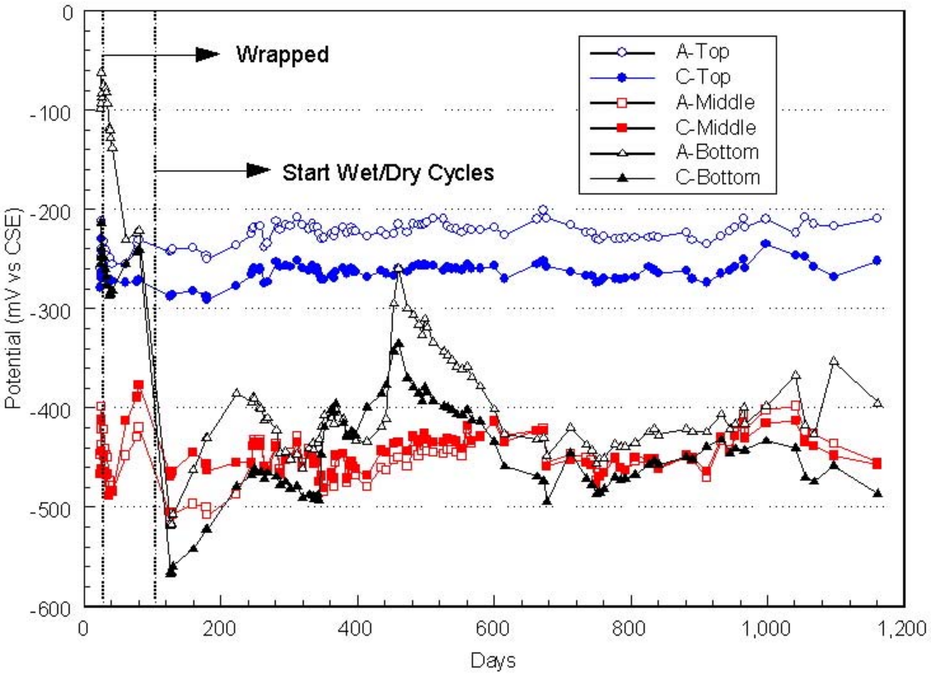


Figure 6.21 Potential Change at Three Levels in 2 Layer CFRP Wrapped Specimen

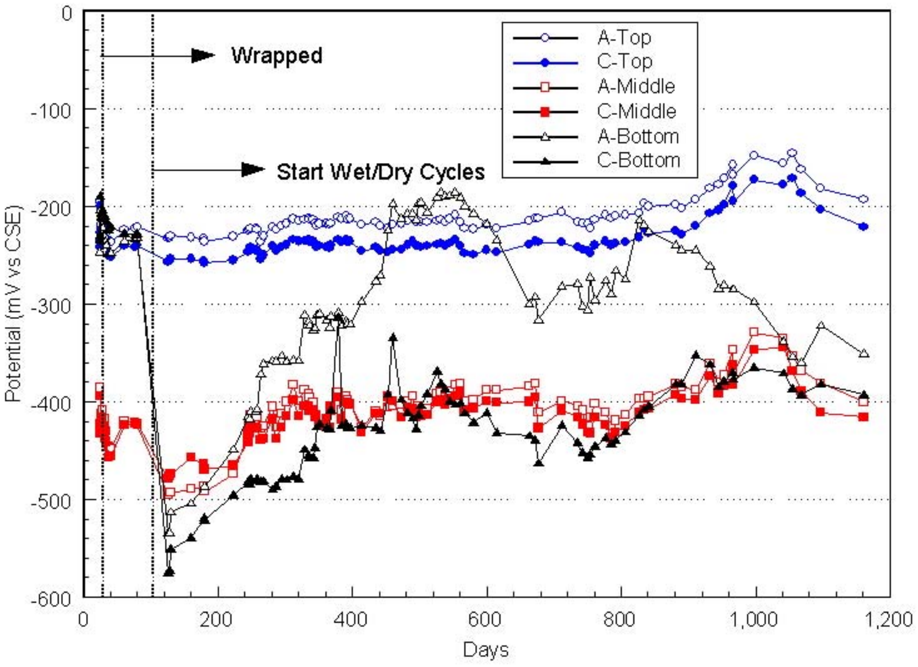


Figure 6.22 Potential Change at Three Level in 4 Layer CFRP Wrapped Specimen

6.6.1.2 Corrosion Rate Result

The variation in corrosion rate with exposure is presented in Fig. 6.23-25. Since linear polarization measurements take a long time (readings for 22 specimens took an entire day) all results relate to the corrosion rate at the middle that was expected to be the highest. An overview of the results is shown in Fig. 6.24 in which the variation in outdoor temperature is also plotted.

Inspection of Fig. 6.23 shows that there is a distinctive difference in the corrosion rate of wrapped and unwrapped specimens. The corrosion rate in mils per year (1 mil = 0.001 in.) was smaller for the wrapped specimens and the gap increased as the exposure period increased. The corrosion rate in wrapped specimens was stable after 150 days and decreased after 400 days. However, the corrosion rate of unwrapped specimens gradually increased. The fluctuation in all the readings seemed to be related to changes in ambient temperature. There appeared to be no difference in the change in the corrosion rate between carbon fiber and glass fiber.

Fig. 6.24 and 6.25 show the effect of the number of wrapping layer in corrosion rate. No discernible difference in performance can be detected at this time.

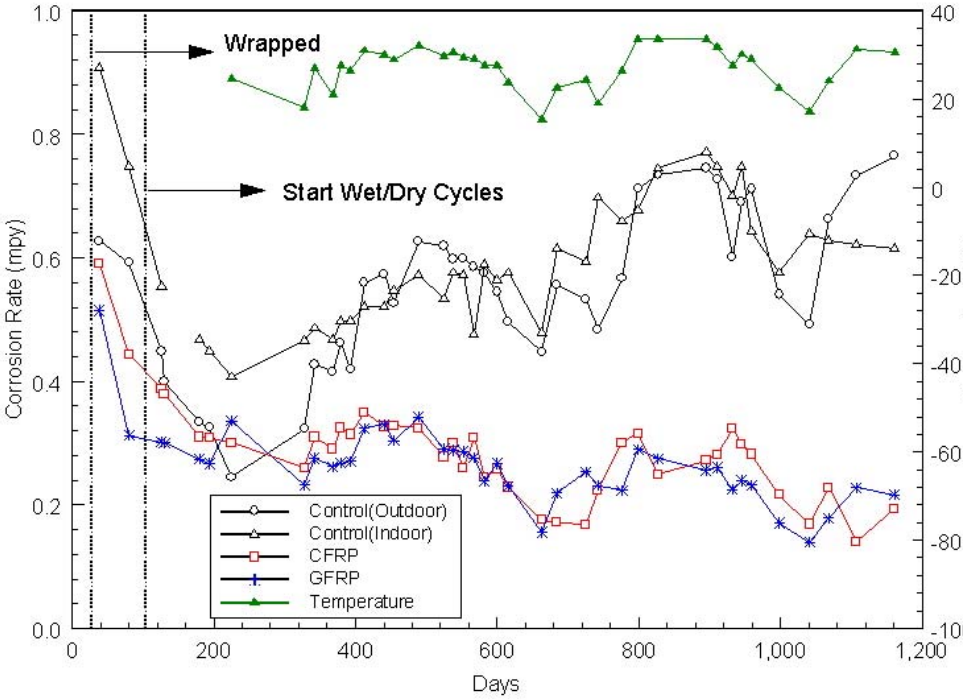


Figure 6.23 Overview of Results

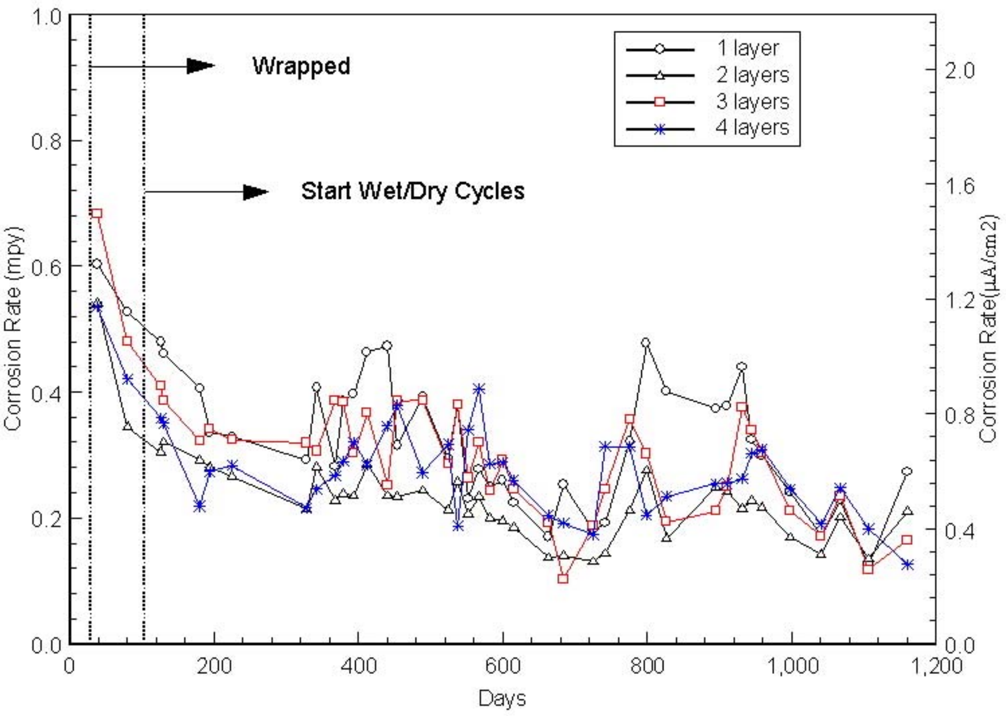


Figure 6.24 Effect of CFRP Layers on Corrosion Rate

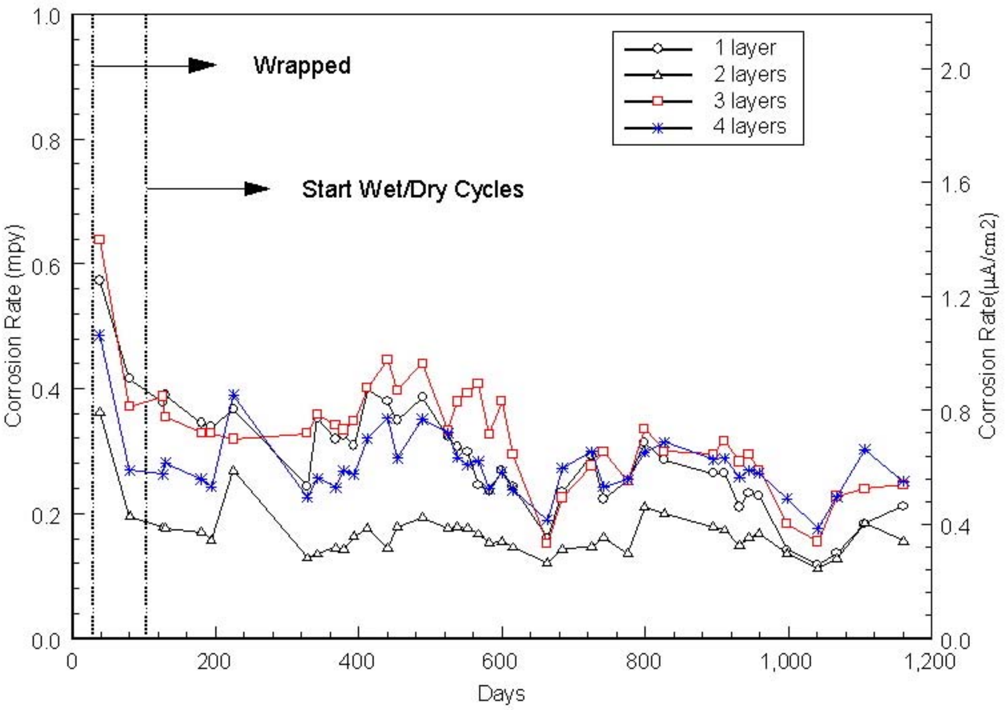


Figure 6.25 Effect of GFRP Layers on Corrosion Rate

6.6.1.3 Corrosion Monitoring Summary

The results of 1160 days of corrosion monitoring showed that FRP wrapped specimens had consistently lower readings for corrosion potential (Fig. 6.14) and corrosion rate (Fig. 6.23) compared to unwrapped specimens when exposed to the same environment. Moreover, in wrapped specimens, the variation of corrosion potential moved towards less negative readings and the corrosion rate gradually decreased with exposure. The performance of both carbon and glass were similar and it was independent of the number of FRP layers (Fig. 6.15, 6.16, 6.24, 6.25). Although readings for outdoor controls seemed to be affected by temperature change, there was little difference in the overall readings of outdoor and indoor controls.

6.6.2 Bond Strength of FRP

To find out if the bond between FRP and concrete was affected by the exposure, pull out tests were conducted using an Elcometer 106 Adhesion Tester. The tester used 1.456 in. diameter aluminum dollies (Fig. 6.26). A total of 8 wrapped specimens were tested in the study. Four were carbon fiber wrapped specimens and four were glass fiber wrapped ones with 1, 2, 3, and 4 FRP layers. The test was performed on two faces per specimen and at three locations per face (Fig. 6.27). The three levels selected were the dry zone (Top), tidal zone (Middle) and the submerged zone (Bottom).

Figure 6.26 Aluminum Dollies (L) and Coring with a Diamond Drill Bit (R)

The perimeter of the dolly was scored on the FRP surface at the selected locations using a 1.75 in. diameter drill bit. After the surface was cleaned with compressed air and acetone, dollies were bonded at these locations with a Power-Fast+ epoxy manufactured by Powers Fasteners, Inc. Following the manufacturer's recommendation, the test was performed after 24 hours of curing.

Figure 6.27 Performing Test with Elcometer (L) and Attached Dollies (R)

Specimens #54, #55, #56 and #57 were selected for the CFRP bond test. Table 6.2 summarizes the test results. The bond strength of carbon fiber wrapped specimens varied from 145 psi (#54A-Top) to 362.4 psi (#57B-Top). Nomenclature for the four sides in a specimen is labeled relative to their position in the casting bed as shown in Fig. 6.1. Most of the bond failures in the top and middle locations occurred in the concrete indicating that the bond was good. However, in the submerged zones failure occurred in the epoxy. Fig. 6.28 shows examples of concrete failure and epoxy failure in carbon fiber wrapped specimens.

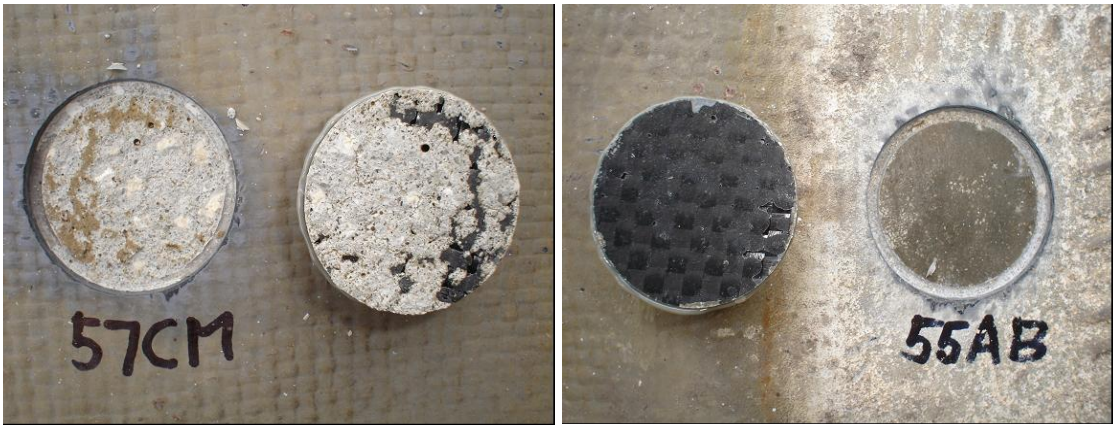


Figure 6.28 Concrete Failure (L) and Epoxy Failure Between Fiber and Concrete Surface in CFRP Wrapped Specimens (R)

Table 6.2 Results of Pull Out Test on CFRP Wrapped Specimens (unit: psi)

Specimen	**Side**	**Top**	**Middle**	**Bottom**
#54 (1 layer)	A	145.0 (epoxy)	145.0 (concrete)	203.0 (epoxy)
	D	275.4 (concrete)	260.9 (concrete)	232.0 (concrete)
	Average	**210.2**	**203.0**	**217.5**
#55 (2 layers)	A	289.9 (concrete)	304.4 (epoxy)	289.9 (epoxy)
	D	347.9 (concrete)	260.9 (concrete)	260.9 (concrete)
	Average	**318.9**	**282.7**	**275.4**
#56 (3 layers)	B	318.9 (concrete)	260.9 (concrete)	289.9 (concrete)
	C	N/A	N/A	N/A
	Average	**318.9**	**260.9**	**289.9**
#57 (4 layers)	B	217.5 (concrete)	275.4 (concrete)	N/A
	C	362.4 (concrete)	347.9 (concrete)	289.9 (epoxy)
	Average	**289.9**	**311.7**	**289.9**
Average		**284.5**	**264.6**	**268.2**

Note: Terms concrete, epoxy refer to failure mode`
Concrete failure refers to failure as shown in Fig. 6.28 (L) indicating good bond
Epoxy failure refers to separation of the FRP from the concrete indicating poor bond

Specimens #48, #47, #50 and #51 were selected for the GFRP bond test. The results of the bond tests on these GFRP wrapped specimens are summarized in Table 6.3. The bond strength of glass fiber wrapped specimens varied from 145 psi (#51A-Bottom) to 434.9 psi (#51B-Top). Although six out of eight test results showed the epoxy or layer failure in the bottom area (similar to CFRP specimens), the ultimate bond values for glass fiber wrapped specimens were higher.

Table 6.3 Results of Pull Out Test on GFRP Wrapped Specimens (unit: psi)

Specimen	**Side**	**Top**	**Middle**	**Bottom**
#48 (1 layer)	A	260.9 (epoxy)	174.0 (epoxy)	174.0 (epoxy)
	D	347.9 (concrete)	260.9 (concrete)	232.0 (epoxy)
	Average	**304.4**	**217.5**	**203.0**
#47 (2 layers)	A	318.9 (epoxy)	260.9 (epoxy)	362.4 (epoxy)
	B	246.4 (concrete)	260.9 (concrete)	347.9 (concrete)
	Average	**282.7**	**260.9**	**355.2**
#50 (3 layers)	A	217.5 (epoxy)	333.4 (concrete)	333.4 (epoxy)
	B	275.4 (concrete)	203.0 (concrete)	405.9 (layer)
	Average	**246.4**	**268.2**	**369.7**
#51 (4 layers)	A	260.9 (epoxy)	318.9 (concrete)	145.0 (epoxy)
	B	434.9 (concrete)	275.4 (concrete)	405.9 (concrete)
	Average	**347.9**	**297.2**	**275.4**
Average		**295.4**	**260.9**	**300.8**

Note: Terms concrete, epoxy, layer refer to failure mode
Concrete failure refers to failure as shown in Fig. 6.29 (L) indicating good bond
Epoxy failure refers to separation of the GFRP from the concrete indicating poor bond
Layer failure refers to separation of GFRP layers indicating the inter-layer bond was poorer than the FRP concrete bond – only 1 such failure in specimen #50 was recorded.

Figure 6.29 Concrete Failure (L) and Epoxy Failure Between Fiber and Concrete Surface in GFRP Wrapped Specimens (R)

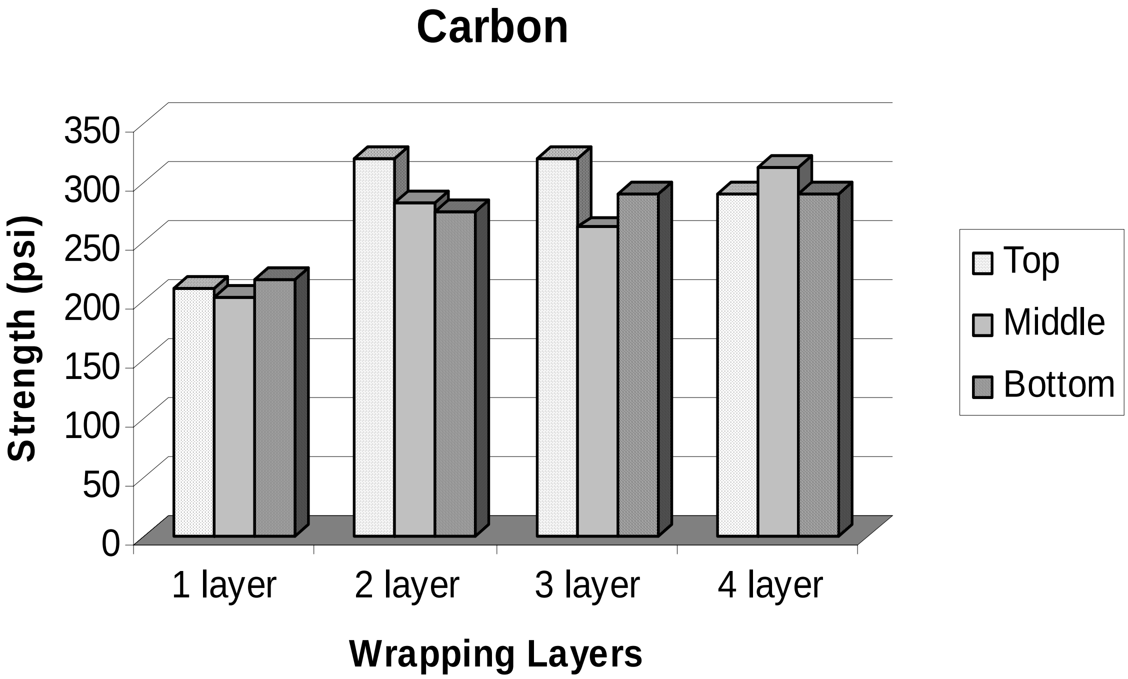


Figure 6.30 Bond Strength of CFRP Wrapped Specimens

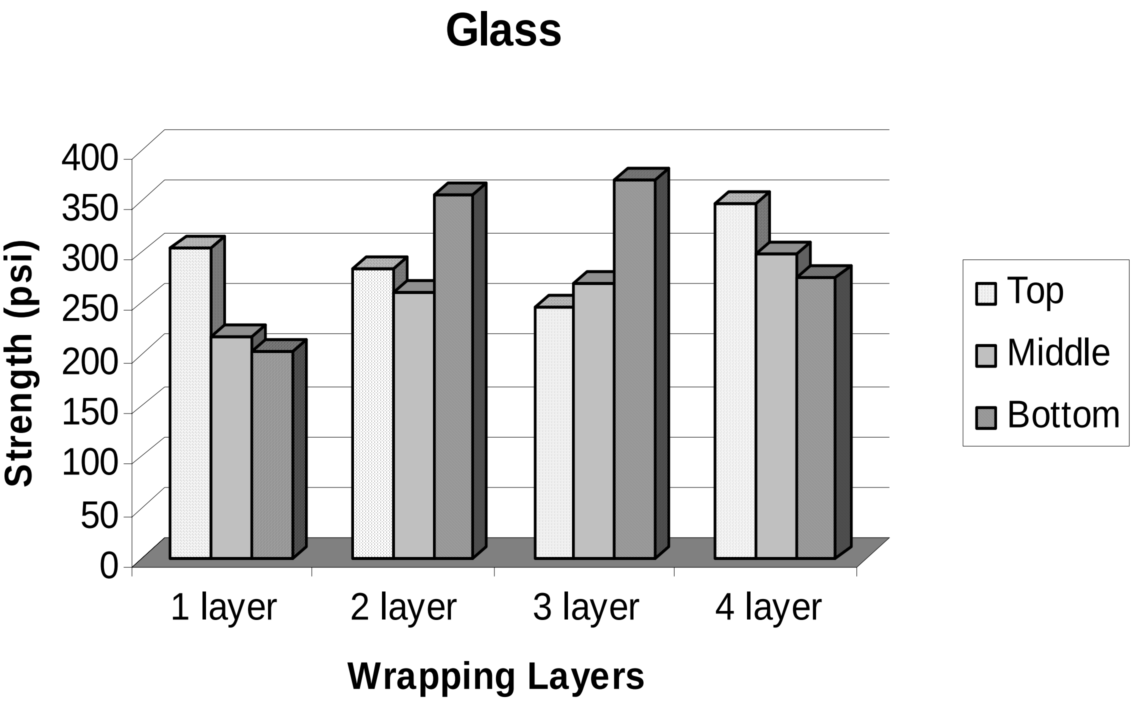


Figure 6.31 Bond Strength of GFRP Wrapped Specimens

Table 6.2 and Fig. 6.30-31 show that the average bond strength varied from 264 psi to 285 psi in CFRP wrapped specimens, and from 260 psi to 300 psi in GFRP wrapped ones. Thus, the average bond strength was higher in the glass fiber wrapped specimens than in the carbon fiber wrapped ones.

Although, epoxy failure was the most commonly occurring mode in the submerged region, the measured ultimate bond stress was not significantly lower than that at other locations. The bond strength was also unaffected by the number of FRP layers indicating that the material was properly bonded in the first place.

6.6.3 Crack Survey

A crack survey was performed on the six unwrapped specimens (Fig. 6.32-33). Table 6.4 shows a summary of the results. All outdoor control specimens had cracks on at least 3 faces while indoor controls (#39, #49) had cracks on only 2 faces. The maximum crack width (0.75 mm) was found in outdoor controls #38 and #44, (Fig. 6.33) and the maximum crack length (35 in.) was found in the indoor control #39 (Fig. 6.32).

All cracks were concentrated in the chloride contaminated area, and occurred on faces B, C and D (see Fig. 6.1 for definition of sides), i.e. all faces other than the top face that was exposed during fabrication of the specimen in the bed.

Table 6.4 Results of Crack Survey

	Name	Max. Cracks	A	B	C	D
Outdoor Controls	#38	Width (mm)	No	0.4	**0.75**	0.4
		Length (in.)		3	26	20
	#44	Width (mm)	**0.75**	**0.75**	0.3	0.25
		Length (in.)	20.5	20.5	6.5	17
	#45	Width (mm)	No	0.4	0.3	0.5
		Length (in.)		9	17	18
	#46	Width (mm)	0.2	0.2	0.3	0.2
		Length (in.)	6.5	11.5	16.5	10
Indoor Controls	#39	Width (mm)	No	No	0.5	0.2
		Length (in.)			**35**	11
	#49	Width (mm)	No	0.4	No	0.4
		Length (in.)		18.5		14

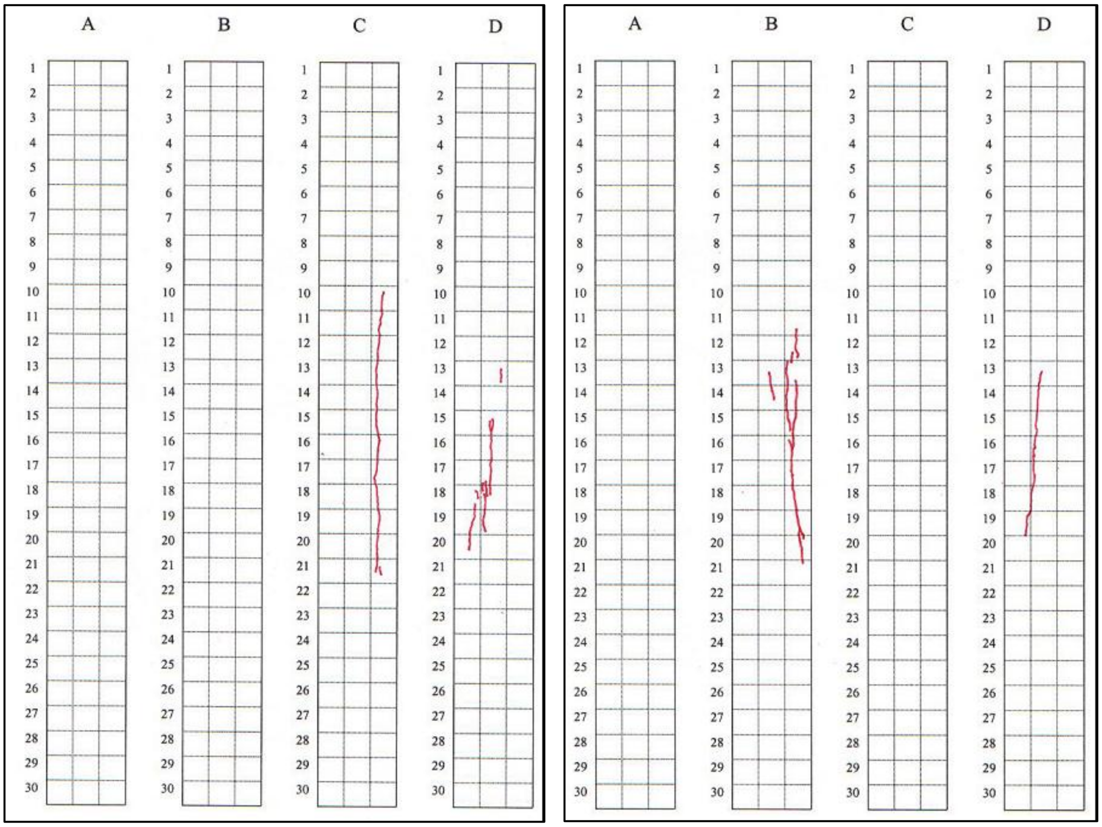
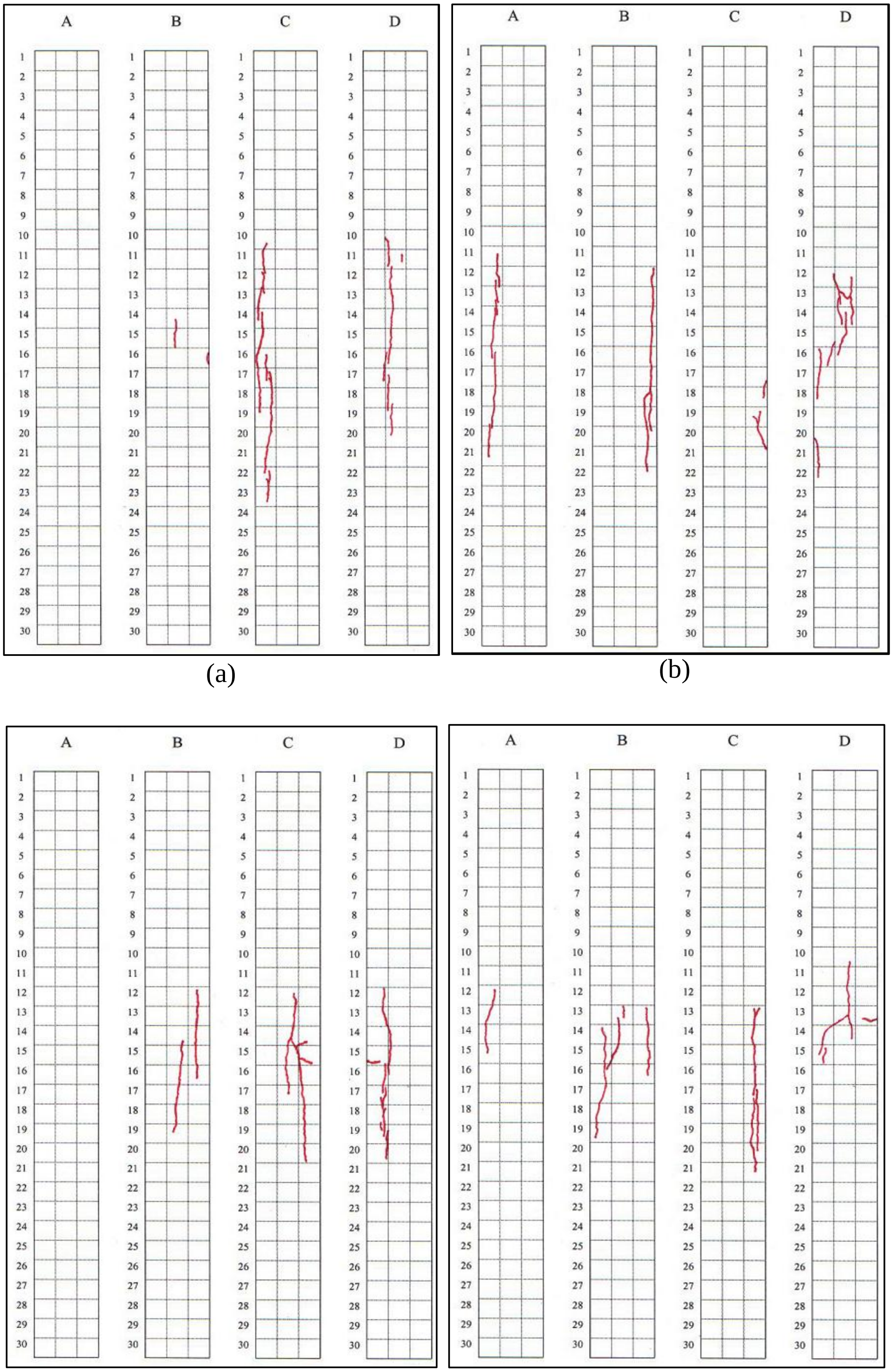


Figure 6.32 Crack Pattern in Indoor Controls #39 (L) and #49 (R)



(a) (b)

(c) (d)

Figure 6.33 Crack Pattern in Outdoor Controls (a) #38, (b) #44, (c) #45, (d) #46

6.6.4 Gravimetric Test

All 22 specimens were gravimetrically tested to measure the actual steel loss. Longitudinal cuts were made on the concrete surface with an electric saw and the cover was then chipped off with a hammer. The distribution of the corrosion products was then measured. Later, prestressing strands and ties were carefully retrieved. The strands were cut to 36 in. length and subsequently cleaned with a wire brush. In the cleaning process, the strands were disassembled into seven separate wires to ensure there was no rust. Since the target area contaminated with chloride was 22 in. at the center, reported steel loss is with respect to this 22 inch section. This provides a slightly higher average loss since the entire metal loss in the 36 in. strand is assumed to occur over this length.

Figure 6.34 Breaking Surface Concrete to Retrieve Strands

As noted earlier, the Texas study [6.1] reported that the FRP wrap had entrapped water that had led to increased corrosion inside the wrap. No similar entrapment was found in this study and no similar corrosion was observed. This is shown in Fig. 6.35 that shows the lower area of wrapped specimens that be seen to be in good condition.

Figure 6.35 Lower Area of Wrapped Specimen Exposed to Wet/Dry Cycles

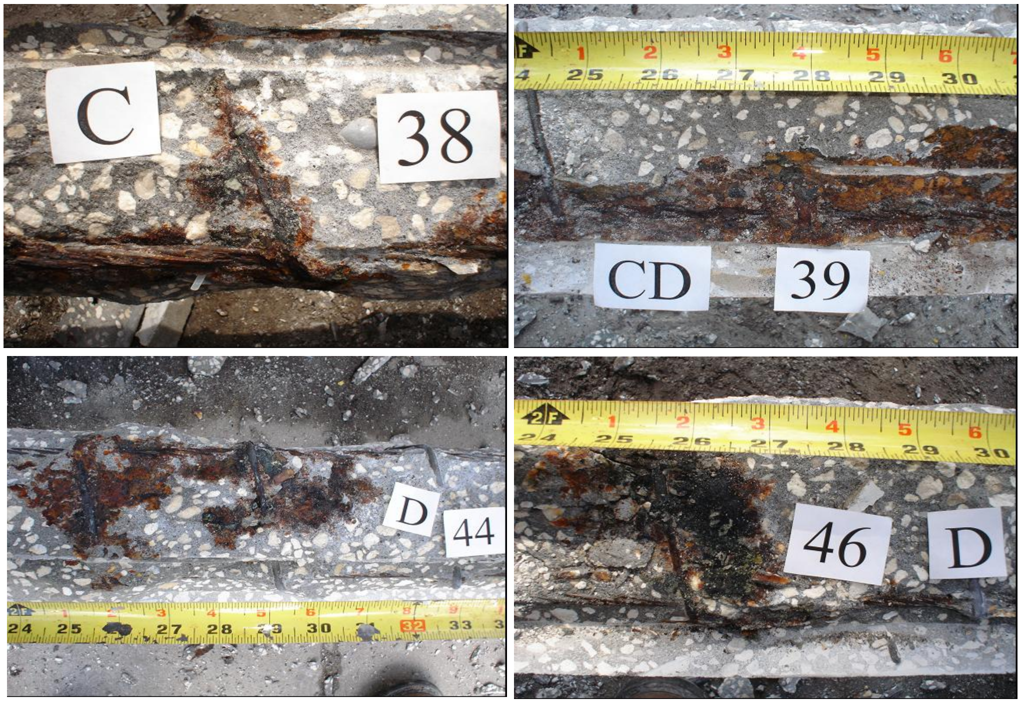


Figure 6.36 Exposed Steel in Unwrapped Control Specimens

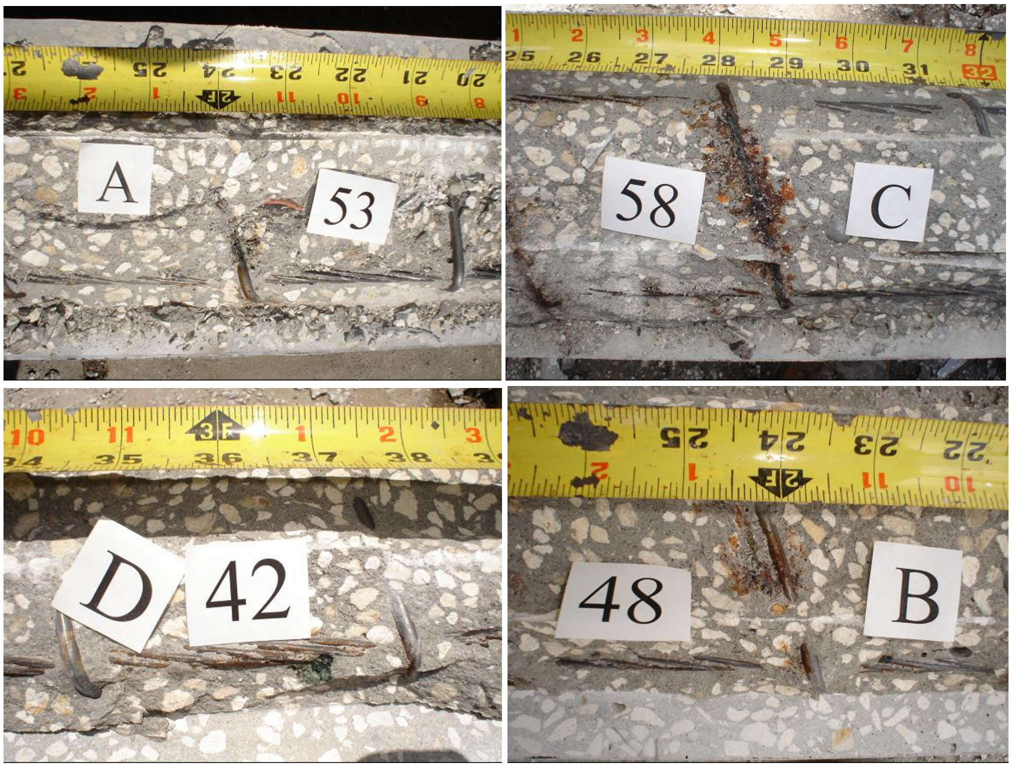


Figure 6.37 Exposed Steel in Wrapped Specimens

Fig. 6.38 shows the distribution of corrosion product in the unwrapped controls. In all specimens, the corrosion product did not extend beyond the chloride contaminated zone and its distribution was symmetric with respect to its center.

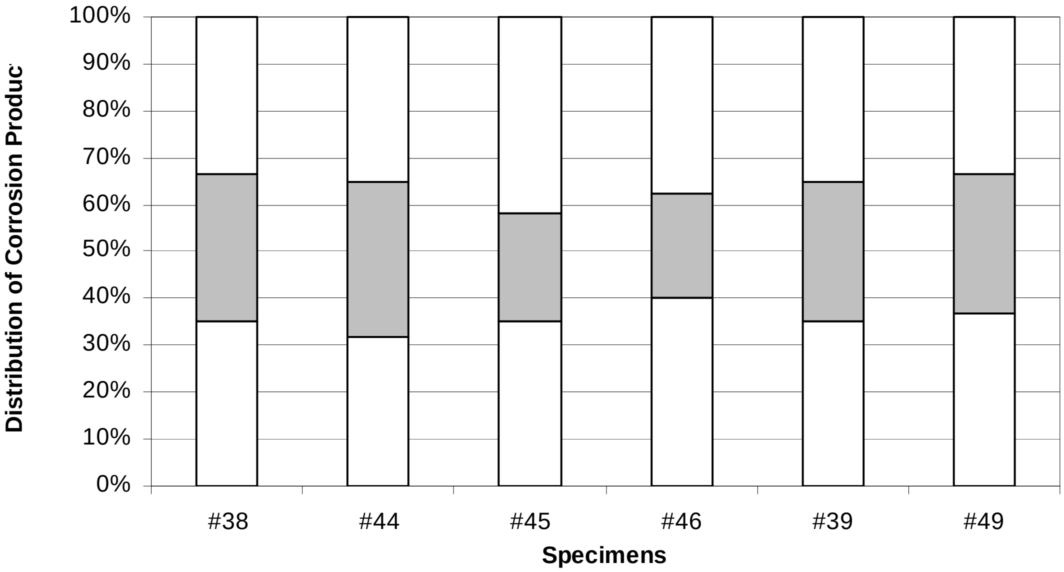


Figure 6.38 Distribution of Corrosion Products in Unwrapped Specimens

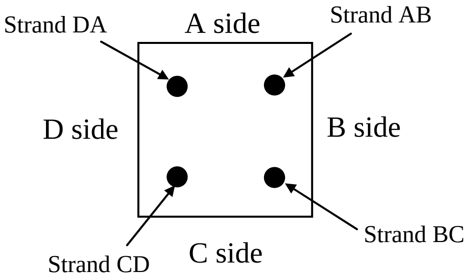


Figure 6.39 Strand Nomenclature
(side A was the exposed top surface in the prestress bed during fabrication)

Table 6.5 Gravimetric Test Results of Controls

Specimen	**Strand / Tie (Fig. 6. 38)**	**Break in Strand Wire**	**Original Weight (g)**	**Lost Weight (g)**	**Percent Loss**
#38 Outdoor	AB	0	168.8	7.4	4.4
	BC	3	168.8	17.5	**10.4**
	CD	0	168.8	10	5.9
	DA	0	168.8	7.9	4.7
	tie	1	332.3	38.9	11.7
#44 Outdoor	AB	0	168.8	8.3	4.9
	BC	0	168.8	16.4	**9.7**
	CD	0	168.8	11.4	6.8
	DA	3	168.8	15.4	9.1
	tie	2	332.3	32.7	9.8
#45 Outdoor	AB	0	168.8	6.2	3.7
	BC	2	168.8	8.9	5.3
	CD	4+1	168.8	19.8	**11.7**
	DA	0	168.8	8.1	4.8
	tie	1	332.3	32.2	9.7
#46 Outdoor	AB	0	168.8	9.8	5.8
	BC	0	168.8	8.2	4.9
	CD	0	168.8	14.5	**8.6**
	DA	0	168.8	7.3	4.3
	tie	2	332.3	30.1	9.1
#39 Indoor	AB	0	168.8	7.1	4.2
	BC	0	168.8	8.3	4.9
	CD	6	168.8	21.2	**12.6**
	DA	0	168.8	7.2	4.3
	tie	2	332.3	33.1	10.0
#49 Indoor	AB	0	168.8	8.8	5.2
	BC	1	168.8	14.3	**8.5**
	CD	0	168.8	8.6	5.1
	DA	1	168.8	13.2	7.8
	tie	1	332.3	26.2	7.9

Note: Where the central wire in a 7-wire strand was broken, it is reported in the form a+1 where a signifies the number of other wires broken. All such breaks occurred in the middle region of the specimen

Table 6.6 Gravimetric Test Results of CFRP Wrapped Specimens

No of Layers	**Specimen**	**Strand / Tie (Fig. 6.38)**	**Break in Strand Wire**	**Original Weight (g)**	**Lost Weight (g)**	**Percent Loss**
1	#54	AB	0	168.8	6.1	3.6
		BC	0	168.8	6.2	**3.7**
		CD	0	168.8	5.6	3.3
		DA	0	168.8	4.9	2.9
		Tie	0	332.3	22.6	6.8
	#58	AB	0	168.8	4.4	2.6
		BC	0	168.8	7.4	**4.4**
		CD	0	168.8	6.4	3.8
		DA	0	168.8	6.2	3.7
		Tie	0	332.3	24.4	7.3
2	#55	AB	1	168.8	5.6	3.3
		BC	0	168.8	5.8	**3.4**
		CD	0	168.8	4.8	2.8
		DA	0	168.8	4.7	2.8
		Tie	0	332.3	20.3	6.1
	#42	AB	0	168.8	4.7	2.8
		BC	0	168.8	5.5	**3.3**
		CD	0	168.8	5.6	**3.3**
		DA	0	168.8	5.4	3.2
		tie	0	332.3	17.3	5.2
3	#56	AB	0	168.8	5.3	3.1
		BC	0	168.8	5.1	3.0
		CD	0	168.8	6.7	**4.0**
		DA	0	168.8	5.5	3.3
		tie	0	332.3	22.2	6.7
	#59	AB	0	168.8	7.3	**4.3**
		BC	0	168.8	5.5	3.3
		CD	0	168.8	5.2	3.1
		DA	0	168.8	5.8	3.4
		tie	0	332.3	23.6	7.1
4	#57	AB	0	168.8	5.1	3.0
		BC	0	168.8	5.4	3.2
		CD	0	168.8	5.9	3.5
		DA	0	168.8	6.7	**4.0**
		tie	0	332.3	25.5	7.7
	#43	AB	0	168.8	5.3	3.1
		BC	0	168.8	5.2	3.1
		CD	0	168.8	6	**3.6**
		DA	0	168.8	5	3.0
		tie	0	332.3	20.2	6.1

Table 6.7 Gravimetric Test Results of GFRP Wrapped Specimens

Layer	**Specimen**	**Strand /Tie Fig. 6.38**	**Break in Strand Wire**	**Original Weight (g)**	**Lost Weight (g)**	**Percent Loss**
1	#48	AB	0	168.8	5.7	3.4
		BC	0	168.8	6.5	**3.9**
		CD	0	168.8	6	3.6
		DA	0	168.8	5.4	3.2
		Tie	0	332.3	21.3	6.4
	#52	AB	0	168.8	6.5	**3.9**
		BC	0	168.8	6.5	**3.9**
		CD	0	168.8	5.5	3.3
		DA	0	168.8	5.9	3.5
		Tie	0	332.3	22.9	6.9
2	#47	AB	0	168.8	5.2	3.1
		BC	0	168.8	6.1	3.6
		CD	0	168.8	6.2	**3.7**
		DA	0	168.8	5.2	3.1
		Tie	0	332.3	20.1	6.0
	#40	AB	0	168.8	5.1	3.0
		BC	0	168.8	5.7	3.4
		CD	0	168.8	6	**3.6**
		DA	0	168.8	5.2	3.1
		Tie	0	332.3	20.9	6.3
3	#50	AB	0	168.8	6.2	3.7
		BC	0	168.8	6.3	3.7
		CD	0	168.8	6.6	**3.9**
		DA	0	168.8	5.9	3.5
		Tie	0	332.3	21.2	6.4
	#53	AB	0	168.8	6.2	**3.7**
		BC	0	168.8	5.8	3.4
		CD	0	168.8	4.9	2.9
		DA	0	168.8	5.3	3.1
		Tie	0	332.3	18.1	5.4
4	#51	AB	0	168.8	6	3.6
		BC	0	168.8	6.3	3.7
		CD	0	168.8	6.6	**3.9**
		DA	0	168.8	5.9	3.5
		Tie	0	332.3	21.2	6.4
	#41	AB	0	168.8	4.9	2.9
		BC	0	168.8	4.7	2.8
		CD	0	168.8	6.3	**3.7**
		DA	0	168.8	4.5	2.7
		Tie	0	332.3	21.9	6.6

The results of the gravimetric tests for controls, CFRP wrapped and GFRP wrapped specimens are summarized in Tables 6.5-6.7 respectively. These tables provide details on the measured metal loss in ties and each of the four strands identified as AB, BC, CD and DA as defined in Fig. 6.39.

Table 6.5 summarizes the results for both the indoor and outdoor controls. In all the specimens, one or two ties were completely corroded though none of the strands were completely corroded. However, with the exception of one outdoor control (#46), there were breaks in individual wires making up the 7 wire strand in the remaining five controls. The largest number of breaks was in the outdoor control #45 in which a total of 7 breaks occurred in two strands (BC and CD – here the center wire was also broken). Unfortunately, the significance of such localized damage is not reflected by the percent loss values in which the metal loss is averaged over 22 in. length. These losses ranged from 3.7% to a maximum of 12.6%.

In contrast to the performance of the unwrapped controls, the wrapped specimens exposed to the same environment fared much better. There was only one break in one wire in one strand in one CFRP wrapped specimen (#55, 2 layer, strand AB in Table 6.6). None of the ties had corroded. The percent metal loss ranged from 2.6% to 7.7% in CFRP wrapped specimens and from 2.7% to 6.9% in the GFRP wrapped specimens (Table 6.7). The effect of number of FRP layers was not significant. This was also the conclusion from the corrosion measurements. This suggests that FRP can only provide a certain level of protection that can be attained using relatively few layers.

A summary of the measured steel loss from all the results is shown in Table 6.8. In this table, the total loss in all 4 strands and ties is averaged and compared for the controls and the wrapped specimens.

It may be seen from Table 6.8 that the averaged steel loss in strands and ties in outdoor and indoor unwrapped specimens were similar (6.6% and 10.1% vs 6.6% and 8.9%). These suggest that temperature and humidity variation did not make as much a difference. Thus, corrosion gains made in the outdoor specimens during summer and fall were offset by lower corrosion rates in winter and spring. In contrast, specimens inside the laboratory corroded at a more or less uniform rate throughout the exposure period.

The performance of the wrapped specimens was much better. The average metal loss in strands was 3.3% for carbon and 3.4% for glass about half that of the 6.6% in the controls. For the ties, average metal loss was 6.9% for carbon and 6.3% for glass compared to 10.1% for the outdoor controls.

The effect of number of FRP layers beyond two layers was minimal. Results that are averaged over 22 in. and for this reason it indicates that they were marginally worse as the number of layers increased. Overall, the results for carbon and glass were comparable with glass providing slightly better protection for the ties and the carbon slightly better for the strands.

Table 6.8 Averaged Steel Loss of Each Specimen (unit: %)

		Name	**Strand**	**Tie**
Outdoor Control		#38	6.3	11.7
		#44	7.6	9.8
		#45	6.4	9.7
		#46	5.9	9.1
		Average	**6.6**	**10.1**
Indoor Control		#39	6.5	10.0
		#49	6.6	7.9
		Average	**6.6**	**8.9**
Carbon	1 layer	#54	3.4	6.8
		#58	3.6	7.3
		Average	**3.5**	**7.1**
	2 layers	#55	3.1	6.1
		#42	3.1	5.2
		Average	**3.1**	**5.7**
	3 layers	#56	3.3	6.7
		#59	3.5	7.1
		Average	**3.4**	**6.9**
	4 layers	#57	3.4	7.7
		#43	3.2	6.1
		Average	**3.3**	**6.9**
	Carbon Average		**3.3**	**6.6**
Glass	1 layer	#48	3.5	6.4
		#52	3.6	6.9
		Average	**3.6**	**6.7**
	2 layers	#47	3.4	6.0
		#40	3.3	6.3
		Average	**3.3**	**6.2**
	3 layers	#50	3.7	6.4
		#53	3.3	5.4
		Average	**3.5**	**5.9**
	4 layers	#51	3.7	6.4
		#41	3.0	6.6
		Average	**3.3**	**6.5**
	Glass Average		**3.4**	**6.3**

The results in Tables 6.5-6.7 are re-plotted in Fig. 6.40-41 to compare the relative performance of carbon and glass respectively with respect to controls. In these plots, the *worst* performance for the controls is compared against the *worst* performance for the wrapped specimen. For example, the highest metal loss in the control was in strand CD in specimen #39 shown in bold in Table 6.5. That for the ties was in specimen #38 in the same table. Similarly, for the CFRP wrapped specimens the largest metal loss in strands was 4.4% for 1 layer (specimen # 58 in Table 6.6) and 3.4% (specimen #55 in Table 6.6). For ties, the largest loss 2 layer was 6.1% (specimen #55 in Table 6.6). Values were glass were similarly obtained from Table 6.7.

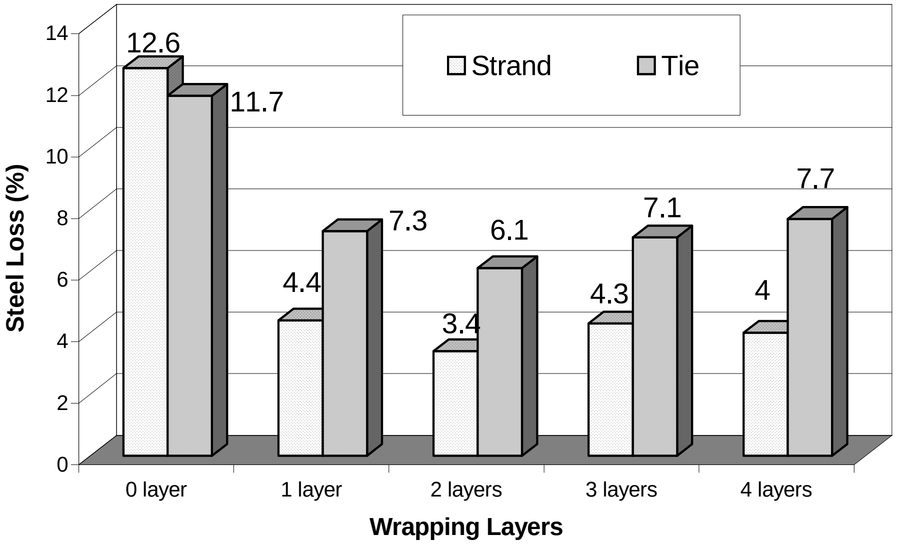


Figure 6.40 Effect of CFRP Wrap on Maximum Steel Loss (unit: %)

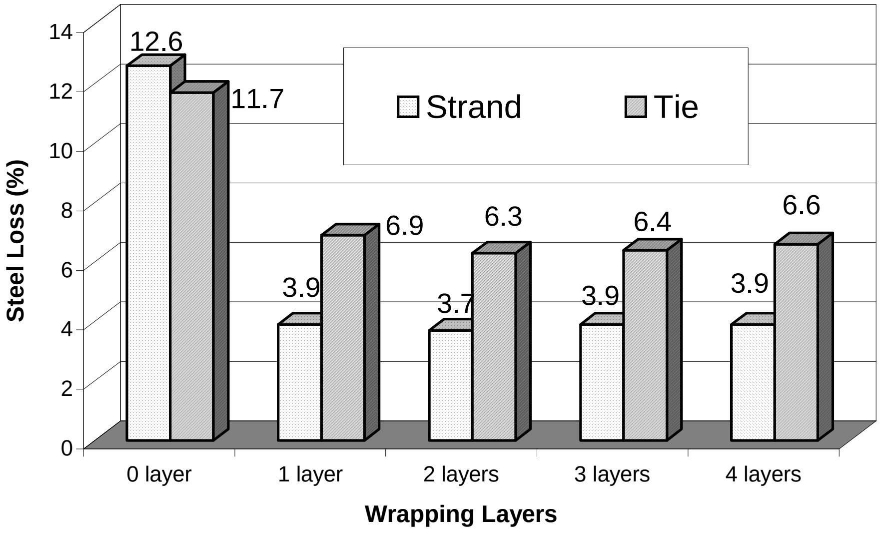


Figure 6.41 Effect of GFRP Wrap on Maximum Steel Loss (unit: %)

Fig. 6.40-41 show that the maximum metal loss in strands in wrapped specimens was about 1/3rd the corresponding maximum metal loss in the unwrapped controls. The improvement was somewhat less – about 50% - for ties. The performance did not improve when the number of FRP layers exceeded two. Also, the performance of carbon and glass were comparable.

Steel loss of strands might be different according to their position in the casting bed. The result of the crack survey (Fig. 6.32–33) showed that cracks were more concentrated along strands BC and CD (see Fig. 6.39 for definition) that were located at the bottom in the casting bed. To determine if the position of the strand has an effect on how much it corroded, the average steel loss of each strand was calculated for the unwrapped controls and the CFRP and GFRP wrapped specimens. This is shown in Fig. 6.42. The steel loss was highest in strand CD and second highest in strand BC among the control specimens. In the wrapped specimens, however, all strands showed similar steel loss.

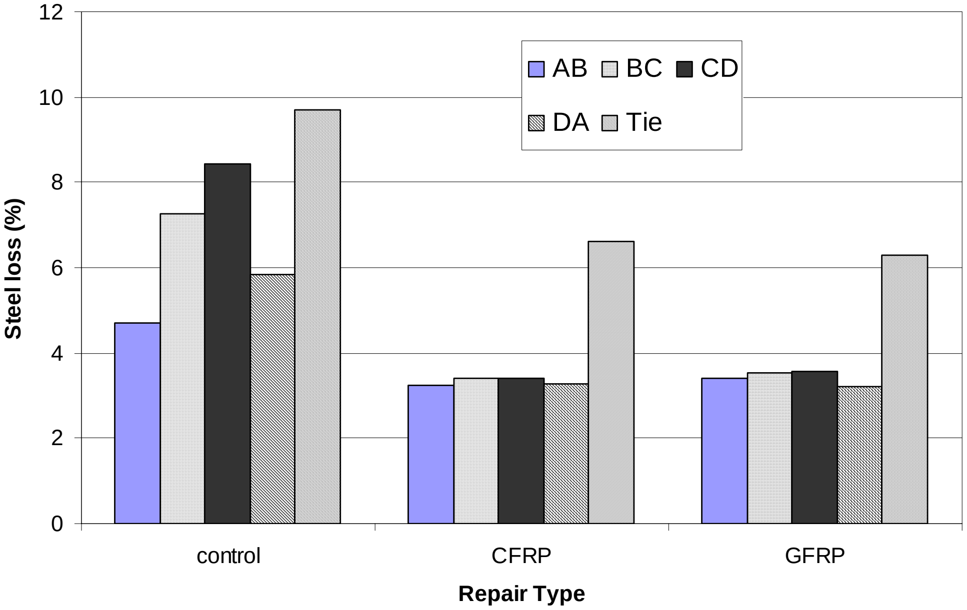


Figure 6.42 Average Steel Loss in Strand

Corrosion rate measurement is widely used for assessing deterioration in embedded steel non-destructively. To verify the effectiveness of these measurements, the corrosion rate reading obtained in this study is compared against measured metal loss from the gravimetric testing.

The actual steel loss of strands in 22 specimens is compared with the final corrosion rate values taken at the middle of the specimens. This is shown in Fig. 6.43 for the corrosion rate.

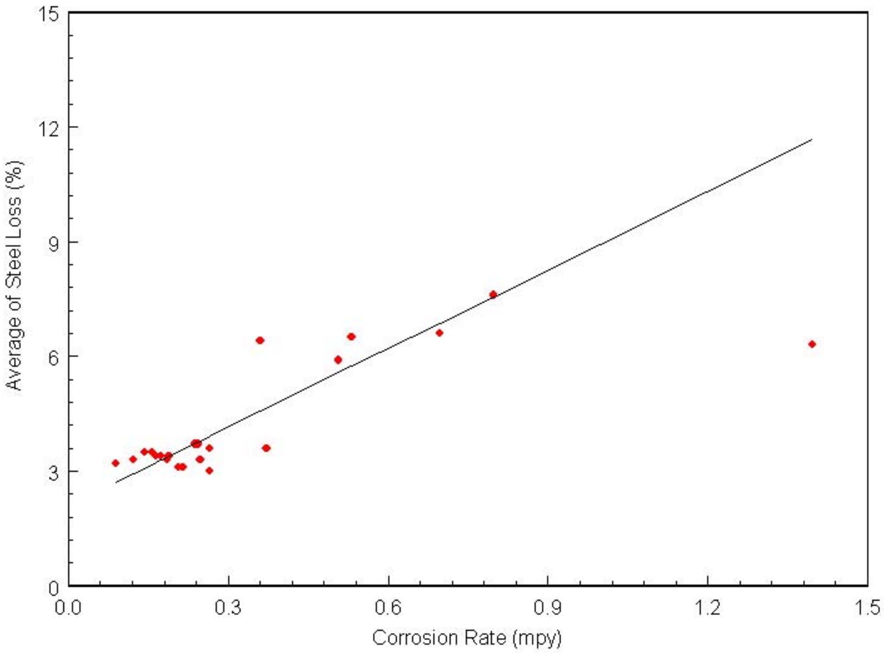


Figure 6.43 Actual Steel Loss vs Corrosion Rate

6.7 Conclusions

Based on the results presented in this chapter, the following conclusions may be drawn.

1. The measured metal loss in wrapped specimens was significantly lower than that in unwrapped controls exposed to the same environment (Fig. 6.40-41). Therefore both CFRP and GFRP are effective in slowing down the rate of corrosion. Although the FRP slowed down the corrosion rate it did not stop corrosion even when four layers were used.

2. The performance of CFRP and GFRP in slowing down the corrosion rate was comparable (Fig. 6.40-6.41). This was also expected from corrosion measurement data that indicated that corrosion rates were similar (Fig. 6.14). The correlation of the corrosion rate with the measured metal loss was reasonable (Fig. 6.43).

3. The level of protection afforded by FRP does not increase with the number of FRP layers. In this study, two layers were found to be the optimal number based on gravimetric testing (Fig. 6.40-6.41). However, more than one layer should be used (See Fig. 6.15, 6.16, 6.24, 6.25)

4. The gravimetric testing method used to determine metal loss is not fully indicative of the severity of the structural impact of the damage as the metal losses are averaged over the exposure length. For example, in the unwrapped controls 3 of the 7 wires in the 7 wire strand broke due to corrosion (Table 6.5) whereas there was only 1 break in 1 wire among all the 16 wrapped specimens (Table 6.6-6.7). (See Fig. 6.43).

5. The epoxy used for the carbon wrap showed better long term bond. There was epoxy failure in only 5 out of 20 tests (Table 6.2) compared to 11 out of 24 tests for the glass wrap (Table 6.3). The bond was weakest in the wet region (4 out of 5 failures in CFRP and 7 out of 11 failures in GFRP). However, since the performance of the CFRP and GFRP from gravimetric testing were comparable, bond may not be as critical a factor for corrosion mitigation applications.

References

6.1 Berver, E., Jirsa, J., Fowler, D., Wheat, H. and Moon, T. (2001). “Effects of Wrapping Chloride Contaminated Concrete with Fiber Reinforced Plastics”. FHWA/TX-03/1774-2, Austin, TX.

6.2 Castro, P., Sagues, A.A., Moreno, E.I. et al (1996). “Characterization of Activated Titanium Solid Reference Electrodes for Corrosion Testing of Steel in Concrete”, Corrosion, Vol. 52, No.8, pp.609-617.

7. SURFACE PREPARATION STUDY

7.1 Introduction

This chapter presents details of a laboratory investigation to evaluate the role of surface preparation on the performance of FRP used for repairing corrosion damaged elements. A brief description of the test program is given in Section 7.2. Details on the accelerated exposure used to corrode specimens are contained in Section 7.3 while information on subsequent repair is in Section 7.4. The setup for the post-wrap accelerated exposure testing is described in Section 7.5. Results from the study are presented in Section 7.6 and the main conclusions summarized in Section 7.7.

7.2 Test Program

The economics of using FRP is strongly influenced by surface preparation. If too much surface preparation is required for the FRP repair to be effective, costs are inevitably higher. If too little surface preparation is carried out and performance is poor, FRP is unlikely to be used. For this reason it is important to establish the role of surface preparation in FRP repair efficiency through strength and gravimetric testing.

The parameters investigated in this study were based on practices used in earlier demonstration projects. In some instances, pre-wrap repairs were kept to a minimum, e.g. only cracks were sealed whereas in others, elaborate procedures for repairing corrosion damage were followed. Additionally, as there had been reports that moisture ingress through the top of a specimen could be detrimental, the effect of sealing the top of a member was investigated. Finally, the performance of full vs partial wrap was evaluated.

A total of 26 prestressed concrete specimens were used in this study (Table 7.1). In addition, another specimen (#11) was used to establish metal loss prior to wrapping through gravimetric testing. This was from the column study reported in Chapter 8.

All specimens were chloride contaminated over a 22 in. length during casting to accelerate corrosion of steel. Ten of the specimens were 6 ft and the remaining sixteen, 5 ft. The 6 ft specimens were earmarked for strength testing at the end of the exposure period while the 5 ft specimens were used for verifying steel loss through gravimetric testing. Four of the specimens were unwrapped controls whereas the remaining twenty two were wrapped using CFRP. In general, two CFRP layers were used.

The specimens were initially subjected to a constant current accelerated corrosion regime for 125 days to attain a targeted metal loss of 25%. Of the 22 specimens that were repaired using FRP, only five specimens - two 6ft (#30, #31) and three 5ft (#62, #63 and #64) were given "full" repairs. In such repairs, the chloride contaminated concrete was removed, the strands cleaned, bonding agents applied and new material used to re-form the section. Subsequently, the repaired section was wrapped using two CFRP layers over 36 in. length in the middle - as in the ambient exposure study (Chapter 6). For the other 17 specimens wrapped, surface preparation was limited to sealing the cracks using epoxy. Different wrapping schemes used are listed in Table 7.1.

Table 7.1 Specimen Details

Specimen Number	**Type**	**Size (ft)**	**Wrap (CFRP)**	**Reforming**	**Concrete Surface Sealing**	**Test Method**
#11	Gravimetric Control	5	No	No	No	Gravimetric
#60	Gravimetric Control	5	No	No	No	Gravimetric
#61	Gravimetric Control	5	No	No	No	Gravimetric
#28	Strength control	6	No	No	No	Strength
#29	Strength control	6	No	No	No	Strength
#62	Full	5	2 layer, 36in	**Yes**	Yes	Gravimetric
#63	Full	5	2 layer, 36in	**Yes**	Yes	Gravimetric
#64	Full	5	2 layer, 36in	**Yes**	**No**	Gravimetric
#65	Minimal	5	1 layer, 36in	No	Yes	Gravimetric
#66	Minimal	5	1 layer, 36in	No	Yes	Gravimetric
#67	Minimal	5	2 layer, 36in	No	Yes	Gravimetric
#68	Minimal	5	2 layer, 36in	No	Yes	Gravimetric
#69	Minimal	5	2 layer, 36in	No	**No**	Gravimetric
#70	Minimal	5	3 layer, 36in	No	Yes	Gravimetric
#71	Minimal	5	3 layer, 36in	No	Yes	Gravimetric
#72	Minimal	5	3 layer, 36in	No	**No**	Gravimetric
#74	Minimal	5	2 layer, 60in	No	Yes	Gravimetric
#75	Minimal	5	2 layer, 60in	No	Yes	Gravimetric
#76	Minimal	5	2 layer, 60in	No	**No**	Gravimetric
#30	Full	6	2 layer, 36in	**Yes**	Yes	Strength
#31	Full	6	2 layer, 36in	**Yes**	Yes	Strength
#32	Minimal	6	2 layer, 36in	No	Yes	Strength
#33	Minimal	6	2 layer, 36in	No	Yes	Strength
#34	Minimal	6	2 layer, 36in	No	**No**	Strength
#35	Minimal	6	2 layer, 72in	No	Yes	Strength
#36	Minimal	6	2 layer, 72in	No	Yes	Strength
#37	Minimal	6	2 layer, 72in	No	**No**	Strength

NOTES: Full: removal of deteriorated concrete, sand blasting, reforming
Minimal: Sealing cracks with epoxy only
Gravimetric Specimen # 11 was also used in the column study (Chapter 8)

7.3 Accelerated Corrosion

All 26 specimens were exposed to a constant current accelerated corrosion scheme. A 110 mA current was impressed for 125 days to attain the 25% targeted steel loss. This current level was selected to limit the current density to 100µA/cm^2 - the measured upper limit in naturally corroding specimens exposed to a marine environment. A soaker hose-sponge system was used to continuously apply moisture to the specimens. The setup is shown in Fig. 7.1. To minimize local corrosion, the impressed current was increased from 10 mA to 110 mA over 6 days. The applied current and voltage was manually monitored. Fig. 7.2 shows the typical variation of voltage with time.

Figure 7.1 Specimens Set-up (L) and Wire Connection (R)

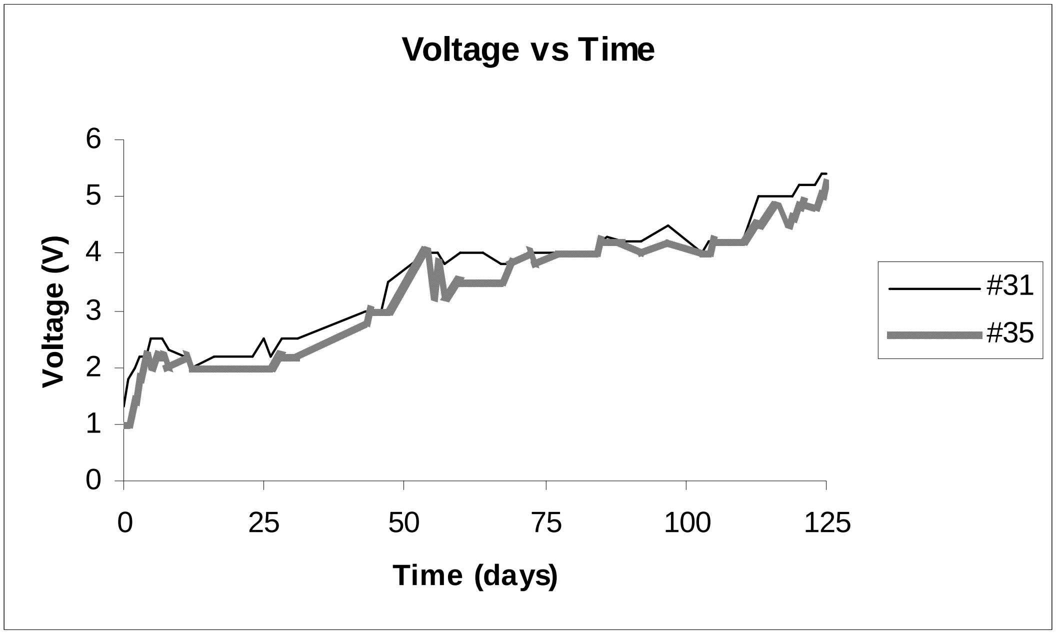


Figure 7.2 Voltage Change

At the end of 125 days accelerated exposure, gravimetric testing was conducted on specimen #11 to establish the actual metal loss. In addition, crack surveys were performed on all 26 specimens that were part of the exposure study. Surface cracks were traced on a transparent plastic sheet (Fig. 7.3) that was subsequently transferred to paper using a magnifying glass and a crack comparator. The averaged crack information for 5 ft and 6 ft specimens is summarized in Table 7.2. Under the same accelerated corrosion regime, 5 ft specimens showed more distress than the 6 ft specimens with a greater number of cracks (34.6 vs 30.4), greater crack length (32.7 in vs 30 in), and maximum crack width (1.4 mm vs 1.2 mm). As expected, most cracks developed along the strands with the largest cracks concentrated in the 22 in. chloride contaminated region. Crack patterns for #60 (5 ft) and #28 (6 ft) specimen are provided in Fig 7.4–7.5. Maximum crack width in all 5 ft and 6 ft specimens are summarized in Tables 7.3–7.4. Crack drawings for the other specimens are presented in Appendix A.

Figure 7.3 Crack Survey after 125 days

Table 7.2 Averaged Crack Information

Length	Number of Specimens	Number of Cracks	Maximum Length (in.)	Maximum Width (mm)
5 ft	16	34.6	32.7	1.4
6 ft	10	30.4	30.0	1.2

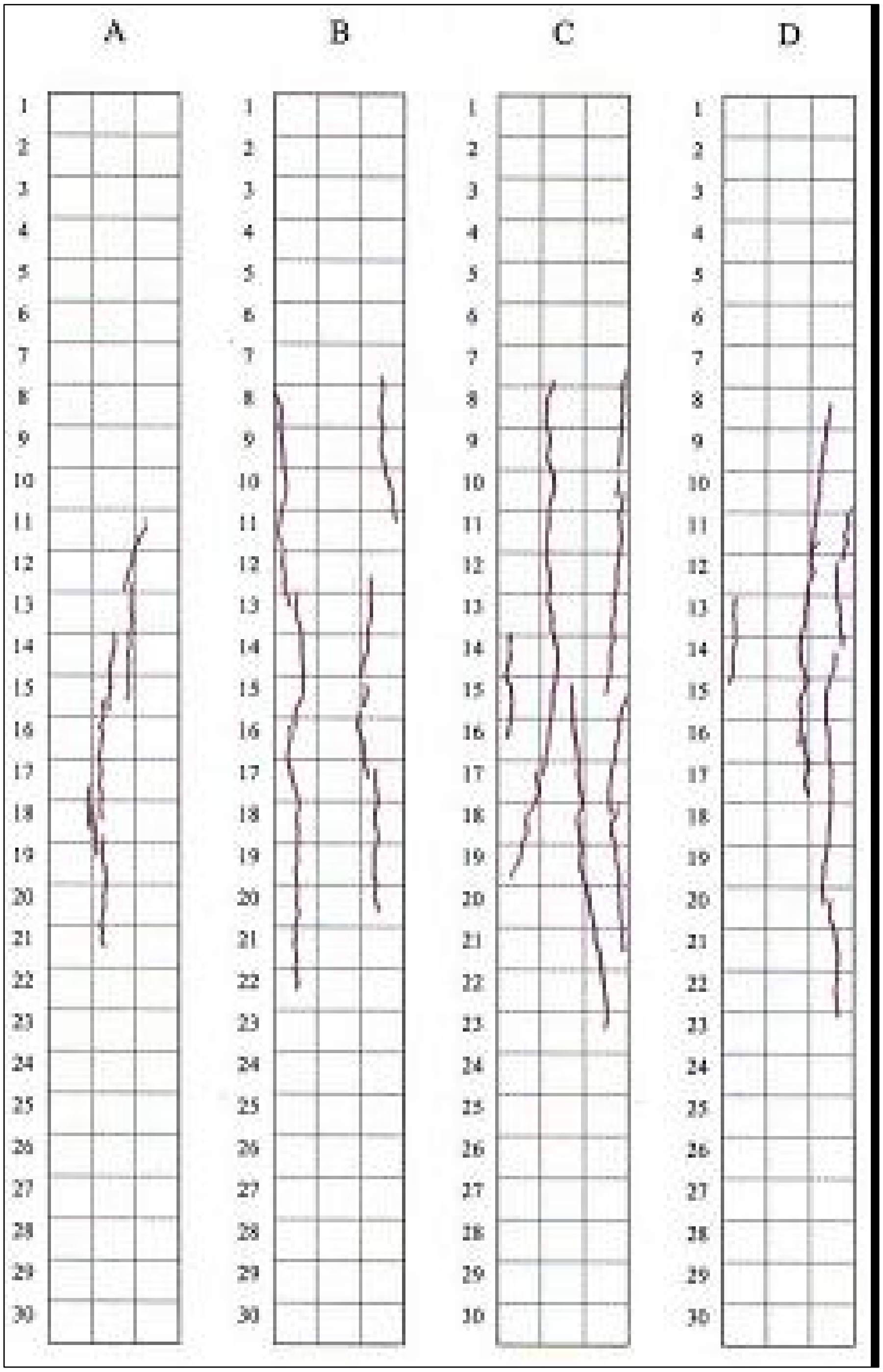


Figure 7.4 Crack Mapping of #60 (5 ft) at 25% of Targeted Steel Loss

Table 7.3 Crack Information of 5 ft Specimens

Specimen	Number of Cracks	Maximum Crack Length (in)	Maximum Crack Width (mm)
#60	33	29.0	0.80
#61	33	33.0	1.25
#62	38	30.5	1.00
#63	40	33.0	1.50
#64	36	32.0	1.25
#65	32	33.5	1.25
#66	31	33.0	1.50
#67	32	34.5	2.00
#68	27	31.0	1.50
#69	29	30.0	1.50
#70	29	34.0	1.50
#71	35	37.0	1.50
#72	48	37.0	1.25
#74	33	36.0	1.50
#75	32	29.5	1.25
#76	46	30.5	1.25

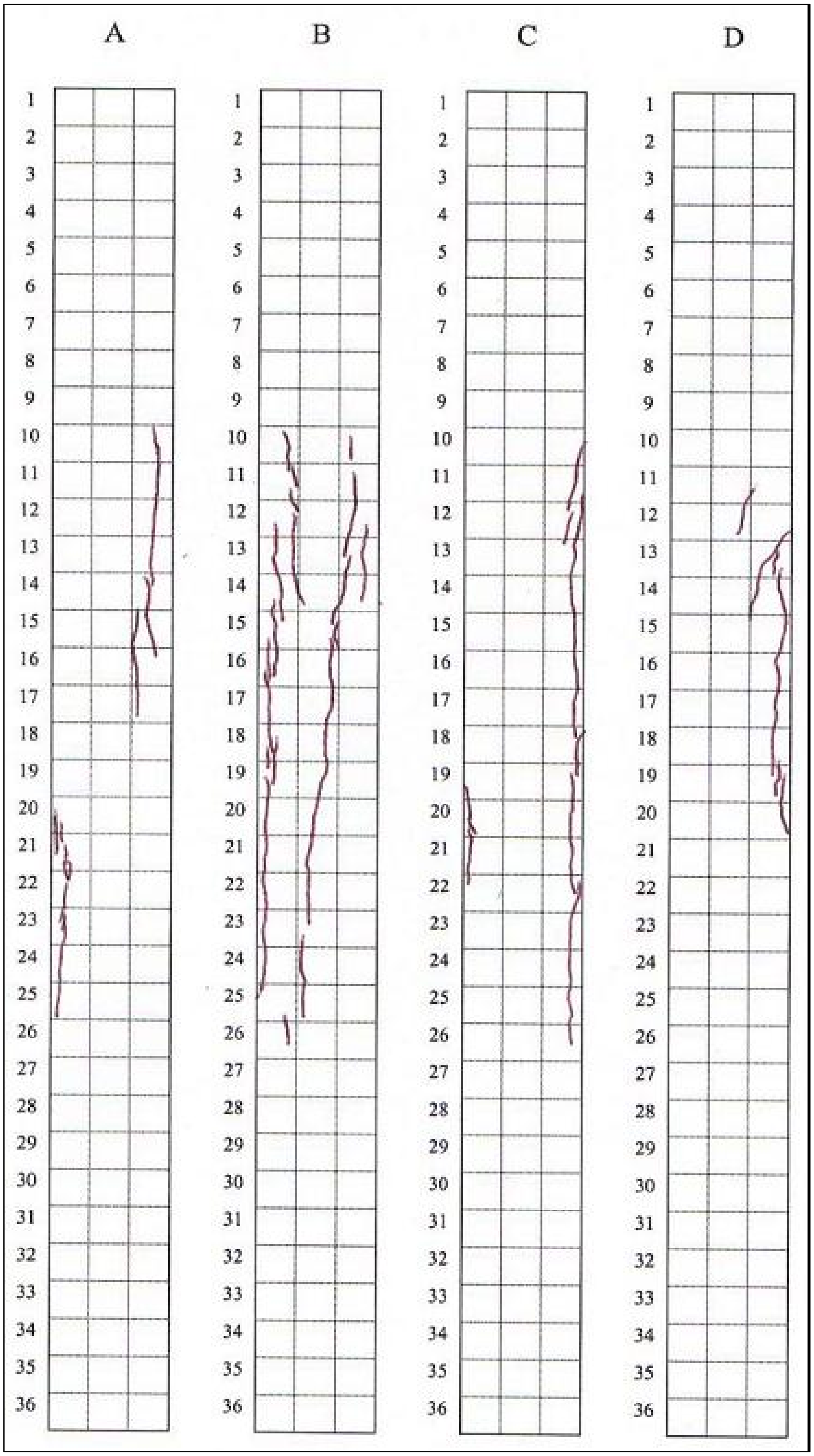


Figure 7.5 Crack Mapping of #28 (6 ft) at 25% Targeted Steel Loss

Table 7.4 Crack Information of 6 ft Specimens

Specimen	Number of Cracks	Maximum Crack Length (in)	Maximum Crack Width (mm)
#28	42	32.0	1.25
#29	25	32.0	1.25
#30	30	31.0	1.25
#31	36	32.0	1.30
#32	19	27.5	0.80
#33	31	28.5	1.00
#34	23	26.5	1.00
#35	43	31.0	1.25
#36	29	32.0	1.00
#37	26	27.5	1.50

7.4 Surface Preparation

A total of 22 specimens including eight 6 ft specimens and fourteen 5 ft specimens were wrapped using carbon fiber. Four other unwrapped specimens served as controls (Table 7.1).

Two types of surface preparation – full and minimal – were evaluated. For full surface preparation, specimens (5) were thoroughly cleaned as required for conventional corrosion repair excepting that chloride contaminated concrete was not removed from under the strands since this could result in failure of the specimens. Deteriorated concrete was removed, embedded strands cleaned, a corrosion inhibitor applied and the section re-formed using repair material. For the majority of the specimens (17), surface preparation was minimal. Only the surface of the concrete was cleaned and all cracks sealed using epoxy. Following FRP wrapping, the concrete surface in selected specimens was sealed (see Table 7.1).

7.4.1 Full Repair

Three 5 ft specimens (#62, #63 and #64) and two 6 ft specimens (#30 and #31) were fully repaired prior to wrapping. The procedure used was as follows:

Deteriorated Concrete Removal

Contaminated concrete was chipped out using an air chisel connected to an air compressor (Fig. 7.6). The delaminated concrete cover was completely removed to expose all the prestressing strands and ties. Some of steel ties were severely corroded and broke off easily.

Figure 7.6 Removing Contaminated Concrete

Cleaning Concrete and Steel

All concrete surfaces and strands were cleaned using sand blasting. Dust and debris were removed by compressed air and strands were cleaned again using acetone (Fig. 7.7).

(a) (b)

Figure 7.7 (a) Sand Blasting; (b) Cleaned Surface

Corrosion Inhibitor Application

After sandblasting, Sika Armatec 110 EpoCem [7.1] manufactured by Sika Corporation was applied as a corrosion inhibitor. The purpose of applying the corrosion inhibitor was to protect the steel from water and chloride penetration. Sika Armatec 110 EpoCem is composed of epoxy-resin (component A), polyamine (component B), and a blend of Portland cements and sands (component C). It acted not only as a corrosion inhibitor, but also as a bonding agent to facilitate bond of the repair material to the existing hardened concrete. The application procedure was as follows (Fig. 7.8).

1. A quarter of component A and a quarter of component B were mixed thoroughly for 30 seconds using a low-speed (400-600 rpm) drill.
2. One bag of component C was slowly added while continuing to mix for 3 minutes. The color of the mixture was concrete grey.
3. The mixed material was applied to the strand and concrete surfaces with a stiff-bristle brush.
4. After the first layer had dried completely (about 2 hours), a second layer was applied.

(a) (b)

(c) (d)

(e) (f)

Figure 7.8 (a) Sika Armatec 110 EpoCem; (b) Mixing Components A and B; (c) Adding Component C; (d) Applying Corrosion Inhibitor; (e) & (f) Completely Coated Specimens

(a) (b)

(c) (d)

(e) (f)

Figure 7.9 (a) Sika MonoTop 611; (b) Mixing; (c) Setting Wooden Form; (d) Pouring Mixed Material; (e) Curing (f) Repaired Specimens

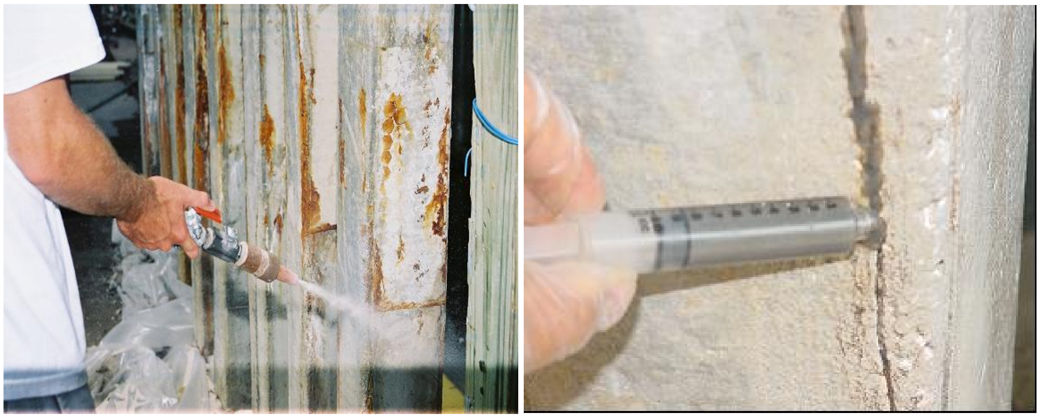
Re-forming Section

Sika MonoTop 611 manufactured by Sika Corporation was used as a patching material for the five cleaned specimens. Sika Mono Top 611 is a silica-fume, polymer-modified Portland cement mortar. It had been successfully applied in previous studies conducted in the state. Its properties as provided by manufacturer are listed in Chapter 3. Wood forms were made for re-forming the cross-section. ¾ in. plywood was used. Four sides of the form were assembled with screws and a hinged opening provided on one side to facilitate pouring of the Sika Mono Top 611. Spray foam was used to seal all the joints and prevent concrete paste from leaking. The following procedure was used (Fig. 7.9).

1. Sika Mono Top 611 and water were thoroughly mixed in a mixer for 3 minutes. One gallon of water was used per bag. The color of the mixture was concrete grey.
2. The mixture was poured into the form and consolidated by tapping the outside of the form with a hammer.
3. Forms were wrapped with a plastic sheet to retain moisture. For the duration of the cure, water was sprayed on the specimens.

7.4.2 Minimal Preparation

Of the 22 specimens wrapped, surface preparation was minimal for 17 specimens including eleven 5 ft and six 6 ft specimens. For these specimens, corrosion products and debris were removed by sand blasting and cracks on the concrete surface sealed using epoxy. A high strength epoxy with a 2 hour cure time was used for this purpose. Syringes were used to inject epoxy into cracks and overflowing epoxy was removed to make concrete surface even. Surface depressions were filled with epoxy.



(a) (b)

Figure 7.10 (a) Sand Blasting on the Surface; (b) Crack Sealing Using Epoxy

7.4.3 CFRP Wrapping

Twenty-two specimens were wrapped using bi-directional carbon fiber supplied by SDR Engineering, Inc. All material properties are presented in Chapter 3. Three 5 ft specimens (#74, #75, #76) and three 6 ft specimens (#35, #36, #37) were fully wrapped with 2 layers. Wrapping was only applied to a 36 in length in the middle for the other specimens. Generally, two layers were used but some specimens were wrapped using 1 or 3 layers to allow possible correlation with the ambient exposure study (Chapter 6).

Wrapping was carried out in accordance with directions [7.2] provided by SDR Engineering, Inc. All specimens were cleaned before wrapping and surfaces and edges of specimens were made smooth using a grinder. And dust and concrete debris produced during the grinding work were removed using compressed air. The unwrapped part of specimen was protected with plastic to prevent epoxy from dripping on its surface during the wrapping operation. The wrapping procedures followed was as follows (Fig. 7.11).

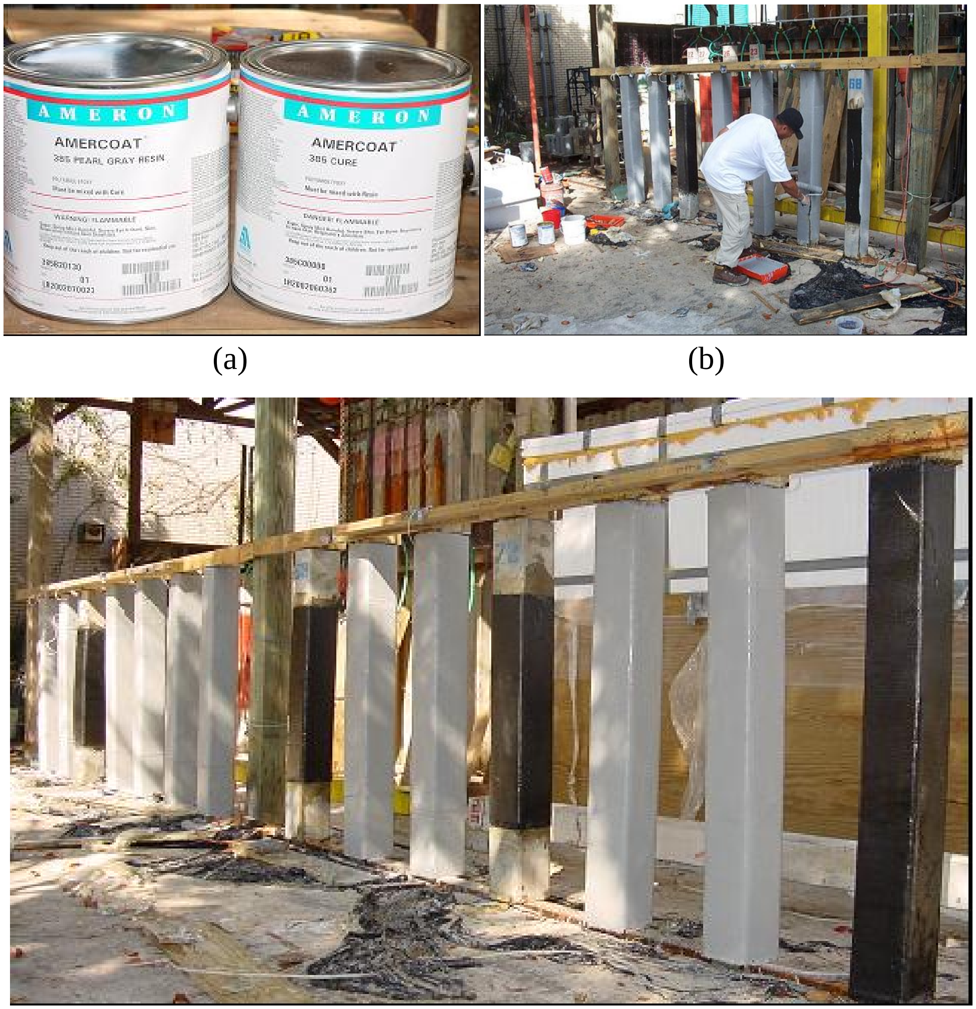
1. CFRP sheet was cut to the required size allowing for a two inch overlap.
2. Resin and hardener were proportioned into a clean dry bucket. Their volume ratio was three to one and they were thoroughly mixed using a stirrer attached to a drill and rotated at a speed of 400 rpm for 3 minutes.
3. Epoxy was uniformly coated on the concrete surface using roller brush and precut CFRP sheets were applied to the concrete using a roller to remove air bubbles. To ensure complete bond between layers, another epoxy coat was applied over installed sheets before applying successive layer of CFRP.

Figure 7.11 Wrapped Specimens

7.4.4 Sealing

Sixteen of the twenty two wrapped specimens were sealed with Amercoat 385. This is manufactured by Ameron Internation and is a two-component sealant. One has a grey color and the other is a yellow. The application procedure was as follows:

1. Clean surfaces of the specimens. The concrete surface was cleaned using a sander and the CFRP surface were cleaned using a brush.
2. The two components were mixed (1:1 by volume) thoroughly using a stirrer installed in a drill. The color of mixed solution was light gray.
3. The material was applied on the entire surfaces of concrete and CFRP of predetermined specimens using a roller. A brush was used for applying coating materials to the holes and edges which roller could not access. It took about one hour to be cured.
4. The second layer of coating material was applied after the first layer had dried.



(a) (b)

(c)

Figure 7.12 (a) Amercoat 385; (b) Applying Material; (c) Sealed & Unsealed Piles

Additionally, to prevent the moisture ingress through the top of a specimen, the concrete surface at the top of sixteen sealed specimens was coated with a high strength epoxy. The others remained unsealed (Fig. 7.13).

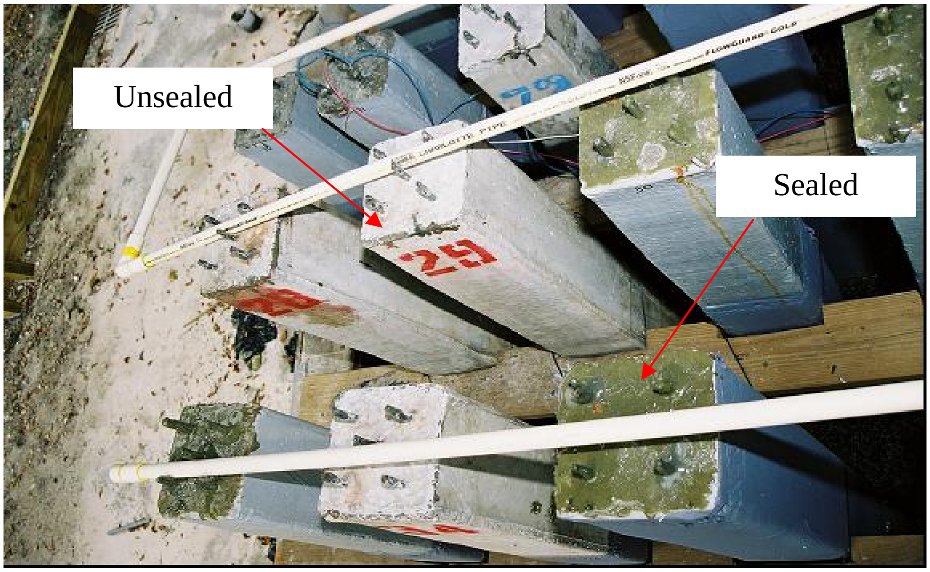


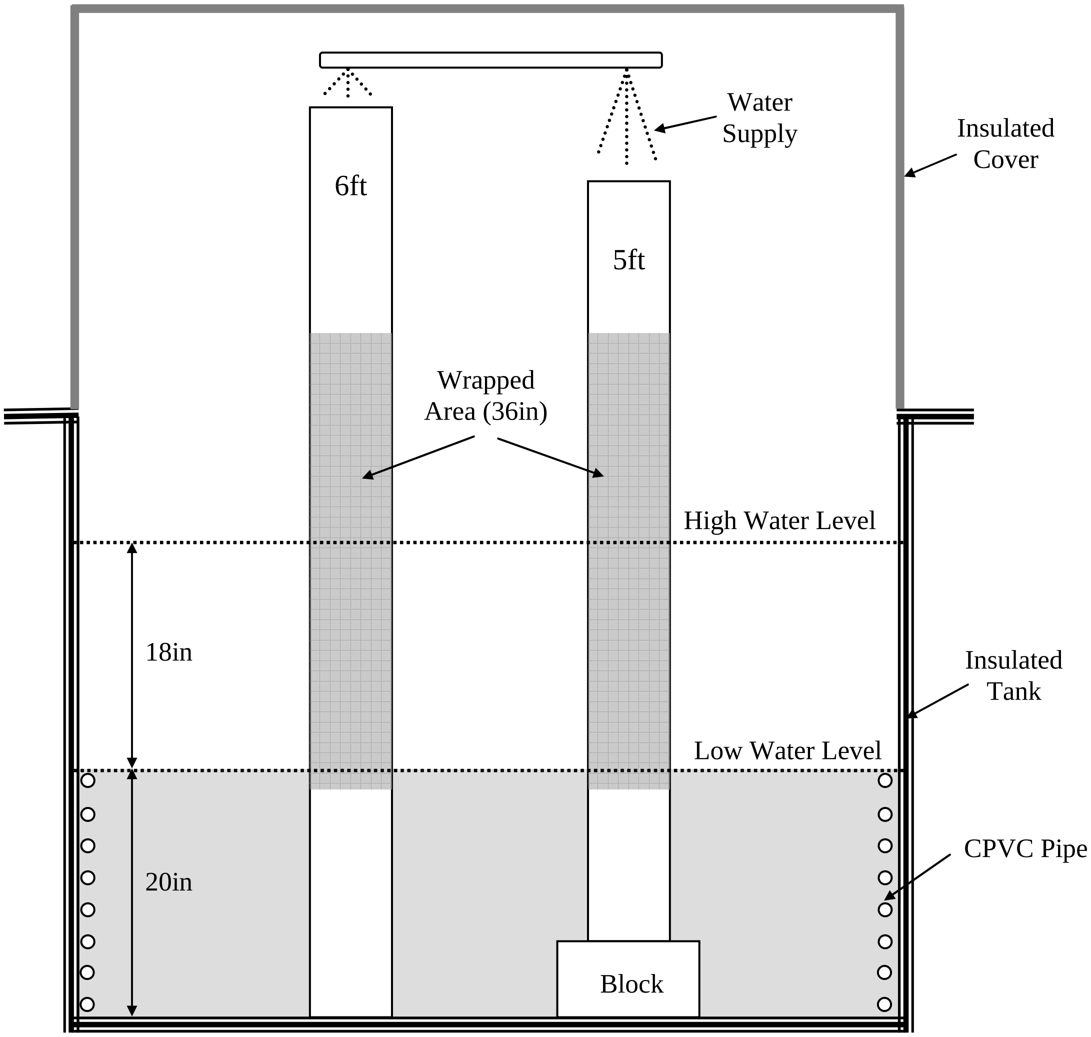
Figure 7.13 Sealing of Concrete Surface on the Top

To protect the CFRP wrap from UV, external latex paint was applied to the wrapping area. The color of the UV paint was grey. The paint was applied on the entire CFRP surface using a brush. It took about 30 minutes for the paint to dry. After the paint had dried, another layer of paint was applied (Fig. 7.14).

Figure 7.14 UV Paint Coated Piles

7.5 Exposure Set-Up

The 26 specimens including 22 wrapped and 4 unwrapped control specimens were placed upright in a 6 ft × 10 ft × 4 ft tank. To accelerate corrosion of the embedded prestressed steel, wet-dry cycles using hot, salt water were used. The targeted temperature of the water was 60 ºC. Actual temperatures were somewhat lower and ranged between 52-60ºC. The water level in the tank was changed every 6 hours as for the ambient study. At high tide, the water level was 32 in (38 in for 6 ft specimens); at low tide it was 14 in. (20 in for 6 ft specimens) – the same as in the ambient study (Chapter 6). A heat exchanger comprising ten CPVC pipes was installed around the inner walls of the tank and circulated hot water. The water level was controlled by a water pump and floating switch. A schematic drawing is shown in Fig. 7.15.



* Water Temperature: 52 – 60 C
* Density: 1.025 (3.5% salt)

Figure 7.15 Setup of Specimens in the Hot Water Tank

The dry cycles affected the lower region of the specimens. In order to investigate the effect of sealing the top of the specimen, a water hose system was set up that allowed hot water to be sprayed from the top. For this purpose, a ¾ in. CPVC (chlorinated polyvinyl chloride) pipe was drilled with 3/16 in. diameter holes that were positioned on top of the specimens (Fig. 7.16a). During the wet cycle, the tank was filled with hot water that was sprayed through these openings until the water level in the tank reached 38 in.

(a) (b)

Figure 7.16 (a) Set Up of Piles; (b) Outside View of Hot Tank System

The accelerated corrosion test started on November 1, 2002. To inspect the status of the specimens, the test was stopped and the tank was uncovered on January 12, 2004. The specimens seemed to be good condition excepting for the unwrapped controls. All pumps, floating switches, and wire connections were replaced. And a new insulation tank cover was built using a steel frame. The test was re-started on February 20, 2004 and ended on March 30, 2005. Accounting for other stoppages due to needed maintenance work, the specimens were exposed for a total of about 850 days (1700 wet/dry cycles).

7.6 Results

When the targeted exposure time was reached, all specimens were taken out of the tank and tests conducted to evaluate their corrosion statues. As shown in Fig. 7.17, unwrapped controls appeared to be severely corroded while the wrapped specimens were in good condition judging from external appearances.

Crack surveys were performed on the four unwrapped controls (two 5 ft and two 6 ft) to determine the progression of cracks due to exposure. Sixteen 5 ft specimens (14 wrapped and 2 controls) were then gravimetrically tested. Eccentric load test was conducted on the remaining ten (8 wrapped and 2 controls) 6 ft specimens.

Figure 7.17 Unwrapped (L) and Wrapped (R) Specimens After the Exposure

7.6.1 Crack Survey Result

Crack surveys were conducted on the two 5 ft (#60, #61) and the two 6 ft controls (#28, #29). Results are summarized in Table 7.5. The size and length of the cracks were very similar in both the 5 ft and 6 ft specimens. The maximum crack width varied from 2.5 mm to 3 mm and the maximum crack length ranged from 35 in. to 39 in.

Table 7.5 Result of Crack Survey on Controls At the End of the Study

Size	**Number**	**Classification (Maximum Value)**	**Before**	**After**	**Increase (%)**
5 ft	#60	Length (in)	29	37	28
		Crack Width (mm)	0.8	3	275
	#61	Length (in.)	33	39	18
		Crack Width (mm)	1.25	3	140
6 ft	#28	Length (in.)	32	35	9
		Crack Width (mm)	1.25	3	140
	#29	Length (in.)	32	37	16
		Crack Width (mm)	1.25	2.5	100

○ : Location of the Maximum Crack

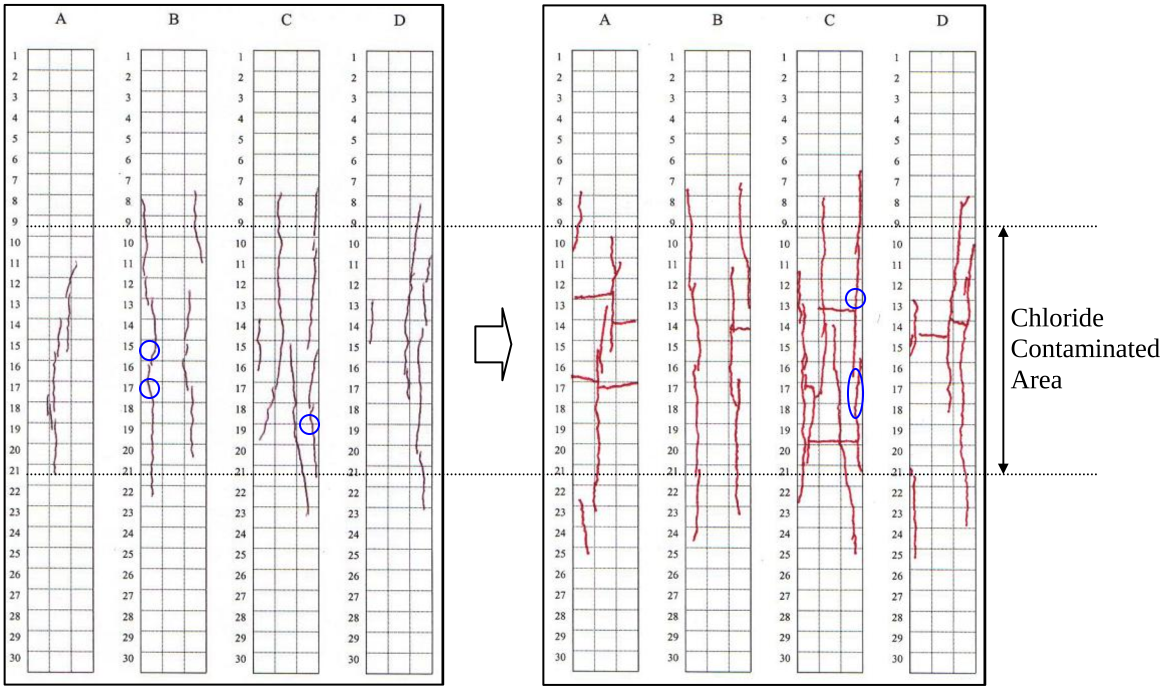


Figure 7.18 Propagation of Cracks in #60 Specimen Before (L) and After (R) Accelerated Hot Water Simulated Cycles

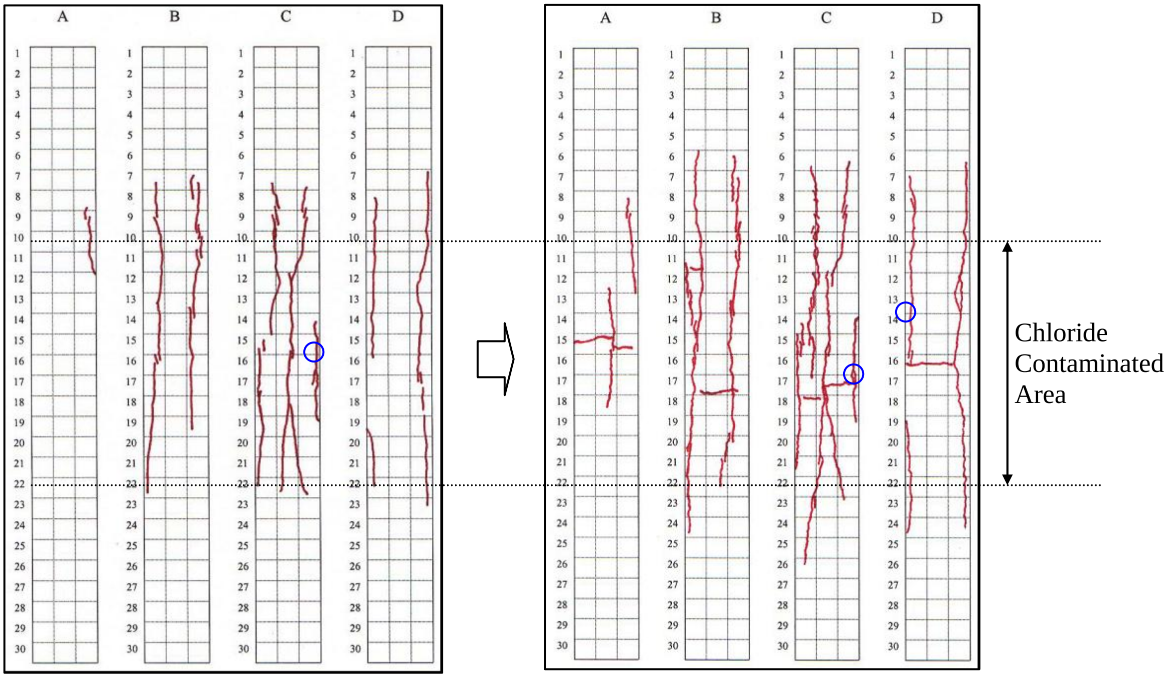


Figure 7.19 Propagation of Cracks in #61 Specimen Before (L) and After (R) Accelerated Hot Water Simulated Cycles

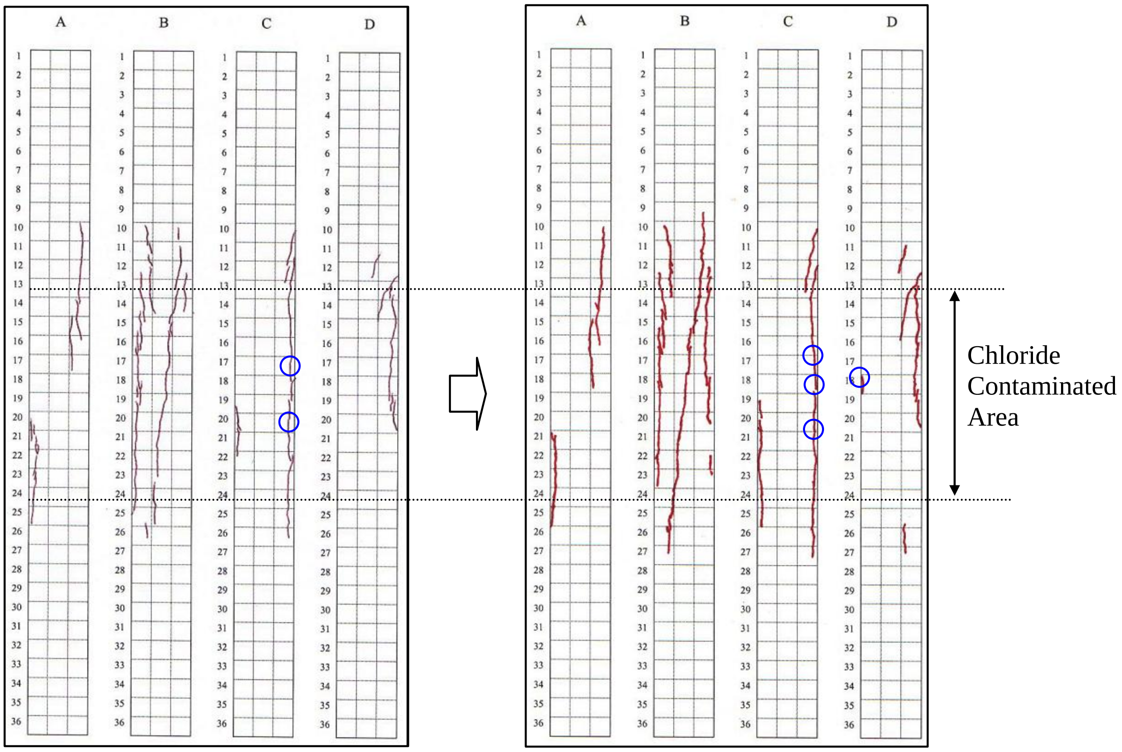


Figure 7.20 **P**ropagation of Cracks of #28 Specimen Before (L) and After (R) Accelerated Hot Water Simulated Cycles

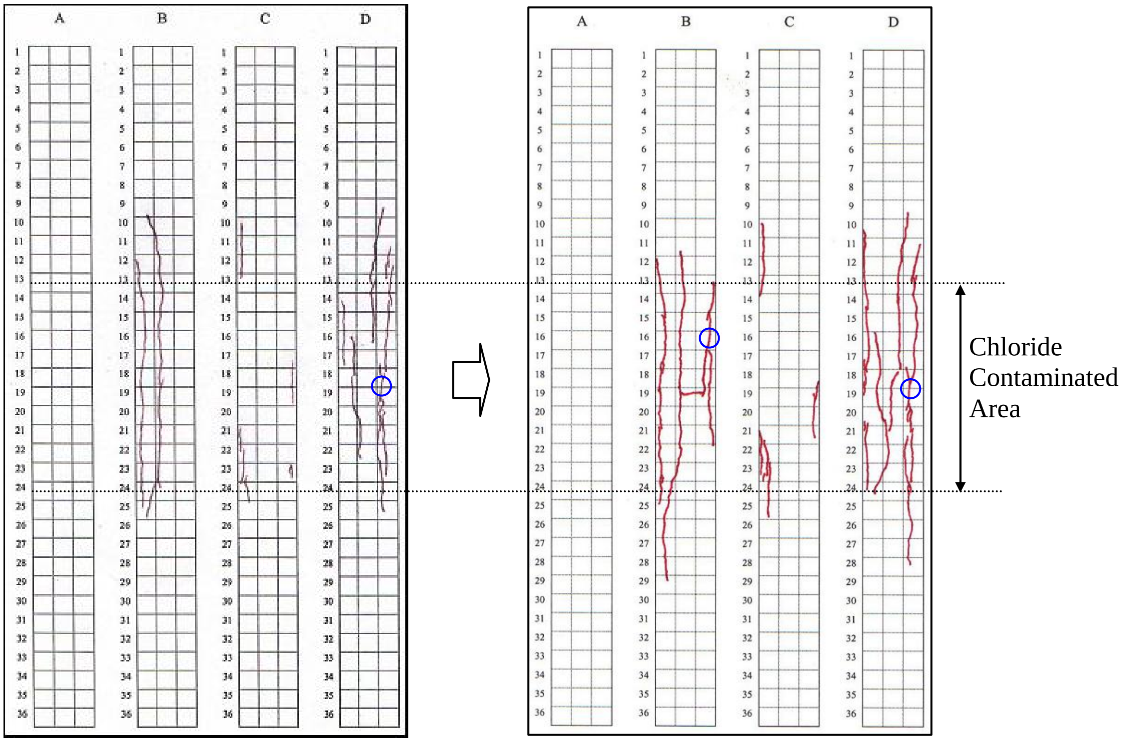


Figure 7.21 Propagation of Cracks of #29 Specimen Before (L) and After (R) Accelerated Hot Water Simulated Cycles

Fig. 7.18–7.21 show the change in crack pattern on the unwrapped control specimens before and after exposure to simulated hot water tidal cycles. As expected, cracks were concentrated in the middle (the chloride contaminated region) and propagated to the lower part of the specimens. Transverse cracks were found on every face of the 5 ft specimens and the concrete surface was delaminated.

7.6.2 Eccentric Load Test Result

A total of ten 6 ft specimens including two unwrapped controls and eight wrapped ones were tested under eccentric load to establish strength loss due to exposure. Testing was carried as described in Section 8.3. This section only presents the test results. The compressive strength of concrete measured right before the eccentric load test was 9 ksi and 7.8 ksi respectively for regular concrete and chloride contaminated concrete. A plot of the concrete variation with time is included in Appendix B (Fig. B.37).

7.6.2.1 Unwrapped Controls (#28 & #29)

Fig. 7.22-7.23 shows the setup and the failure mode for the two unwrapped controls (#28 and #29). Both specimens failed in the middle area and their failure modes were similar. The ultimate load capacities of #28 and #29 specimens were 61.7 kips and 61.4 kips, respectively compared to their original undamaged failure load of 88.6 kips (Table 8.8). From the failed section, it appeared that very little of the ties remained intact and most strands were completely corroded in the middle of specimens. Fig. 7.24 shows a plot of the lateral deflection with load at mid span for both the specimens. Plots showing the strain variation with load are presented in Appendix B.

Figure 7.22 Failure of Unwrapped Control #28

Figure 7.23 Failure of Unwrapped Control #29

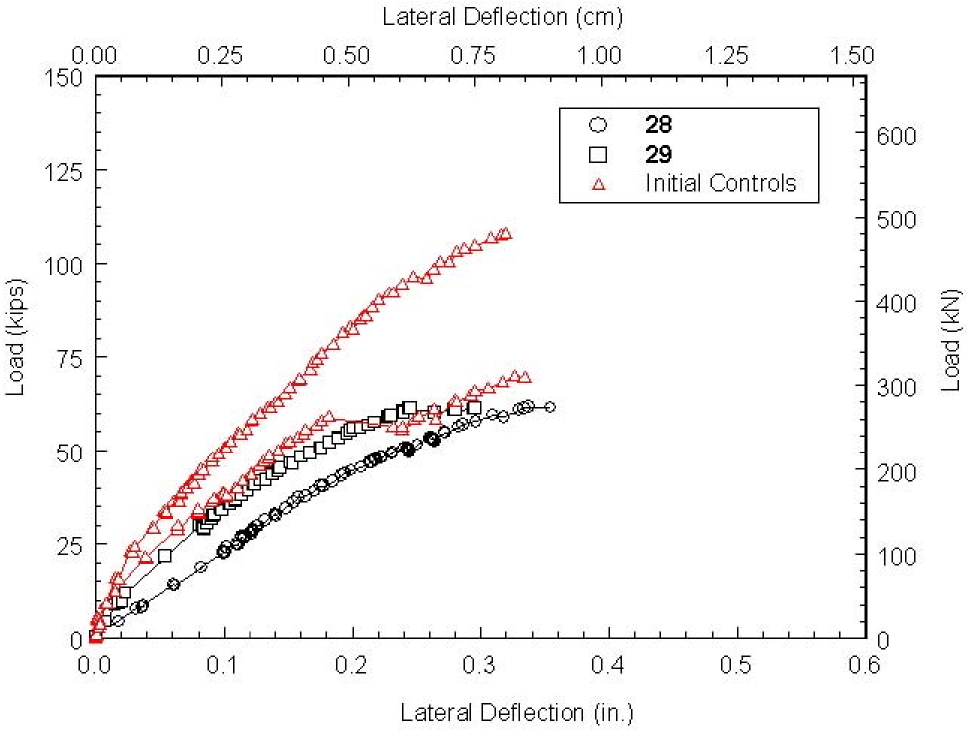


Figure 7.24 Load vs Deflection Plot for Unwrapped Control Specimens

7.6.2.2 Full Repair / 2 layers / 36in (#30 & #31)

As explained in Section 7.2, two 6 ft specimens (#30 and #31) were fully repaired before CFRP wrapping. The deteriorated concrete was removed, corroded steel cleaned and coated with a corrosion inhibitor, and special patching material applied to re-form the cross-section. After the patch had cured, the two specimens were wrapped in the middle with 2 layers of CFRP. The exposed concrete was sealed.

The test set up and failure modes for specimens #30 and #31 are shown in Fig. 7.25-26. In both cases, premature failure occurred unexpectedly at the ends.

The plot of the lateral deflection with load at mid span is presented in Fig. 7.27. The maximum loads were 79.1 kips and 106.4 kips, respectively.

Figure 7.25 Failure of Specimen #30 (Full Repair, 36 in Wrapped, Sealed)

Figure 7.26 Failure of Specimen #31 (Full Repair, 36 in. Wrapped, Sealed)

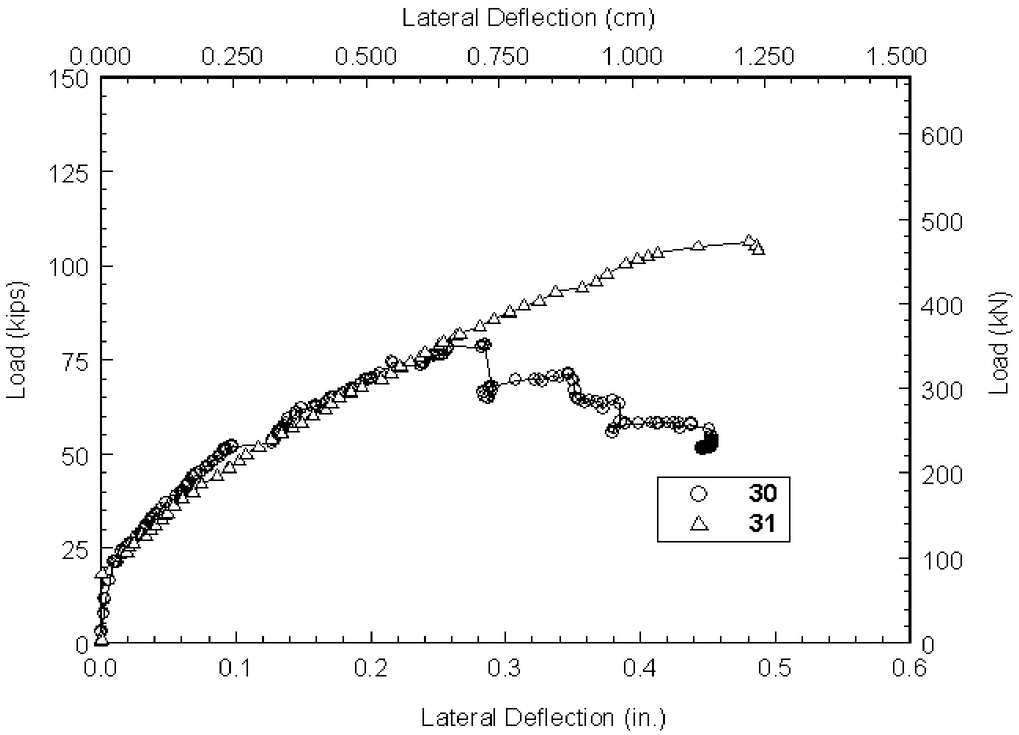


Figure 7.27 Load vs Deflection Plot for Full Repair and Wrapped with 36 in CFRP

7.6.2.3 Minimal Repair / CFRP / 2 layers / 36in. (#32, #33 and #34)

In these specimens only cracks on the surface concrete were sealed with epoxy prior to wrap with two layers of CFRP on the center (36 in length). The exposed concrete surface of specimens #32 and #33 was sealed (Section 7.4.4) while specimen #34 was left unsealed.

Fig. 7.28–30 shows the failure modes of #32, #33 and #34 specimens. As in the previous case, end failure occurred in specimens #32 and #33 at 97.1 kips and 87.3 kips, respectively. However, specimen #34 failed at mid span at an ultimate load of 96.2 kips as shown in Fig 7.29. Exposed ties appeared to have corroded completely but strands were intact.

The plot of mid-span lateral deflection with load for all three specimens is shown in Fig. 7.31. There was little difference in the response of sealed and unsealed specimens.

Figure 7.28 Failure of #32 Specimen (Minimal Repair, 36 in Wrap, Seal)

Figure 7.29 Failure of #33 Specimen (Minimal Repair, 36 in Wrap, Seal)

Figure 7.30 Failure of #34 Specimen (Minimal Repair, 36 in Wrap, Unseal)

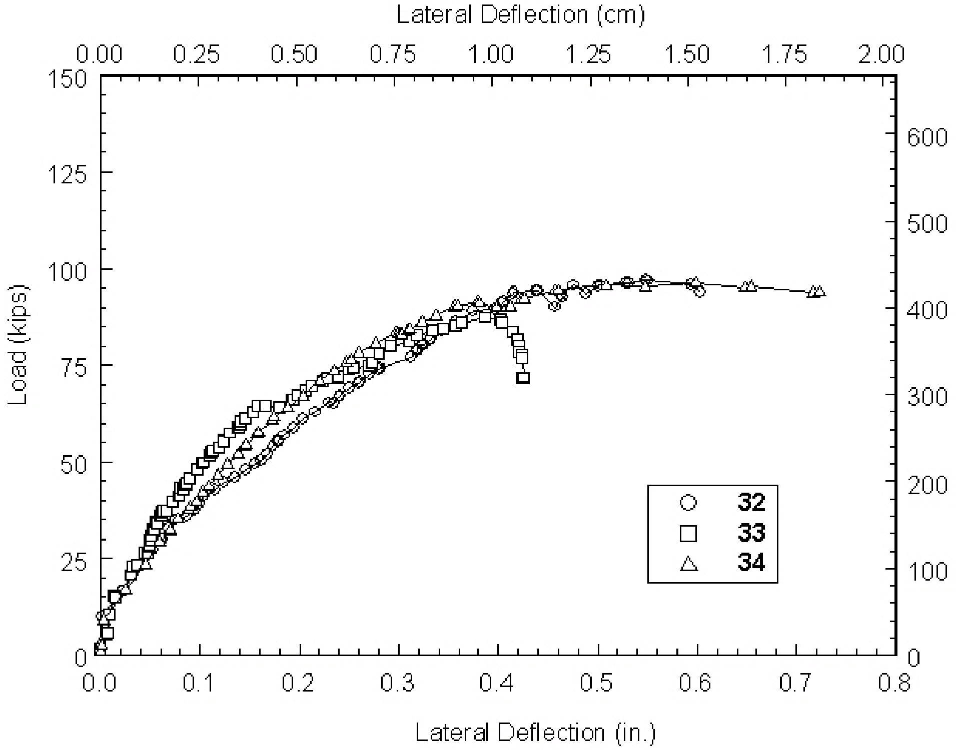


Figure 7.31 Load vs Deflection Plot for Minimal Repaired and Wrapped with 36in CFRP

7.6.2.4 Minimal Repair / CFRP / 2 layers / 72in (#35, #36 and #37)

These specimens were identical to the ones reported in the previous section excepting that the CFRP wrap was applied over the entire length. The exposed concrete in specimens #35 and #36 was sealed while that in #37 specimen was not sealed.

The failure mode of the tested specimens is shown in Fig. 7.32–34. Failure occurred in the chloride contaminated region at mid span in all three specimens. The CFRP was ruptured in the lateral direction on the tension side and it ripped in both the lateral and longitudinal directions on the compression side. The ultimate load capacity was 96.6 kips, 84.5 kips and 88.5 kips for #35, #36 and #37, respectively. The sealing appeared to have little or no effect on strength.

Fig. 7.35 shows variation in the mid-span lateral deflection with load in all three specimens. In contrast to specimens wrapped over a 36 in. length, the fully wrapped specimens showed larger deformation at failure. Plots showing the strain variation with load are presented in Appendix B.

Figure 7.32 Failure of #35 Specimen (Minimal Repair, 72 in Wrap, Seal)

Figure 7.33 Failure of #36 Specimen (Minimal Repair, 72 in Wrap, Seal)

Figure 7.34 Failure of #37 Specimen (Minimal Repair, 72 in Wrap, Unseal)

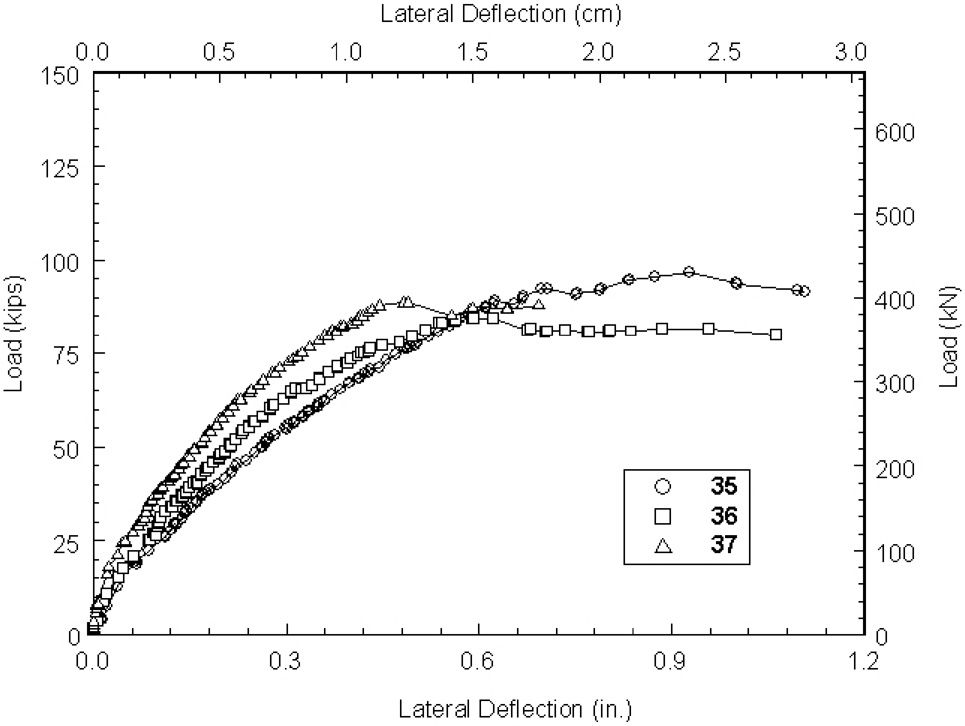


Figure 7.35 Load vs Deflection Plot for Minimal Repaired and Wrapped with 72 in CFRP

7.6.2.5 Summary of Eccentric Load Test

The ultimate load capacities for all the specimens are summarized in Table 7.6. Considering that the average ultimate load capacity of unwrapped specimen before the exposure of hot-water corrosion acceleration was 88.6 kips (Table 8.8), the ultimate load capacity of the unwrapped control specimens decreased by 30.6%. However, the corresponding averaged capacities of the wrapped piles exceeded their original capacity despite premature failure at the ends in specimens #30, #31, #32 and #33 (Fig. 7.36).

This exposure was extremely severe since specimens were subjected to a steamy, high temperature environment for over 2 years. Based on the result of load test, it could be said that FRP bond was effective in increasing the axial load capacity of corroded piles despite the extreme exposure. The highest capacity was attained with full repair. The loads could have been higher but for the end failure. However, the epoxy seal did not affect strength. Full wrap did not increase capacity but improved ductility.

Table 7.6 Summary of Eccentric Load Test on Exposed Specimens

Type		**Identifier**	**Ultimate Load (kips)**	**Increase (%)**
Unwrapped Control		#28	61.7	-30.4
		#29	61.4	-30.7
Average			**61.5**	**-30.6**
Full Repair 36 in CFRP Sealed		#30	79.1	End failure
		#31	106.4	End failure
Average			**92.7**	**4.7**
Minimal Repair 36 in CFRP	Sealed	#32	97.1	End failure
		#33	87.3	End failure
	Average		**92.2**	**4.0**
	Unsealed	#34	96.2	8.6
Minimal Repair 72in CFRP	Sealed	#35	96.6	9.1
		#36	84.5	-4.7
	Average		**90.5**	**2.2**
	Unsealed	#37	88.5	-0.1

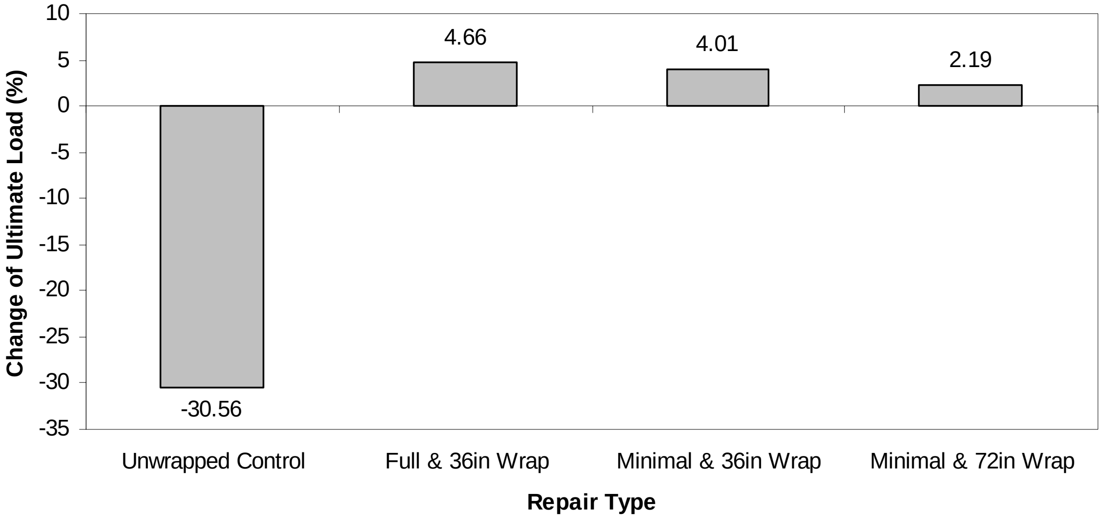


Figure 7.36 Change in Ultimate Load Capacity After Exposure to Simulated Cycles in Hot Salt Water

7.6.3 Gravimetric Test Result

A total of sixteen 5 ft specimens including 14 wrapped and 2 unwrapped controls were gravimetrically tested to measure the actual steel loss due to corrosion and to evaluate the effectiveness of different repair methods. Of the 14 wrapped specimens, three were wrapped over the entire length while the remaining 11 were wrapped over 3 ft. In gravimetric testing, the surface concrete was first removed to measure the distribution of corrosion products and then strands and ties were retrieved. The retrieved strands were cut to 3 ft length. The seven wires that make a strand were carefully disassembled (Fig. 7.37) and after each wire had been cleaned using a wire brush, the strand was re-assembled and its weight accurately measured to determine metal loss.

Figure 7.37 Retrieved Steel Cage and Disassembled Wires

Since the chloride contaminated length was 22 in. all metal losses reported are averaged over this length for consistency. The crack pattern (Fig. 7.18-7.21) indicates that some corrosion occurred outside this region. Thus, the average metal loss reported will be slightly higher.

7.6.3.1 Unwrapped Controls (#60 & #61)

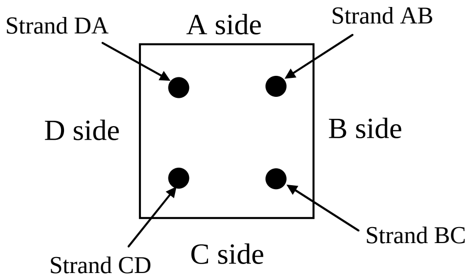
The unwrapped controls were severely corroded and the concrete surface delaminated. The strands and ties in the middle section in both specimens were completely corroded as shown in Fig.7.38. This is shown more clearly in Fig. 7.39 which shows the eight retrieved strands and ties from both these specimens

The results from the gravimetric test for these unwrapped controls are summarized in Table 7.7. The strands are identified as AB, BC, CD and DA in accordance with convention shown in the table. The maximum loss in a strand was 86.5 % (#61-BC) while the maximum loss in the tie was 87.4% (#60). The averaged steel losses of strands in #60 and #61 specimens were 82.3 % and 77.9%, respectively.

Figure 7.38 Distribution of Corrosion Product in #60 (L) and #61 (R) Specimens

Figure 7.39 Retrieved Strands and Ties From Unwrapped Controls

Table 7.7 Results of Gravimetric Test for Controls (#60 and #61)

Name	Steel	Break in Strand Wires	22 in.			Nomenclature for Strand
			Original Weight (g)	Lost weight (g)	Percent Loss	
#60	AB	6+1	168.8	136.8	81.0	
	BC	6+1	168.8	145.2	86.0	
	CD	6+1	168.8	142.5	84.4	
	DA	6+1	168.8	131.3	77.8	
	Ave		**168.8**	**139.0**	**82.3**	
	Tie	N/A	332.3	290.4	87.4	
#61	AB	6+1	168.8	120.7	71.5	
	BC	6+1	168.8	146	**86.5**	
	CD	6+1	168.8	142.4	84.4	
	DA	6+1	168.8	117	69.3	
	Ave.		**168.8**	**131.5**	**77.9**	
	Tie	N/A	332.3	289.2	87.0	

Note: Where the central wire in a 7-wire strand was broken, it is reported in the form a+1 where a signifies the number of other wires broken. All such breaks occurred in the middle region of the specimen

7.6.3.2 Full Repair / CFRP / 2 layers / 36in (#62, #63 and #64)

In these specimens, deteriorated concrete was removed, the strands completely cleaned, coated with corrosion inhibitor, patched, and then wrapped with 2 layers of CFRP over a 36 in length. Exposed concrete in specimens #62 and #63 were sealed while #64 was not. As shown in Fig. 7.40, patching material and corrosion inhibitor were still clinging to the strands and there was little corrosion product around the test area.

Figure 7.40 Distribution of Corrosion Product in #62 (L), #63 (M) and #64 (R)

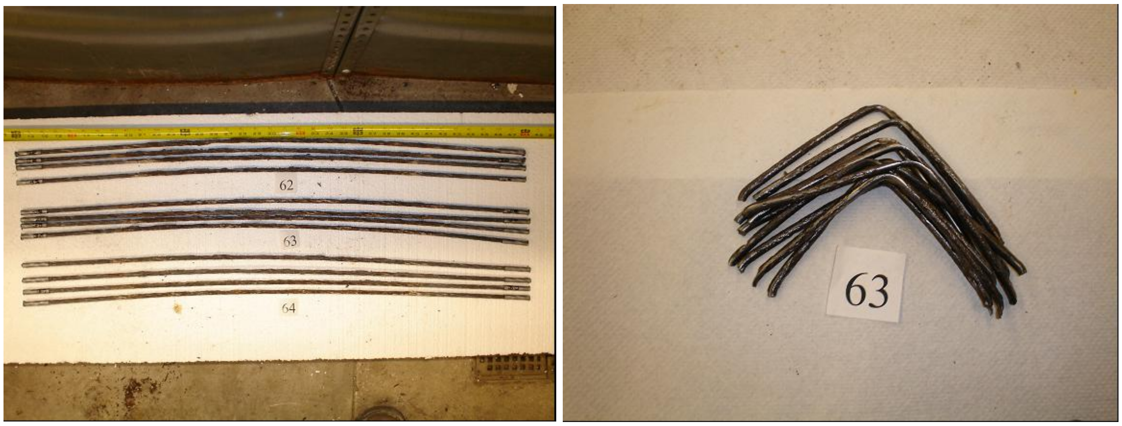


Figure 7.41 Retrieved Strands and Ties From #62, #63 and #64

Table 7.8 Results of Gravimetric Test for Full Repair 2 layer 36 in. (#62, #63 and #64)

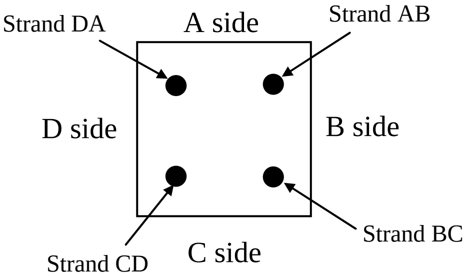
<table>
<tr><th rowspan="2">Name</th><th rowspan="2">Steel</th><th rowspan="2">Break in Strand Wires</th><th colspan="3">22 in.</th><th rowspan="2">Nomenclature of Strands</th></tr>
<tr><th>Original Weight (g)</th><th>Lost weight (g)</th><th>Loss Ratio (%)</th></tr>
<tr><td rowspan="6">#62
(S)</td><td>AB</td><td>4</td><td>168.8</td><td>39.1</td><td>23.2</td><td rowspan="18">
</td></tr>
<tr><td>BC</td><td>6</td><td>168.8</td><td>36.3</td><td>21.5</td></tr>
<tr><td>CD</td><td>4</td><td>168.8</td><td>34.4</td><td>20.4</td></tr>
<tr><td>DA</td><td>3</td><td>168.8</td><td>38.7</td><td>22.9</td></tr>
<tr><td>Ave.</td><td></td><td>168.8</td><td>37.1</td><td>22.0</td></tr>
<tr><td>Tie</td><td>N/A</td><td>332.3</td><td>N/A</td><td>N/A</td></tr>
<tr><td rowspan="6">#63
(S)</td><td>AB</td><td>3</td><td>168.8</td><td>38.5</td><td>22.8</td></tr>
<tr><td>BC</td><td>0</td><td>168.8</td><td>31.9</td><td>18.9</td></tr>
<tr><td>CD</td><td>1</td><td>168.8</td><td>38.5</td><td>22.8</td></tr>
<tr><td>DA</td><td>2</td><td>168.8</td><td>39.5</td><td>23.4</td></tr>
<tr><td>Ave.</td><td></td><td>168.8</td><td>37.1</td><td>22.0</td></tr>
<tr><td>Tie</td><td>N/A</td><td>332.3</td><td>83.3</td><td>25.1</td></tr>
<tr><td rowspan="6">#64
(U)</td><td>AB</td><td>2</td><td>168.8</td><td>35.3</td><td>20.9</td></tr>
<tr><td>BC</td><td>5</td><td>168.8</td><td>36.3</td><td>21.5</td></tr>
<tr><td>CD</td><td>0</td><td>168.8</td><td>32.8</td><td>19.4</td></tr>
<tr><td>DA</td><td>0</td><td>168.8</td><td>35.7</td><td>21.1</td></tr>
<tr><td>Ave.</td><td></td><td>168.8</td><td>35.0</td><td>20.7</td></tr>
<tr><td>Tie</td><td>N/A</td><td>332.3</td><td>N/A</td><td>N/A</td></tr>
</table>

Fig. 7.41 shows the retrieved strands and ties. The total measured steel loss is summarized in Table 7.8. The maximum steel loss in the strand was 23.4% (#63-DA) and it was 25.1% in the ties. The average steel loss in the strands was 22%, 22% and 20.7% in #62, #63, and #64, respectively. The metal loss in the unsealed specimen (#64) was slightly smaller. Note N/A in Table 7.8 for ties signifies that the ties had corroded before wrapping.

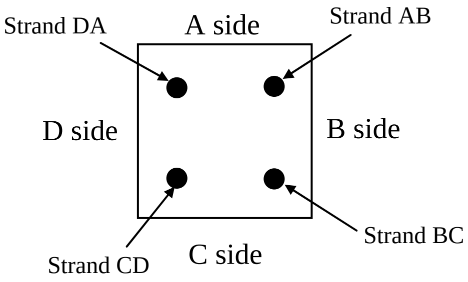
7.6.3.3 Minimal Repair / CFRP / 1 layer / 36in (#65 and #66)

In these specimens, the only cracks were filled with epoxy. One layer of CFRP was used to wrap over 36 in. The corrosion products were distributed over the 22 in. chloride contaminated region (Fig. 7.42). The maximum metal loss in the strands was 27.3 % (#66-BC) and it was 23.9% (#65) in the ties (Table 7.9). The average steel loss in the strands in #65 and #66 were 24.2% and 25.3 % respectively. Many wires in the strands were completely corroded. This was especially the case for specimen #65 in which 93% of the wires in the strands were broken due to corrosion and #66 where 61% of the wires were broken.

Figure 7.42 Distribution of Corrosion Product in #65 (T) and #66 (B)

Figure 7.43 Retrieved Strands and Ties From #65 and #66

Table 7.9 Results of Gravimetric Test for Minimal 1 layer 36 in (#65 and #66)

Name	Steel	Break in Strand Wires	22 in			Nomenclature of Strands
			Original Weight (g)	Lost weight (g)	Loss Ratio (%)	
#65 (S)	AB	6	168.8	43.8	25.9	
	BC	6+1	168.8	37.3	22.1	
	CD	6+1	168.8	37.7	22.3	
	DA	6	168.8	44.7	26.5	
	Ave.		**168.8**	**40.9**	**24.2**	
	Tie	N/A	332.3	79.4	**23.9**	
#66 (S)	AB	5	168.8	42.6	25.2	
	BC	6+1	168.8	46.1	**27.3**	
	CD	0	168.8	36.2	21.4	
	DA	5	168.8	45.6	27.0	
	Ave.		**168.8**	**42.6**	**25.3**	
	Tie	N/A	332.3	75.5	22.7	

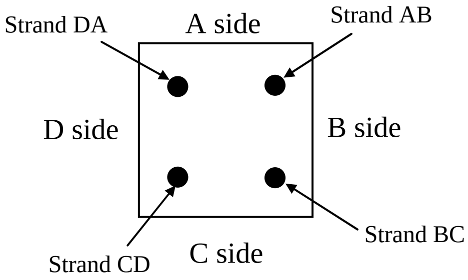
7.6.3.4 Minimal Repair / CFRP / 2 layers / 36in (#67, #68 and #69)

This set was similar to the previous set excepting that instead of one layer, two CFRP layers were used. In addition, a third specimen, #69 was not sealed with epoxy. The maximum loss in the strands was 24.1 % (#67-AB) and it was 24.4% (#69) in the ties (Table 7.10). The average steel loss in the strands was 22.3% (#67), 20.8% (#68) and 20.3% (#69). The steel loss in the sealed specimens was marginally higher than that in the unsealed specimen. In the unsealed specimen, 61% of wires in strands were broken, and 36% (#67) and 46% (#68) of wires were broken in sealed specimens.

Figure 7.44 Distribution of Corrosion Product in #67 (T), #68 (M) and #69 (B)

Figure 7.45 Retrieved Strands and Ties From #67, #68 and #69

Table 7.10 Result of Gravimetric Test for Minimal 2 layer 36 in (#67, #68 and #69)

Name	Steel	Broken Wires	22 in			Nomenclature of Strands
			Original Weight (g)	Lost weight (g)	Loss Ratio (%)	
#67 (S)	AB	4	168.8	40.6	**24.1**	
	BC	2	168.8	37.5	22.2	
	CD	0	168.8	33.6	19.9	
	DA	4	168.8	39.1	23.2	
	Ave.		**168.8**	**37.7**	**22.3**	
	Tie	N/A	332.3	76.9	23.1	
#68 (S)	AB	2	168.8	37	21.9	
	BC	4	168.8	33.6	19.9	
	CD	3	168.8	34.1	20.2	
	DA	4	168.8	35.9	21.3	
	Ave.		**168.8**	**35.2**	**20.8**	
	Tie	N/A	332.3	77.1	23.2	
#69 (U)	AB	4	168.8	34.3	20.3	
	BC	4	168.8	35.4	21.0	
	CD	4	168.8	32.8	19.4	
	DA	5	168.8	34.4	20.4	
	Ave.		**168.8**	**34.2**	**20.3**	
	Tie	N/A	332.3	81	**24.4**	

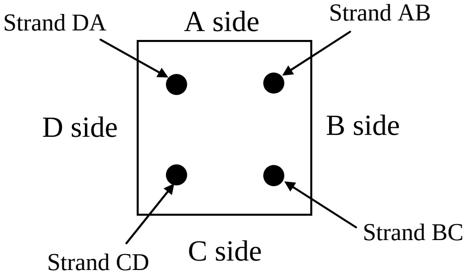
7.6.3.5 Minimal Repair / CFRP / 3 layers / 36 in. (#70, #71 and #72)

This set was identical to the previous set excepting that three CFRP layers were bonded to epoxy repaired specimens. The maximum loss in the strands was 34.4 % (#71-DA) and it was 24.% (#70) in the ties (Table 7.11) The average steel loss in the strands was 22.4% (#70), 27.6% (#71) and 21% (#72). The steel loss in the sealed specimens (#70, #71) was higher than in the unsealed specimen (#72). Forty-three and Eighty-six percent of wires were disconnected in #43 and #71, respectively. In unsealed pile, 57% of wires were broken.

Figure 7.46 Distribution of Corrosion Product in #70 (T), #71 (M) and #72 (B)

Figure 7.47 Retrieved Strands and Ties From #70, #71 and #72

Table 7.11 Results of Gravimetric Test for Minimal 3 layer 36 in. (#70, #71 and #72)

Name	Steel	Break in Strand Wires	22 in			Nomenclature of Strands
			Original Weight (g)	Lost weight (g)	Loss Ratio (%)	
#70 (S)	AB	4	168.8	41.1	24.3	
	BC	1	168.8	39.5	23.4	
	CD	3	168.8	33.3	19.7	
	DA	4	168.8	37.6	22.3	
	Ave.		**168.8**	**37.9**	**22.4**	
	Tie	N/A	332.3	82.1	**24.7**	
#71 (S)	AB	4	168.8	37.8	22.4	
	BC	6	168.8	45.8	27.1	
	CD	6+1	168.8	44.4	26.3	
	DA	6+1	168.8	58.1	**34.4**	
	Ave.		**168.8**	**46.5**	**27.6**	
	Tie	N/A	332.3	73.7	22.2	
#72 (U)	AB	6+1	168.8	36.2	21.4	
	BC	3	168.8	36.5	21.6	
	CD	0	168.8	29.6	17.5	
	DA	6	168.8	39.2	23.2	
	Ave.		**168.8**	**35.4**	**21.0**	
	Tie	N/A	332.3	77.6	23.4	

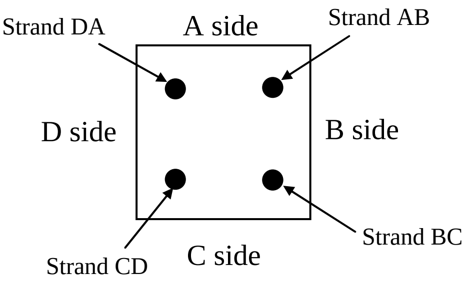
7.6.3.6 Minimal Repair / CFRP / 2 layers / 60 in. (#74, #75 and #76)

These specimens were similar to #67-#69 (2 layers, 36 in.) excepting that the entire length was wrapped. While exposed surfaces in specimens #74 and #75 were sealed, #76 was not sealed. The maximum loss in the strands was 24.9 % (#76-BC) and it was 29.3% (#74) in the ties (Table 7.12). The average steel loss in the strands was 22.1% (#74), 20.7% (#75) and 21.4% (#76). And about 57% and 43% of wires in strands were broken in specimen #74 and #75, while 46% of wires was disconnected in unsealed specimen #76.

Figure 7.48 Distribution of Corrosion Product in #74 (T), #75 (M) and #76 (B)

Figure 7.49 Retrieved Strands and Ties From #70, #71 and #72

Table 7.12 Results of Gravimetric Test for Minimal 2 layer 60 in. (#74, #75 and #76)

Name	Steel	Break in Strand Wires	22 in			Nomenclature of Strand
			Original Weight (g)	**Lost weight (g)**	**Loss Ratio (%)**	
#74 (S)	AB	2	168.8	34.9	20.7	
	BC	6	168.8	40.5	24.0	
	CD	4	168.8	35.5	21.0	
	DA	4	168.8	38.1	22.6	
	Ave.		**168.8**	**37.3**	**22.1**	
	Tie	N/A	332.3	97.3	**29.3**	
#75 (S)	AB	6	168.8	37.3	22.1	
	BC	0	168.8	34.2	20.3	
	CD	3	168.8	34.1	20.2	
	DA	3	168.8	33.9	20.1	
	Ave.		**168.8**	**34.9**	**20.7**	
	Tie	N/A	332.3	74.9	22.5	
#76 (U)	AB	3	168.8	33.2	19.7	
	BC	4	168.8	42	**24.9**	
	CD	4	168.8	32.7	19.4	
	DA	2	168.8	36.7	21.7	
	Ave.		**168.8**	**36.2**	**21.4**	
	Tie	N/A	332.3	61.3	18.4	

7.6.3.7 Summary of Gravimetric Test

The maximum incremental steel losses in the strands and ties for the different repair methods are summarized in Table 7.13. For convenience, only results for sealed specimens are shown in this table.

Assuming that the maximum steel loss in the strand and ties in the unwrapped specimens before exposure to the hot-water tidal cycles was 22.3% and 21.3%, respectively (Table 8.5), the incremental loss in these controls can be readily determined by subtracting this value from the total measured steel loss value reported in Table 7.7. This gives an incremental loss of 64.2% for the strand and 66.1% for the tie. Incremental losses for other types of repairs were similarly determined from the values reported in Tables 7.8-7.12 and are summarized in Table 7.13. Note these are averaged losses over the 22 in. chloride contaminated section.

Table 7.13 Maximum Steel Loss for Different Repair Schemes
(excluding unsealed specimens)

Repair Method	**Strand**		**Tie**	
	Maximum Steel Loss (%)	**Percent Increase**	**Maximum Steel Loss (%)**	**Percent Increase**
Unwrapped Controls	86.5	64.2	87.4	66.1
Full Repair & 2 layer 36in wrap	23.4	1.1	25.1	3.8
Epoxy Repair & 1 layer 36 in. wrap	27.3	5	23.9	2.6
Epoxy Repair& 2 layer 36 in wrap	24.1	1.8	23.2	1.9
Epoxy Repair& 3 layer 36 in. wrap	34.4	12.1	24.7	3.4
Epoxy Repair & 2 layer 60 in. wrap	24	1.7	29.3	8

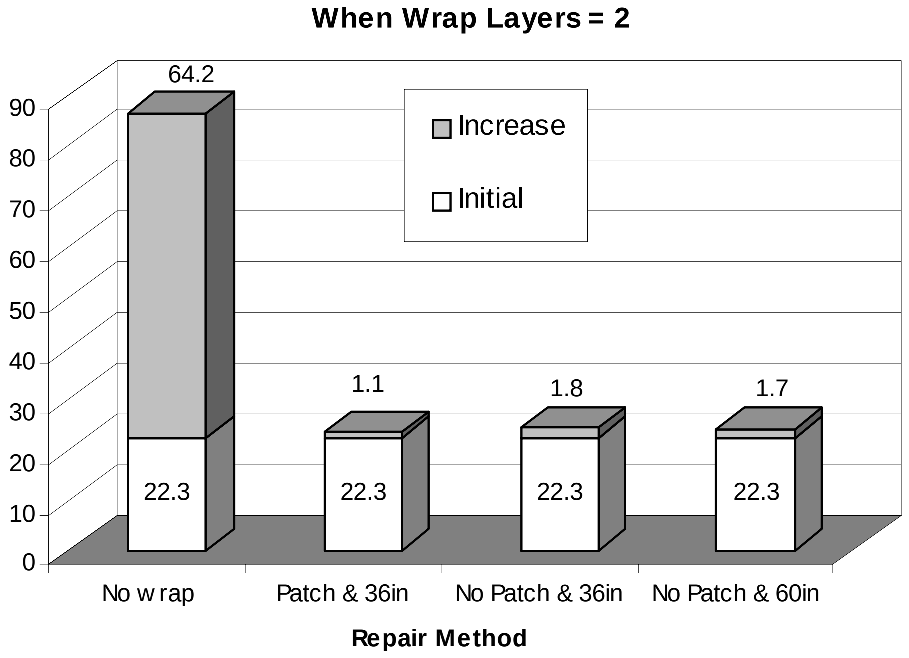


Figure 7.50 Maximum Steel Loss Increase in Strands Wrapped with 2 CFRP Layers
(Patch refers to full repair and No Patch to epoxy repair)

Fig. 7.50 plots the increase in steel loss in strands in controls and specimens wrapped with two CFRP layers using data in Table 7.13. Compared to the miniscule incremental losses in the wrapped specimens, the controls sustained significant (64.2%) metal loss. This was because the control specimens were cracked. The combination of full repair and wrap performed best marginally (1.1% increase). And full wrapping was slightly more effective than the 36 in wrapping. However, epoxy repairs were remarkably effective (1.7% (full - 60 in) or 1.8% (partial – 36 in) vs 1.1% for full repair).

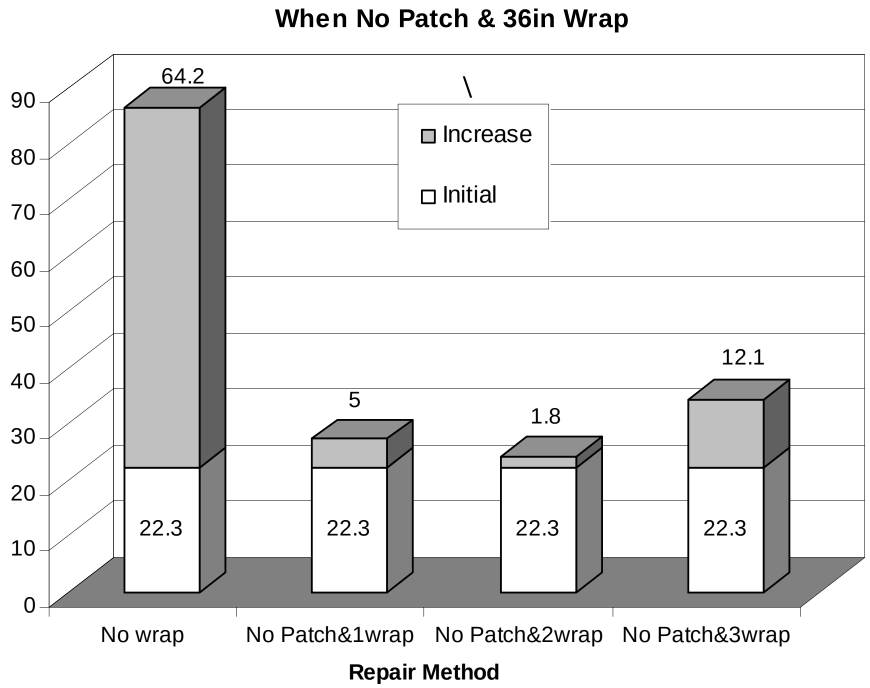


Figure 7.51 Maximum Steel Loss Increase in Strands Wrapped 36 in.

Fig. 7.51 shows the role of the number of CFRP wrap layers in preventing incremental metal loss in the strands when epoxy repairs were used and the wrap covered 36 in. Two layers (1.8%) were more effective than one layer (5%); however, three layers were less effective (12.1%). This could be because the base level corrosion assumed to be 22.3% was higher. Table 7.14 summarizes information on the number of prestressing wires that completely corroded as a result of the exposure. The data in Table 7.14 corroborate the findings in Table 7.13 and Fig 7.50-7.51.

Table 7.14 Number of Broken Wires in Strands from Different Repair Methods (excluding unsealed specimens)

Repair Methods	Broken Wires in Strands
Unwrapped Controls	56
Full Repair & 2 layer 36 in. wrap	23
Epoxy Repair & 1 layer 36 in. wrap	43
Epoxy Repair & 2 layer 36 in. wrap	23
Epoxy Repair & 3 layer 36 in. wrap	36
Epoxy Repair & 2 wrap 72 in wrap	28

Fig. 7.52 shows the correlation between numbers of broken wires and averaged actual steel loss. The horizontal axis represents the sum of broken wires in the strands in each specimen and the vertical axis shows the averaged metal loss of four strands. This graph shows that all four strands might be completely corroded if the steel loss is over 30% since the total number of wires in a specimen is 4 (strands) x 7 (wires) =28.

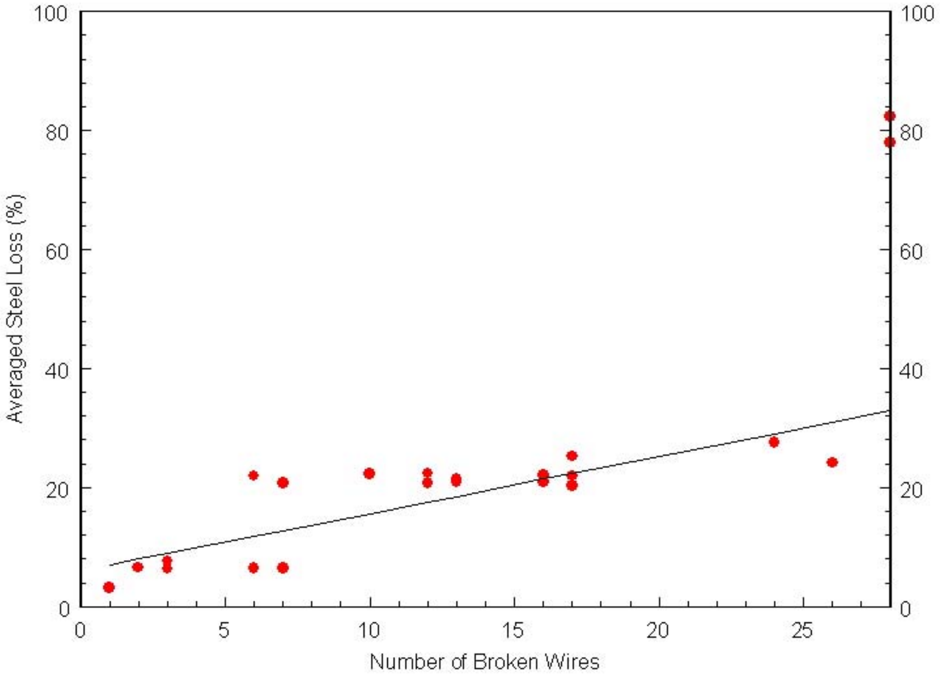


Figure 7.52 Relationship Between Number of Broken Wires and Actual Steel Loss

7.7 Conclusions

Based on the information presented in this chapter the following conclusions may be drawn:

1. Exposure to an extreme environment in which specimens were subjected to near 100% humidity and high temperatures (140F) for nearly 2 years did not adversely affect the FRP-concrete bond from the standpoint of strengthening. Ultimate strength tests on exposed wrapped specimens indicated that they retained or exceeded their pre-wrap strength this despite end failures that occurred before the section had reached its full capacity. In contrast, unwrapped controls showed strength losses of over 30%. (Table 7.6, Fig. 7.36).

2. Gravimetric tests showed that the incremental metal loss in unwrapped controls at the end of the exposure period was over 60%. In contrast, losses in wrapped specimens ranged from 1.1% to 12.1% (Table 7.13).

3. The effect of sealing the concrete surface and the top surface had limited effect. By and large, the performance of sealed and unsealed specimens were very similar (see Table 7.8-7.12).

4. The performance of full repair and epoxy repair in terms of total steel loss was similar (Table 7.13-14). Total metal loss and wire breaks were almost the same. However, the ultimate capacity of the full repair was somewhat better. The merit of using full repair may be limited.

5. The effectiveness of the FRP repair is not proportional to the number of FRP layers (Fig. 7.51). This finding agrees with that from the ambient temperature exposure study.

6. The analysis of the gravimetric test data showed that if the steel loss is over 30%, all four strands might be completely corroded (Fig. 7.52)

References

7.1 Sika Corporation, www.sika.com

7.2 Application Instruction Note sent from SDR Engineering (Apr. 15, 2002)

8. COLUMN STUDY

8.1 Introduction

This chapter presents results of ultimate load tests on column specimens to evaluate the efficacy of underwater FRP wrap. Prior to the wrap, specimens were subjected to constant current accelerated corrosion to attain targeted metal loss levels. Complete details of the test program including information of pre- and post-wrap measurements undertaken are contained in Section 8.2. The setup for the eccentric column tests is described in Section 8.3. The results of eccentric load test are presented in Section 8.4 and gravimetric results summarized in Section 8.5. Results of limited pullout tests to evaluate bond are given in Section 8.6. The principal findings are summarized in Section 8.7.

8.2 Test Program

Since FRP strengthening is a bond-critical application, the goal of this experimental study was to evaluate the effectiveness of the underwater FRP-concrete bond through ultimate load column tests. Additionally, gravimetric tests were undertaken to verify the extent of metal loss due to accelerated corrosion. A total of 11 specimens were utilized in this study (Table 8.1). Ten of these were 6 ft long specimens that were used in ultimate load tests. An additional 5 ft specimen was used for gravimetric testing. Of the ten column specimens, four were wrapped and six were unwrapped controls. Targeted steel loss levels were 25% and 50%.

Two series of tests were carried out. In the first series, specimens were corroded to a targeted metal loss level of 25%, wrapped and tested. In this series, a total of six specimens were tested – four controls and two wrapped specimens. The four unwrapped controls corresponded to 0% metal loss (#18, #19) and 25% metal loss (#20, #21). These tests provided baseline data that could be used to assess the performance of the two wrapped specimens (#24, #25) that had previously been corroded to the same 25% targeted metal loss. Results from these tests provide an immediate measure of the enhanced performance due to FRP wrapping. Additionally, an identically corroded 5 ft specimen (#11) was tested gravimetrically to establish the actual metal loss.

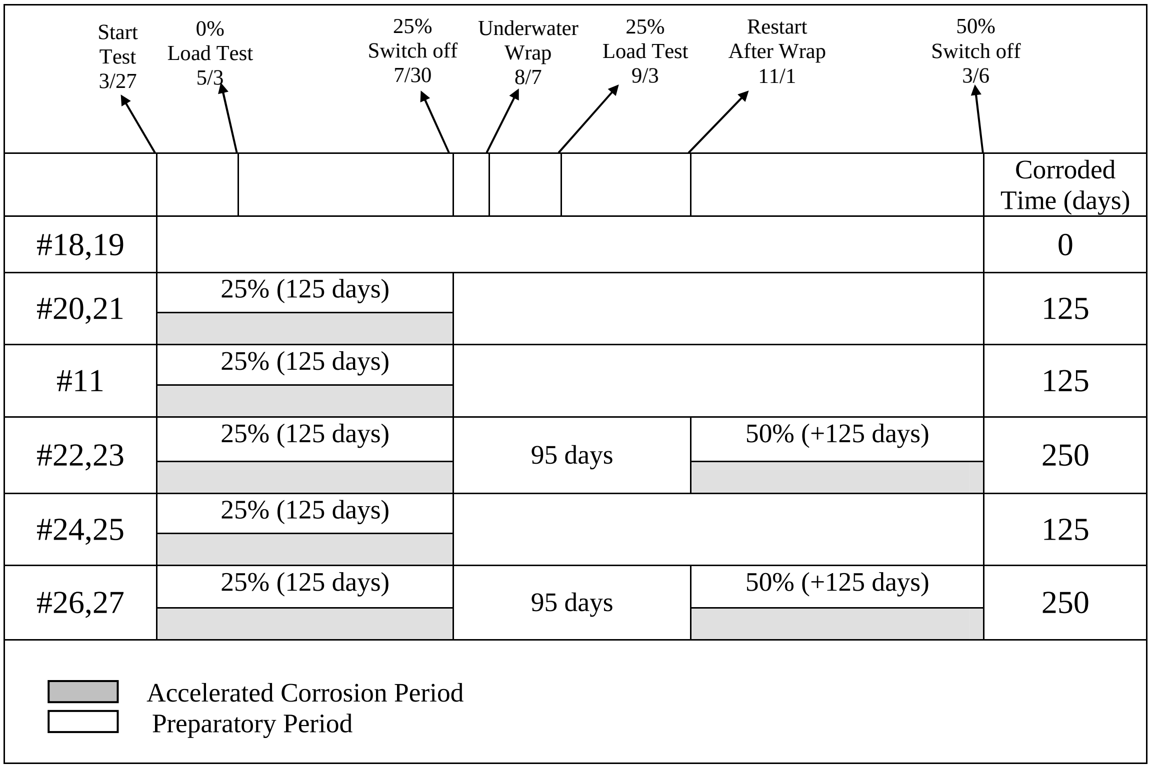
In the second series, four column specimens were tested. Two of these were wrapped (#26, #27) and two were unwrapped controls (#22, 23). The wrapped specimens were first corroded to 25%, wrapped and then subjected to further accelerated corrosion to attain a targeted metal loss levels of 50%. The unwrapped controls were subjected to the same regime. At the end of the exposure period, all specimens were tested eccentrically to obtain residual capacities. At these high levels of corrosion, it is not possible to retrieve the corroded steel and therefore no attempt was made to conduct gravimetric testing.

Table 8.1 Specimen Details

Specimen Number	Type	Size (ft)	Corrosion Acceleration	Wrap (CFRP)	Target Steel Loss
#18 #19	Strength Control	6	No	No	0%
#20 #21	Strength Control	6	Yes	No	25%
#11	Gravimetric Control	5	Yes	No	25%
#22 #23	Strength Control	6	Yes	No	50%
#24 #25	Strength Wrap	6	Yes	2 layers	25%
#26 #27	Strength Wrap	6	Yes	2 layers	50%

Details of the test program are summarized in Table 8.1. The time line is shown in Table 8.2. This presents detailed information on when the corrosion acceleration was stopped, the underwater repair undertaken and eccentric load tests performed.

Table 8.2 Time Line



				Corroded Time (days)
#18,19				0
#20,21	25% (125 days)			125
#11	25% (125 days)			125
#22,23	25% (125 days)	95 days	50% (+125 days)	250
#24,25	25% (125 days)			125
#26,27	25% (125 days)	95 days	50% (+125 days)	250

▨ Accelerated Corrosion Period
☐ Preparatory Period

8.2.1 Pre-Wrap Experiments

8.2.2 Corrosion Acceleration

The accelerated corrosion scheme utilized was similar to that used in an earlier research project [1]. In the setup (Fig. 8.1), all specimens were exposed to a constant current of 110 mA so that the applied current density was limited to 100 µA/cm^2 – the upper limit of the measured corrosion rate in naturally corroding specimens exposed to a marine environment. The steel area used for this calculation includes the four strands and adjoining spiral ties embedded in the 22 in. chloride contaminated zone.

The center strand served as a cathode while the other four strands attached electrically to the ties served as the anode. This arrangement was used since it permitted specimens to be corroded even after they had been wrapped. A soaker hose-sponge system was used to apply continuous moisture to the specimens to reduce the resistivity of the concrete.

To minimize local corrosion, the applied current was gradually increased from 10 mA to 110 mA over 6 days. The applied current and voltage was manually monitored throughout the study. Fig. 8.2 shows the variation of current with time for 125 days – the targeted time for attaining 25% metal loss. There was a steep increase in voltage for the first six days as the impressed current gradually increased to 110 mA. After that, increased internal concrete resistance due to corrosion products and cracking led to an increase in the voltage since the current remained constant (Fig. 8.3).

(a) (b)

Figure 8.1 (a) Specimens Set-up; (b) Wire Connection

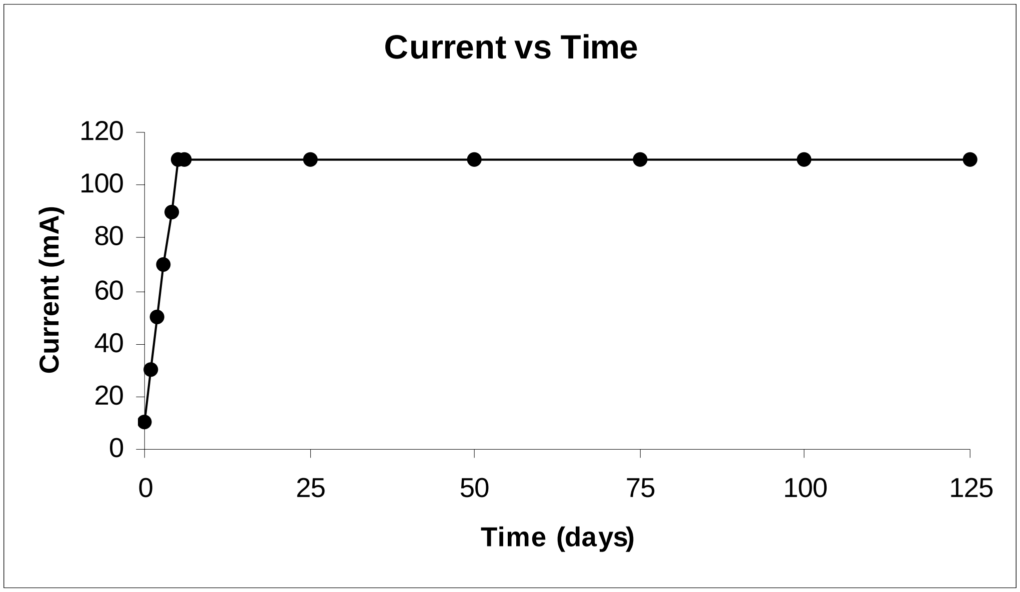


Figure 8.2 Current vs Time Graph

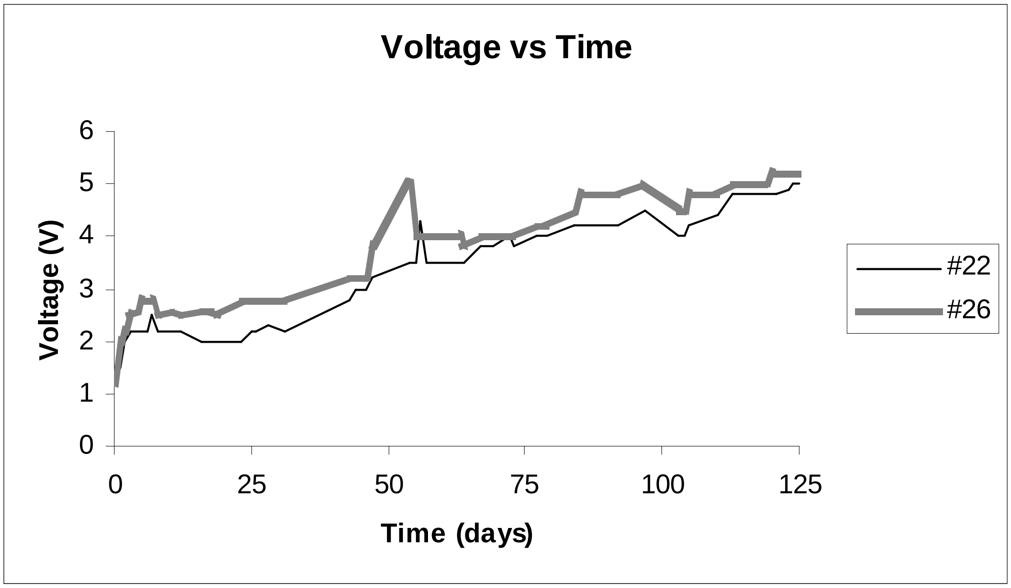


Figure 8.3 Voltage vs Time Graph

8.2.2.1 Crack Survey

Crack surveys were performed on all nine specimens at 25% targeted steel loss. The location of cracks was mapped by tracing them onto a plastic sheet. This was then plotted on a 2 in. x 2 in. grid. Table 8.3 shows a summary of the crack survey results. These values were averaged for the ten 6 ft specimens. Cracks in 5 ft specimen (#11) were greater than those for the 6ft specimens in terms of number of cracks (39 vs 32.4), maximum length (32.5in vs 28.7in), and maximum crack width (3 mm vs 1.2 mm). Although they were under the same corrosion acceleration scheme and have same 22in of chloride contaminated area, the specimen size seemed to affect corrosion of steel. The crack pattern for #11 specimen is shown in Fig. 8.4. Most cracks were concentrated in the chloride contaminated region. The location of cracks in the other 6 ft piles are presented in Figs. 8.5 and 8.6.

Table 8.3 Averaged Crack Information

	Number	Maximum Length (in.)	Maximum Width (mm)
5ft specimen (#11)	39	32.5	3
6ft specimens	32.4	28.7	1.2

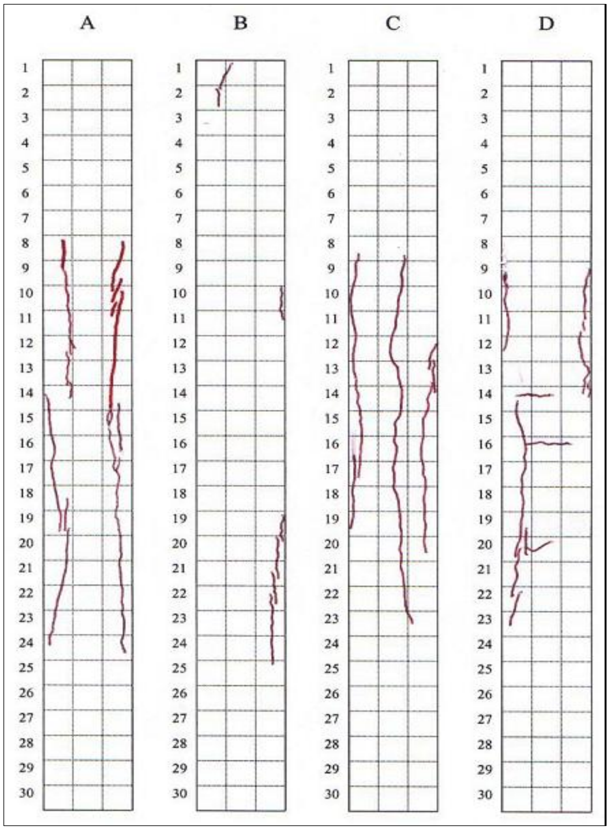
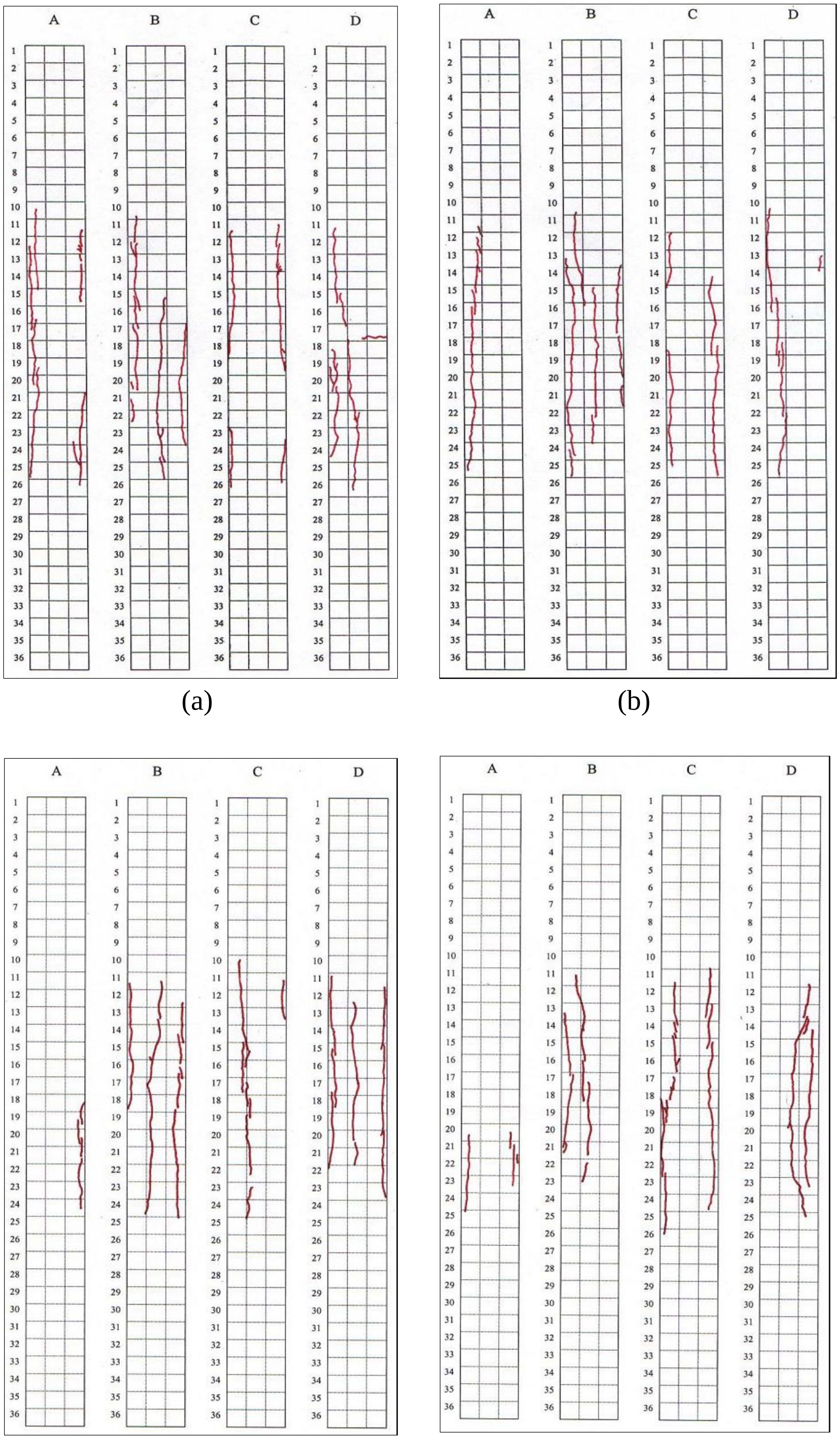


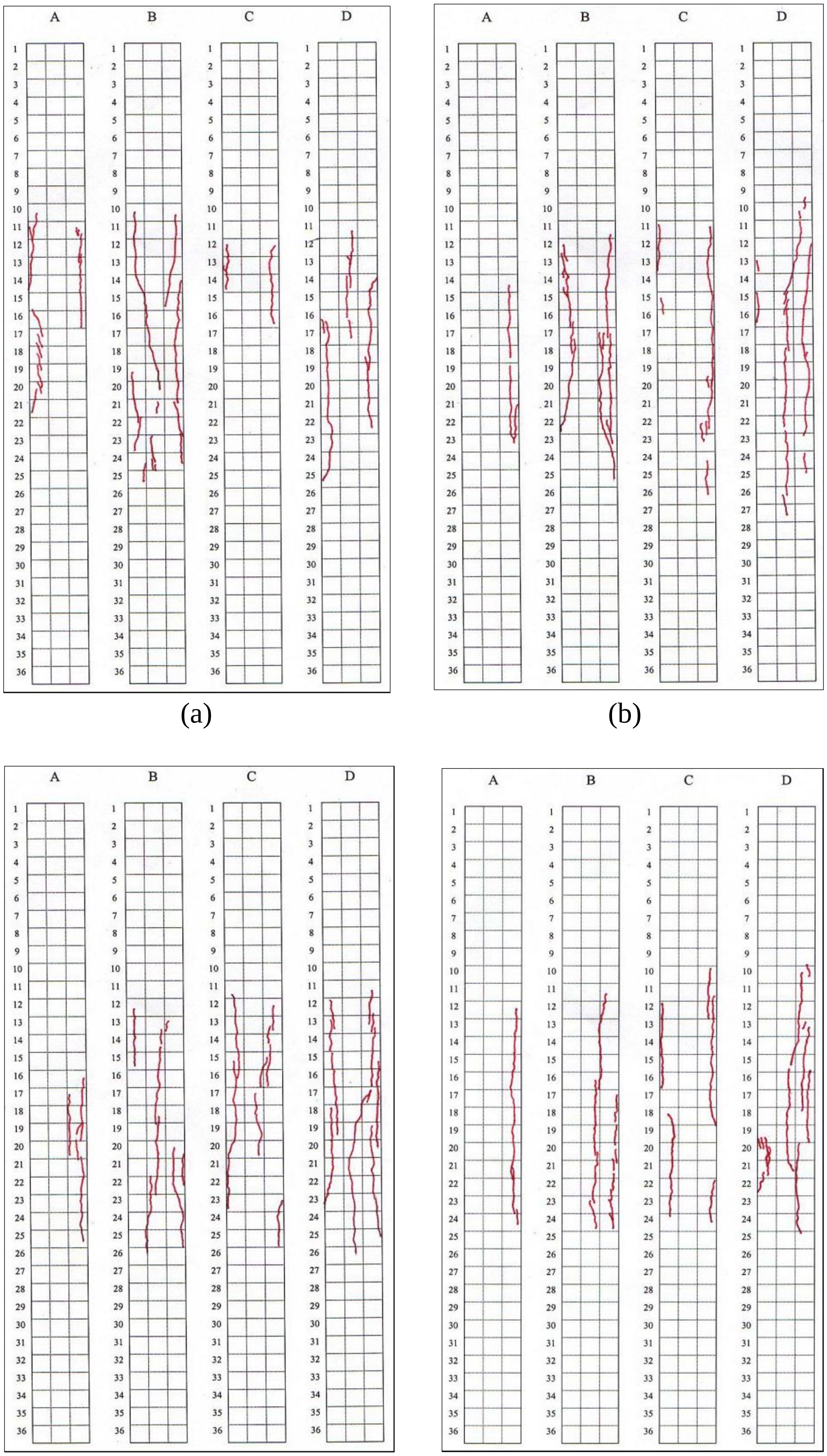
Figure 8.4 Crack Pattern of #11 Specimen at 25% Targeted Steel Loss



(a) (b)

(c) (d)

Figure 8.5 Crack Patterns of (a) #20, (b) #21, (c) #22 and (d) #23



(a) (b)

(c) (d)

Figure 8.6 Crack Patterns of (a) #24, (b) #25, (c) #26 and (d) #27

Table 8.4 Crack Information of 6 ft Specimens at 25% Targeted Steel Loss

Specimen	The Number of Cracks	Maximum Crack Length (in)	Maximum Crack Width (mm)
#20	40	30	2
#21	24	30.5	1.5
#22	29	26	1
#23	31	28.5	0.8
#24	36	20.5	1.5
#25	35	34.5	1.25
#26	31	30	1
#27	33	29.5	0.6

8.2.2.2 Gravimetric Test

The targeted steel loss level of 25% was estimated to take 125 days based on Faraday's Laws. This estimate had been found to under-predict actual metal loss in a previous study [1] using external cathodes. However, as internal cathodes were used here, one specimen (#11) was cut off to verify the actual metal loss after 125 days (Fig. 8.7).

The result of the gravimetric test is summarized in Table 8.3. The actual metal loss was smaller than estimated but still very significant. The steel loss in the four strands was 20.51% while it was 21.31% for the spiral ties. Therefore, the total steel loss was 20.77% after 125 days.

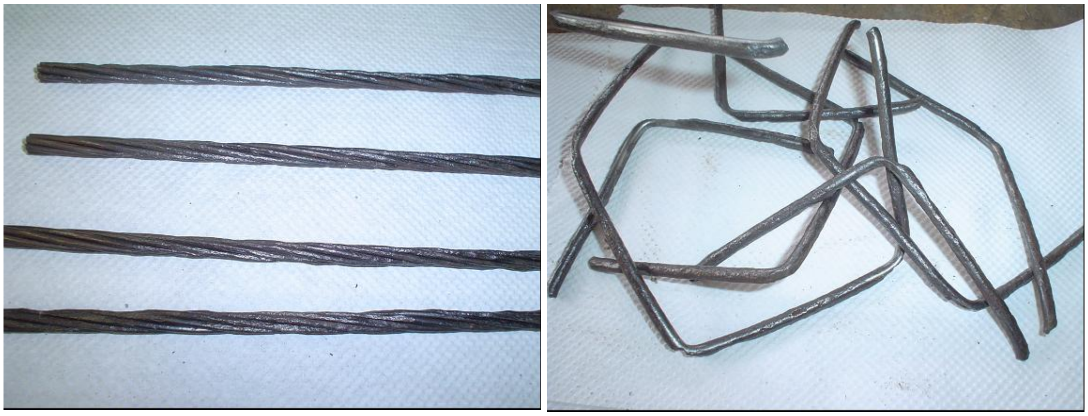

(a) (b)

Figure 8.7 (a) Cleaned Strands; (b) Cleaned Ties

Table 8.5 Result of Gravimetric Test of #11 at 25% Targeted Steel Loss

Type	Initial Weight (g)	Final Weight (g)	Weight Loss (g)	Metal Loss (%)
AB	168.78	133.01	35.77	21.19
BC	168.78	132.14	36.64	21.71
CD	168.78	140.30	28.48	16.88
DA	168.78	131.20	37.58	22.27
Strands	675.13	536.65	138.48	**20.51**
Tie	332.55	261.70	70.85	**21.31**
Total	**1007.68**	**798.35**	**209.33**	**20.77**

8.2.3 Underwater Wrapping

A total of five specimens were wrapped in salt water using Aquawrap Repair System developed by Air Logistics. All five specimens were wrapped over a 3 ft length in the middle using 2 layers of a bi-directional carbon fiber. Four of these wrapped specimens were later tested eccentrically as described in the Section 8.2. The fifth was used to evaluate the FRP-concrete substrate bond using pull out tests. The properties of the materials used for the underwater wrapping were given in Chapter 3.

To simulate underwater FRP wrapping of corrosion-damaged piles in salt water, a 6 ft x 10 ft x 3.5 ft fiberglass tank was built. The inside wall of the tank was coated with epoxy based glass fiber and filled with salt water to a depth of 3 ft. The surfaces of specimens were prepared and sharp edges ground using a hand grinder. All five specimens were set up vertically inside the tank as shown in Fig. 8.8.

The procedure for wrapping the specimens under water was as follows.

i. Mix the base primer composed of a red colored part A and a clear brown colored part B.
ii. Apply the primer to the prepared pile surface by hand.
iii. Place the 4 in. wide bi-directional carbon fiber spirally over the primer-coated area in two continuous layers without overlap.
iv. Place one layer of the 6 in. wide glass fiber veil over the carbon fiber with a 2 in. overlap to consolidate the wrap and provide the better finish.
v. Place the blue colored plastic stretch film over the veil and puncture its surface with a sharp tool to allow gases to escape.
vi. Remove the stretch film after curing completely (1 day).
vii. Place the mixed base primer over the veil for the more protection of the wrap.

(a) (b)

(c) (d)

(e) (f)

Figure 8.8 (a) Mixing Primer; (b) Applying Primer to Surface; (c) Wrapping CFRP; (d) Wrapping Glass Fiber Veil; (e) Wrapping Stretch; (f) Coating Primer

8.2.4 Post-Wrap Experiments

8.2.4.1 Corrosion Acceleration

Two wrapped (#26, #27) and two unwrapped (#22, #23) specimens were subjected to acceleration corrosion following completion of the wrapping operation. As before, 110 mA of impressed current was applied to all four specimens for another 125 days and sponge-water soaker system used to lower concrete resistivity. Fig. 8.9 shows the set-up of corrosion acceleration after wrapping.

Figure 8.9 Corrosion Acceleration Set-up after Wrapping

The applied current and voltage was manually monitored throughout the study. Fig. 8.10 shows the variation of voltage with time for another 125 days – the targeted time for attaining 50% metal loss. There was a steep increase in voltage for the first six days as the impressed current gradually increased to 110 mA. Compared to the pre-wrap corrosion acceleration (Fig. 8.3), however, the value of voltage was much higher but its rate of increase was gentler.

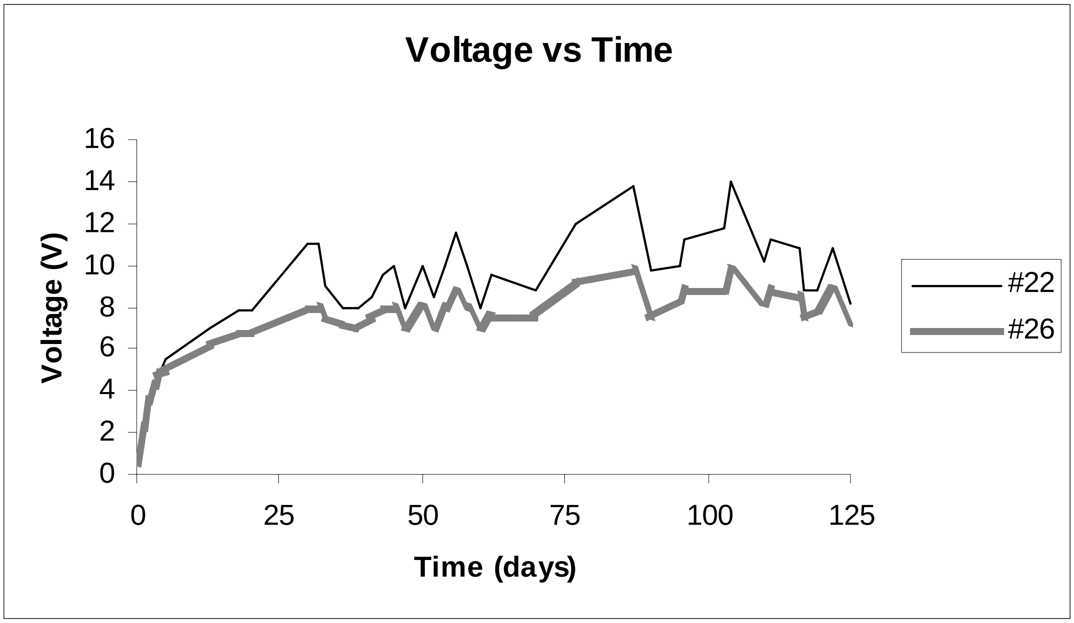


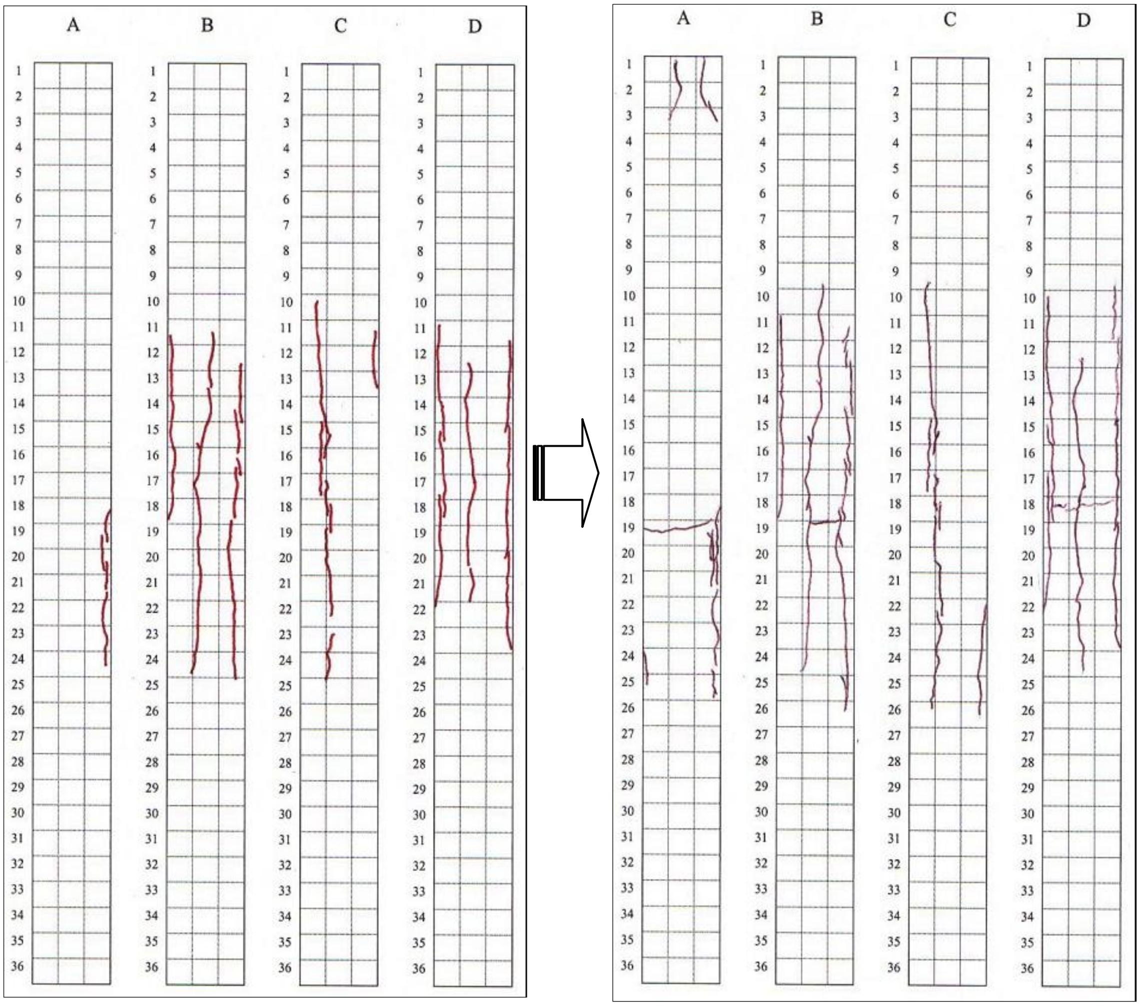
Figure 8.10 Voltage vs Time Graph for Another 125 days

8.2.4.2 Crack Survey

After 125 days of accelerated corrosion, a crack survey was conducted on the four specimens. Table 8.6 shows the result of the crack survey of the two unwrapped control specimens. Compared to the crack distribution for the 25% steel loss, the number of cracks, the maximum crack length, and the maximum crack width were increased by 104%, 38%, and 200%, respectively in the unwrapped specimens. As expected, cracks were produced along the strands. Some cracks were generated along the corners of the specimen that had been rounded for easy application of wrapping during casting. Lateral cracks were generated in unwrapped specimens at 50% metal loss (Fig. 8.11 and 12). Some cracks were present around the ends of the specimens and they seemed to be due to the corrosion of exposed strands that were required for electrical connection. Fig. 8.13 shows the crack pattern of wrapped specimens at 50% targeted steel loss. No cracks extended over the wrapped area and some cracks were produced around the ends of the specimens.

Table 8.6 Crack Information of Unwrapped Specimens 25% vs 50% Steel Loss

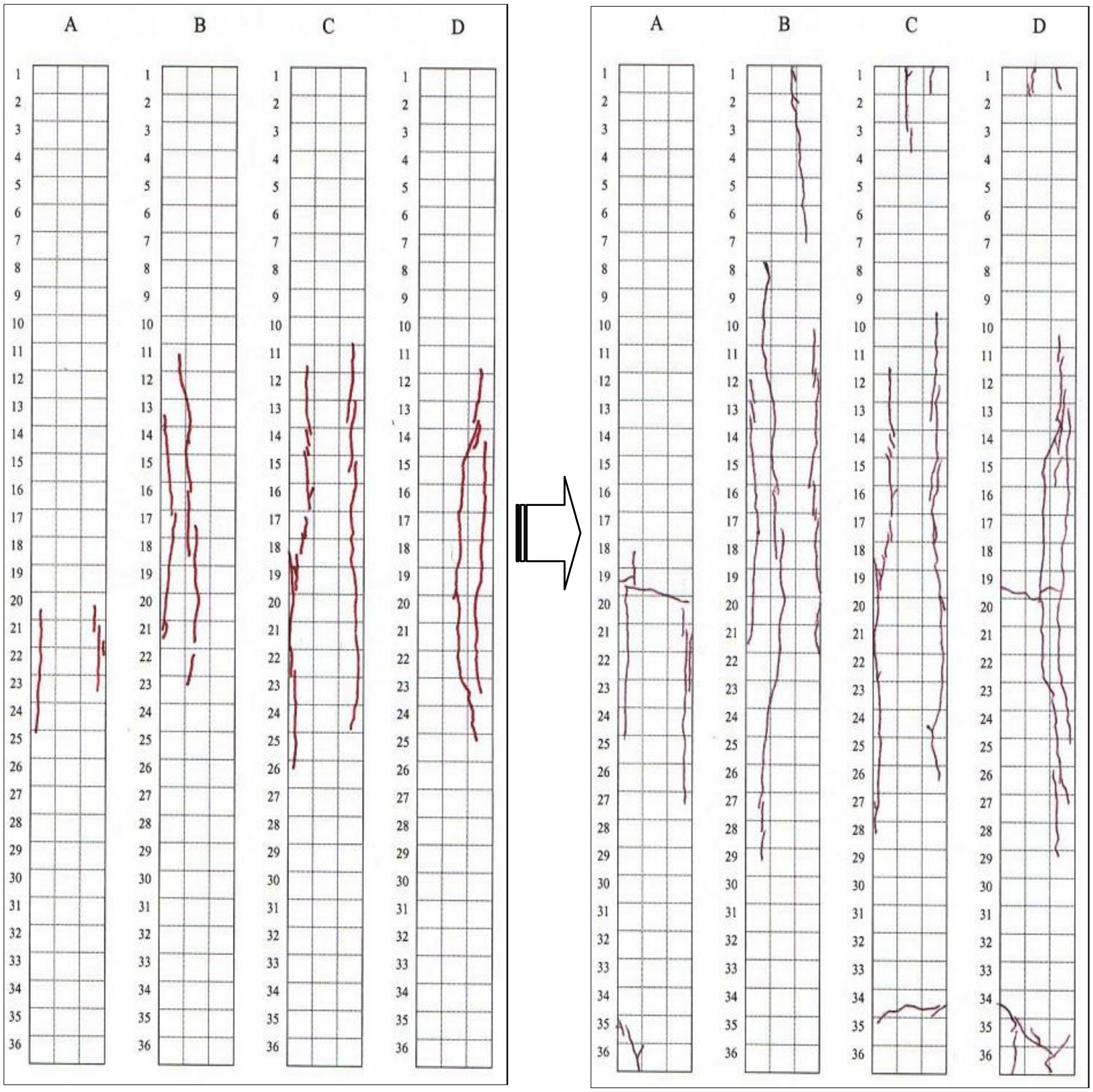
Specimen	Number of Cracks	Increase (%)	Maximum Crack Length (in)	Increase (%)	Maximum Crack Width (mm)	Increase (%)
#22	49	+69	32.5	+25	3.5	+250
#23	74	+139	43	+51	2	+150
Average	61.5	+104	37.8	+38	2.8	+200



25%

50%

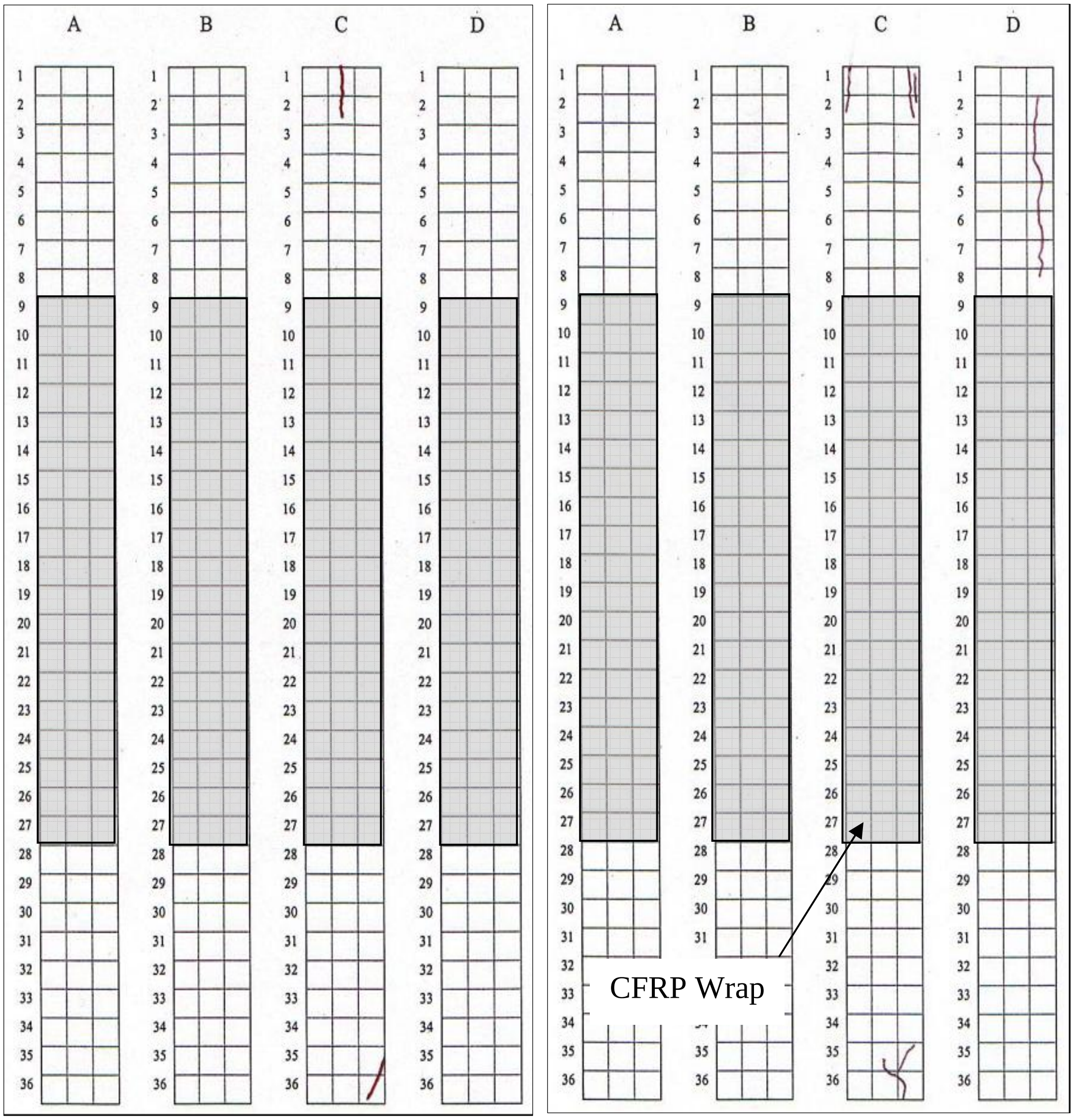
Figure 8.11 Crack Change of #22 Specimen at 50% of Targeted Steel Loss



25%

50%

Figure 8.12 Crack Change of #23 Specimen at 50% of Targeted Steel Loss



#26 #27

Figure 8.13 Crack Patterns of Wrapped Specimens at 50% of Targeted Steel Loss

8.3 Eccentric Load Tests

As stated earlier, ten specimens including six controls were tested under eccentric loading. Initially, two unwrapped control specimens were tested to determine the baseline strength of the uncorroded specimens. After 125 days of exposure to corrosion acceleration, two unwrapped controls and two wrapped specimens were tested. Finally, after another 125 days of exposure, two unwrapped and two wrapped specimens were tested. Details are summarized in Table 8.1.


(a) (b)
(c) (d)
(e) (f)

Figure 8.14 (a) Steel Swivel; (b) Roller Assembly; (c) Roller-Swivel Assembly; (d) Hydraulic Cylinder; (e) Hydraulic Jack; (f) Setting up at the Bottom

8.3.1 Test Set-Up

The eccentric load test was conducted using two roller-swivel assemblies, one for each end of the column (Fig. 8.14). The steel swivel was composed of two 8 in. diameter hemispherical members designed to rotate in any direction [2]. A roller with a 1.5 in. diameter and 6 in. length was placed between two steel plate and four cylindrical guide rods were welded on plates to ensure that the roller could only rotate in one direction. The roller was bolted to the swivel and a 16 in. x 16 in. square steel plate bolted to the roller-swivel assembly to provide a flat contact surface with the specimen. The roller was placed exactly 1.2 in. from the centerline of specimen to provide an eccentricity ratio, e/h of 0.2 for the 6 in. square specimens.

One roller-swivel assembly was placed on the load cell at the bottom and the other was attached to the piston ram of a hydraulic cylinder with a 300 ton capacity at the top. The ends of specimen were positioned on a flat steel plate so that the applied load was uniformly distributed. To prevent premature end failure, 6 in. steel plates were attached to both ends of the specimens and fixed with bolts. The exact position of the column in the test frame was adjusted by monitoring the strain readings under the nominal loading (Fig. 8.14).

8.3.2 Specimen Preparation

The concrete surface in contact with the steel plate at the ends had to be smooth so that uniform load was applied. Therefore, strands protruding from the concrete at the ends had to be cut off and the surface ground to a smooth finish.

Initially, the strands protruding at the bottom end were cut and epoxy coated to prevent corrosion. The strands protruding at the top end however could not be cut since they were required to allow electrical connection to the impressed current accelerated corrosion scheme. As a result, cracks and concrete spalling developed during the time the specimen was being corroded outdoors. To prevent premature end failure, the spalled concrete was patched using Sika 611 and an epoxy based CFRP system wrapped over a 6 in depth at the end. After curing, the concrete surface at both ends were ground to provide a flat surface for testing (Fig. 8.15).

(a) (b)

Figure 8.15 (a) Damaged End; (b) Repaired End

8.3.3 Instrumentation

To monitor strain changes on the concrete surface, PL-60-11-1L strain gages were attached to the concrete surface. A total of 12 strain gages were mounted at three levels – 12 in. from each end and at the middle - on all four faces of specimen. Before strain gages were attached, concrete surfaces were ground smooth and cleaned using acetone (Fig. 8.16).

Axial deflections were measured using two LVDTs having a 0.2 in. stroke. Lateral deflections were measured using four LVDTs with a 4 in. stroke. These were placed 18 in. apart. All load-strain plots are included in the Appendix B.

(a) (b)

Figure 8.16 (a) Installing Strain Gages; (b) Installing LVDT

8.3.4 Test Procedure

A MEGADAC 3100 data acquisition system was used for monitoring and recording data from all the strain gages, LVDTs, and loads. A 300 ton load cell manufactured by GEOKON was used to measure the load. The load was applied by a hydraulic jack connected to an electrically operated pump. The hydraulic jack was manufactured by Force Resources, Inc. and had a 300 ton and 13 in stroke capacity.

After checking all the connections to the MEGADAC system, data was initialized to zero. The position of the column inside the test frame was adjusted by monitoring measured strains and calculated. When the specimen was positioned correctly, the load was monotonically increased.

8.4 Test Results

8.4.1 0% of Targeted Steel Loss

Two unwrapped, uncorroded controls (#18 and #19) were tested eccentrically to establish the baseline strength of the columns. Results are summarized in Table 8.7. The average ultimate load was 126.7 kips from the two tests. This value was used subsequently for calculating the strength gain (or loss) for different targeted steel loss values. Plots showing the lateral deflection and strain variation with load at the mid span section are presented in Appendix B.

Fig. 8.17 shows the failure mode of specimens. All failures occurred in the middle region. As shown in the photo, the exposed steel is uncorroded and the strands were perfectly confined by the spiral stirrups.

Table 8.7 Result of Eccentric Load Test at 0% Steel Loss

Name	Type	Ultimate Load (kips)
#18	Control	125.8
#19	Control	127.5
Average		**126.7**

Figure 8.17 Failure of Unwrapped Controls at 0% Steel Loss

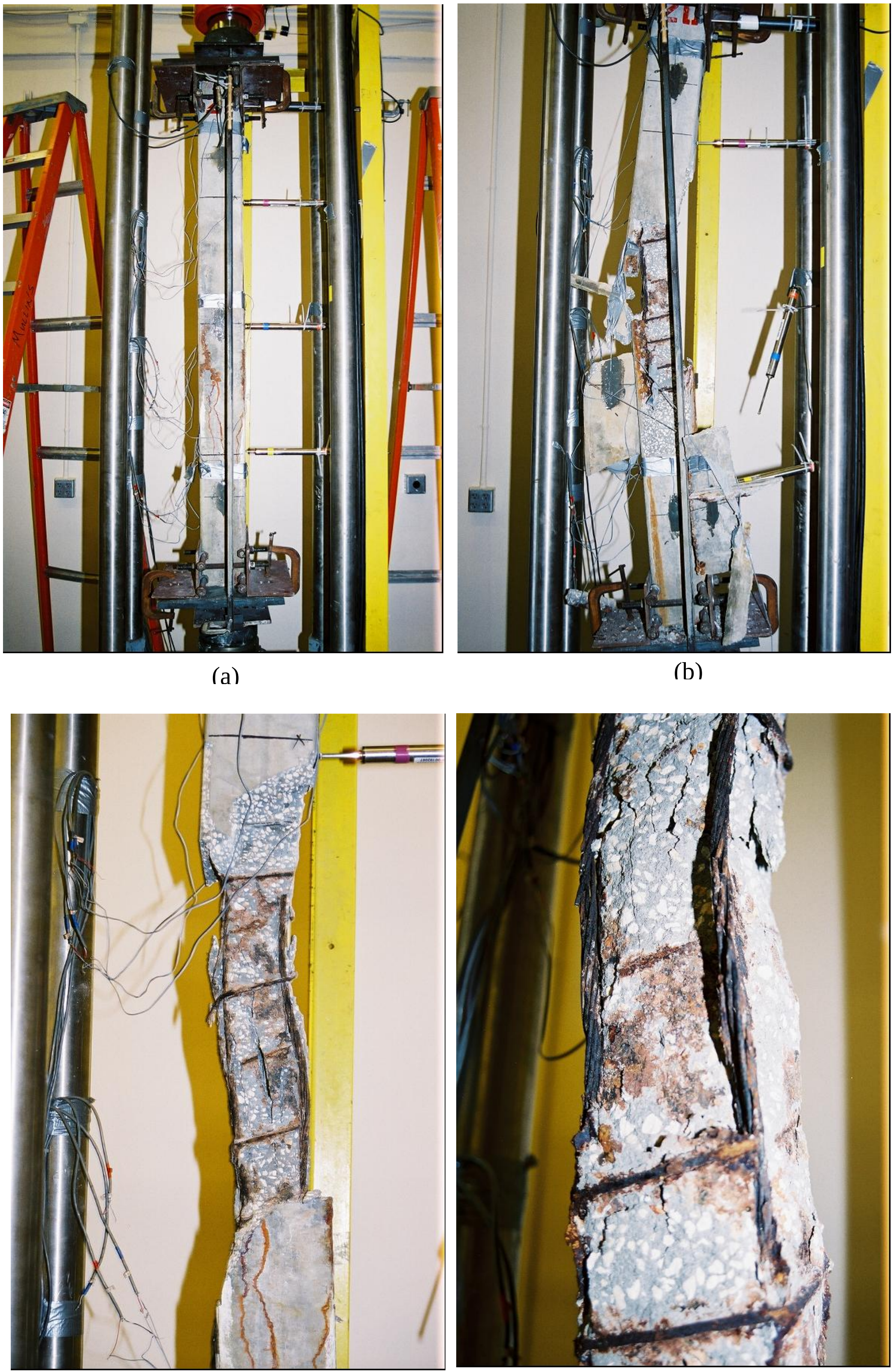
8.4.2 25% of Targeted Steel Loss

After 125 days' exposure to the accelerated corrosion set up for a targeted metal loss of 25%, two wrapped specimens and two unwrapped controls were tested eccentrically. Results are summarized in Table 8.8. For the targeted metal loss, the ultimate capacity was 88.6 kips for the unwrapped controls but 137.6 kips for the wrapped specimens. This means that while the strength of the corroded control specimen had decreased by 30% (see Table 8.7), wrapping had led to an 8.7% increase over its original uncorroded capacity. Plots of lateral deflections and strain variations with load are shown in Appendix B.

The failure mode of the tested specimens are shown in Figs. 8.18 and Figs. 8.19. As before, failure occurred in the mid-area for both unwrapped and wrapped specimens. It may be seen from Fig. 8.18, that stirrups around the mid-area were broken due to corrosion. This resulted in a 45.2% decrease in strength capacity in specimen #20 accompanied by large deflection. However, wrapped specimens showed less deflection. Interestingly, FRP ripped in the lateral direction on the tension side while it was tore both laterally and longitudinally on the compression side.

Table 8.8 Result of Eccentric Load Test at 25% Steel Loss

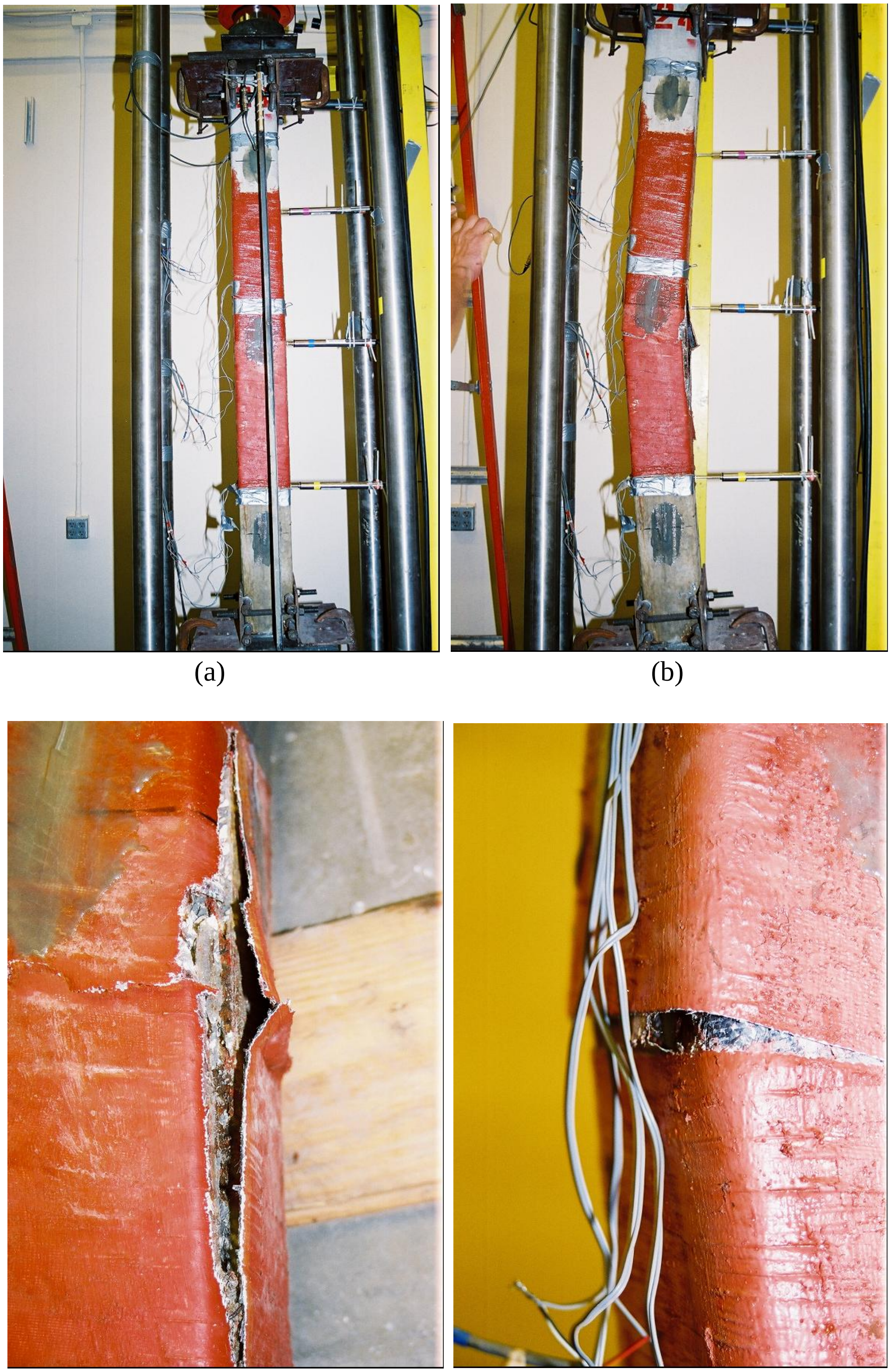
Name	**Type**	**Ultimate Load (kips)**	**Increase (%)**
#20	Controls	69.4	-45.2
#21		107.8	-14.9
Average		**88.6**	**-30.0**
#24	Wrapped	130.0	2.6
#25		145.3	14.7
Average		**137.6**	**8.7**



(a) (b)

(c) (d)

Figure 8.18 Failure of Unwrapped Controls at 25% Steel Loss



(a) (b)

(c) (d)

Figure 8.19 Failure of Wrapped Controls at 25% Steel Loss

8.4.3 50% of Targeted Steel Loss

Four specimens – two wrapped and two unwrapped - were exposed to the corrosion acceleration scheme for a further 125 days to achieve a targeted 50% metal loss. These specimens were then tested eccentrically as before.

Results are summarized in Table 8.9. The average of ultimate load capacity was 79.6 kips for the unwrapped specimens and 151.3 kips for the wrapped specimens. The capacity of the control specimens decreased by 37.2% due to the increased metal loss. However, in the wrapped specimens, strength capacity increased by 19.5%. Interestingly, the strength increase of 19.5% at 50% steel loss was larger than its increase of 8.7% at 25% steel loss.

Increase in concrete strength may have partially contributed to the observed strength gain. Table 8.10 shows the result of the concrete cylinder test. As the steel loss increased from 25% to 50%, the cylinder strength increased from 8.88 ksi to 9.03 ksi in the regular concrete and from 8.16 ksi to 8.34 ksi in the chloride contaminated concrete. Plots are presented in Appendix B.

Table 8.9 Result of Eccentric Load Test at 50% Steel Loss

Name	**Type**	**Ultimate Load (kips)**	**Increase (%)**
#22	Controls	71.2	-43.7
#23		87.9	-30.6
Average		**79.6**	**-37.2**
#26	Wrapped	146.5	15.7
#27		156.1	23.2
Average		**151.3**	**19.5**

A bar plot of all the test results are shown in Fig. 8. 20. The results show that the metal loss and capacity were not directly related. This is not surprising since corrosion was not necessarily uniform in all the strands resulting in non-symmetric cracking. This could result in the applied load being more or less eccentric.

Table 8.10 Result of Concrete Cylinder Test (Unit: ksi)

	0 %	25%		50%	
	Strength (ksi)	Strength (ksi)	Increase (%)	Strength (ksi)	Increase (%)
Regular Concrete	8.10	8.88	9.5	9.03	11.4
Chloride Mixed Concrete	7.28	8.16	12.0	8.34	14.6

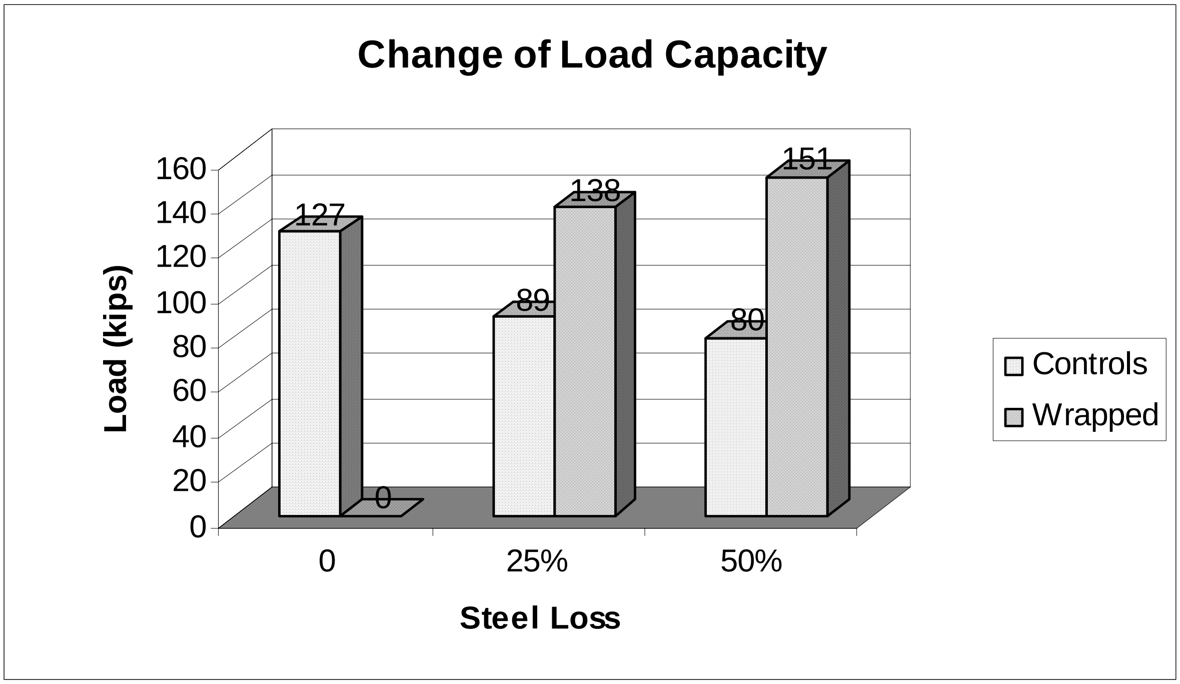
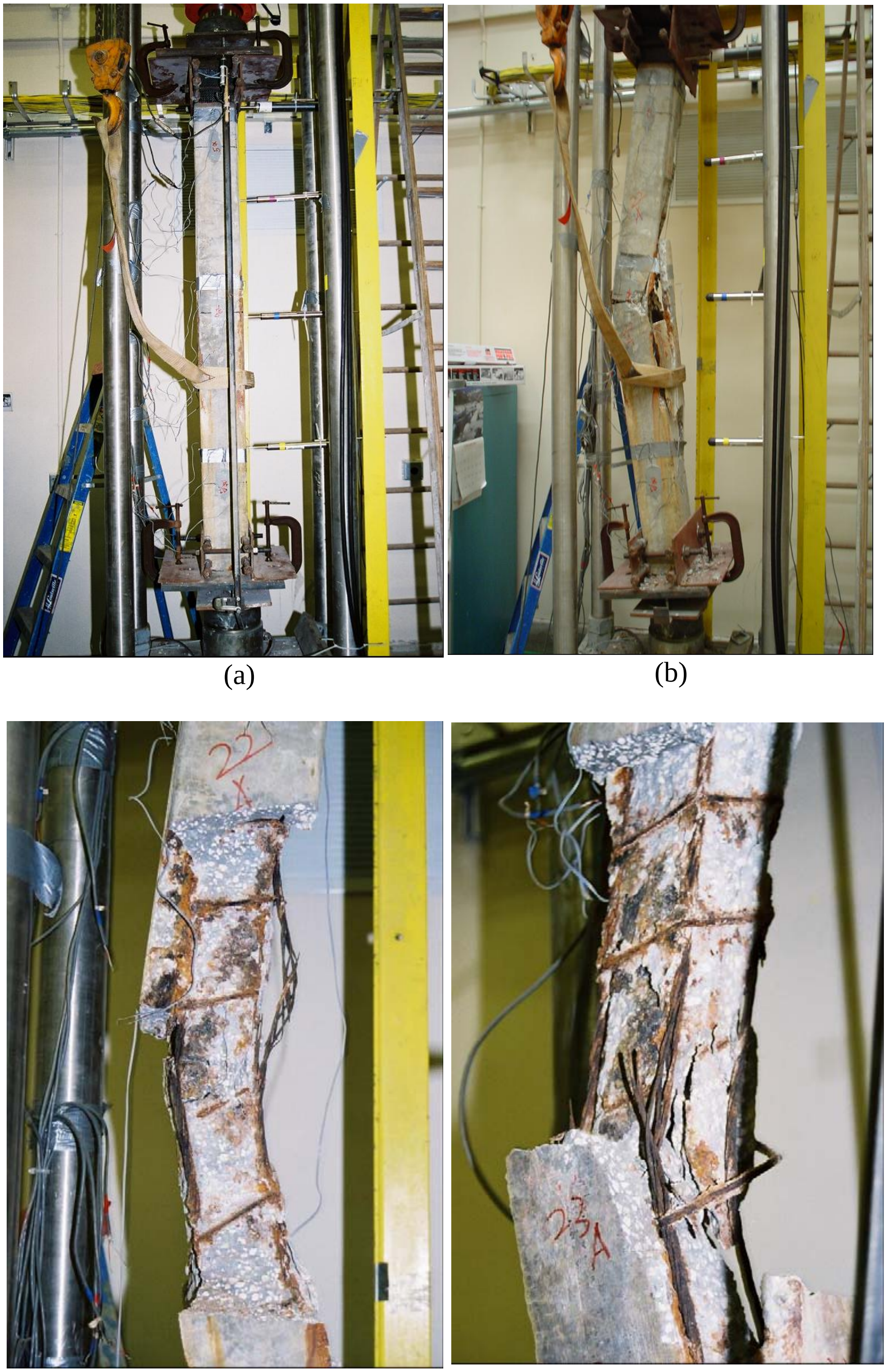


Figure 8.20 Change of Load Capacity

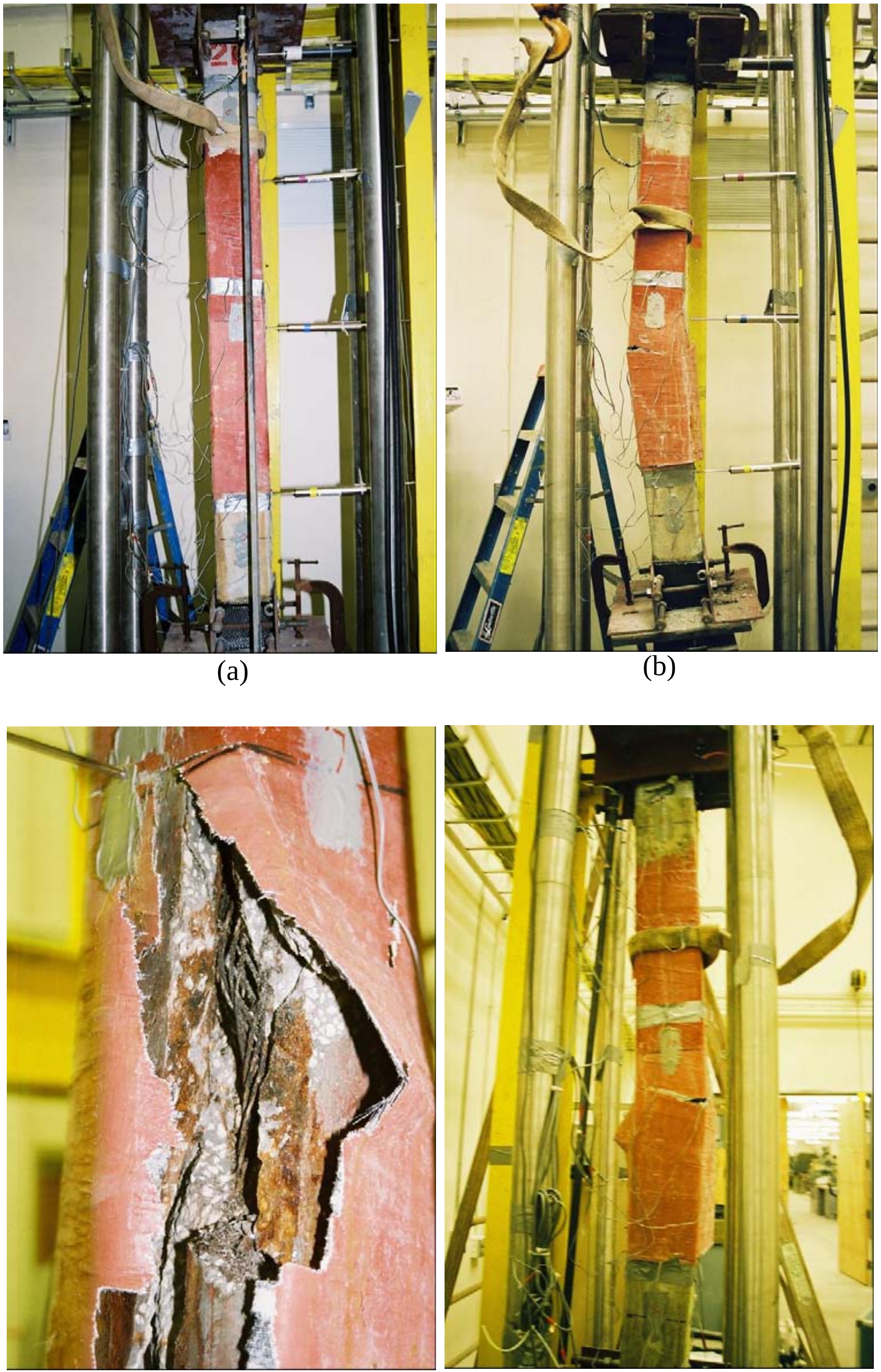
Photos showing the failure mode of unwrapped specimens at 50% steel loss appear in Fig. 8.21. Ties in the mid-area were completely broken off and strands were severely corroded. The figure also shows the failure mode of the wrapped specimens. As before, FRP on the compression side was torn in both the longitudinal and lateral directions, while on the tension side it was only torn in the lateral direction. Surprisingly, it was found that the FRP could be easily removed from the concrete once it had cracked. Therefore, to check the bond strength between concrete and FRP, pull out test were tests were conducted. The results are summarized in following section.



(a) (b)

(c) (d)

Figure 8.21 Failure of Unwrapped Controls at 50% Steel Loss



(a) (b)

(c) (d)

Figure 8.22 Failure of Wrapped Controls at 50% Steel Loss

8.5 Gravimetric Test Using Load Tested Piles

As shown in the crack survey, corrosion of steel appeared to be dissimilar for the 5ft and 6ft specimens even though they had been subjected to the same accelerated corrosion regime. Therefore, to verify the actual steel loss, gravimetric testing was performed using the load-tested specimens. A total of five specimens were tested gravimetrically. Two specimens (#24, #25) corroded to the 25% targeted steel loss and three specimens (#23, #26, #27) corroded to the 50% targeted steel loss were tested. The results are summarized in Table 8.11. The actual steel loss in the 6 ft specimen for the 25% targeted steel loss was 16.4%. For the specimens corroded for a further 125 days, steel loss in strands in the unwrapped specimens was 24.1% showing a 47% increase in the steel loss. However, in the wrapped specimens, the steel loss was only 21.3%. Thus, accelerated corrosion does not work as well when specimens are wrapped.

(a) (b)

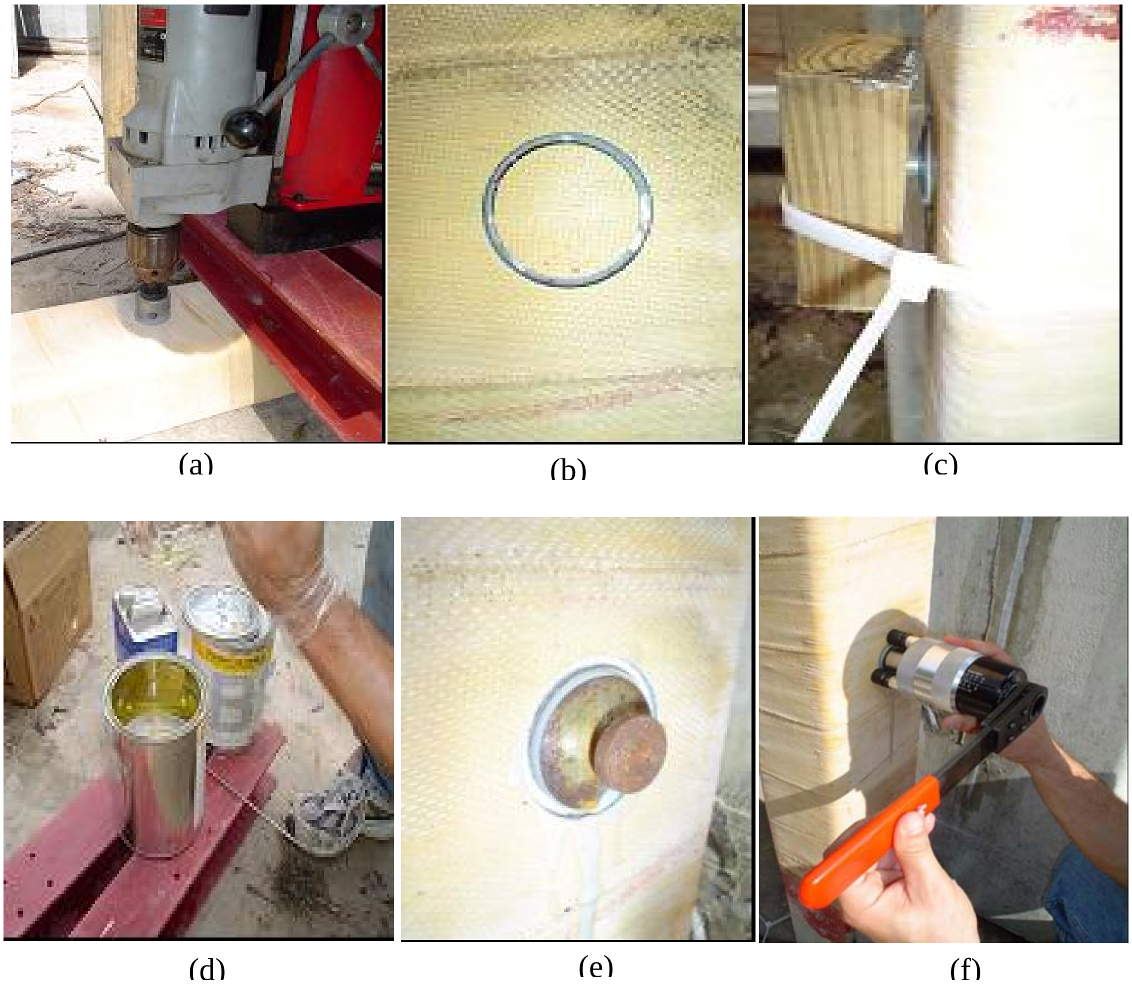
Figure 8.23 (a) Cleaned Ties; (b) Cleaned Strands of #26 Specimen

Table 8.11 Actual Steel Loss of 6ft Specimens at Each of Targeted Steel Loss

	25%	**50% (Unwrapped)**		**50% (Wrapped)**	
	Steel Loss (%)	**Steel Loss (%)**	**Increase (%)**	**Steel Loss (%)**	**Increase (%)**
Strands	16.4	24.1	+47	21.3	+29.9
Tie	19.7	NA	NA	26.2	+33.0

8.6 Pull Out Test

Pull out tests were conducted to evaluate the FRP-concrete bond using the fifth underwater wrapped specimen. An Elcometer 106 Adhesion Tester manufactured by Elcometer Instruments Ltd was used for the testing. The Elcometer 106 tester included its own dollies with 0.784 in. and 1.456 in. diameter. In this test, the 1.456 in. diameter dollies were used. The procedures used for the test was as follows and is shown in Fig. 8.24.



(a) (b) (c)

(d) (e) (f)

Figure 8.24 Pull Out Test

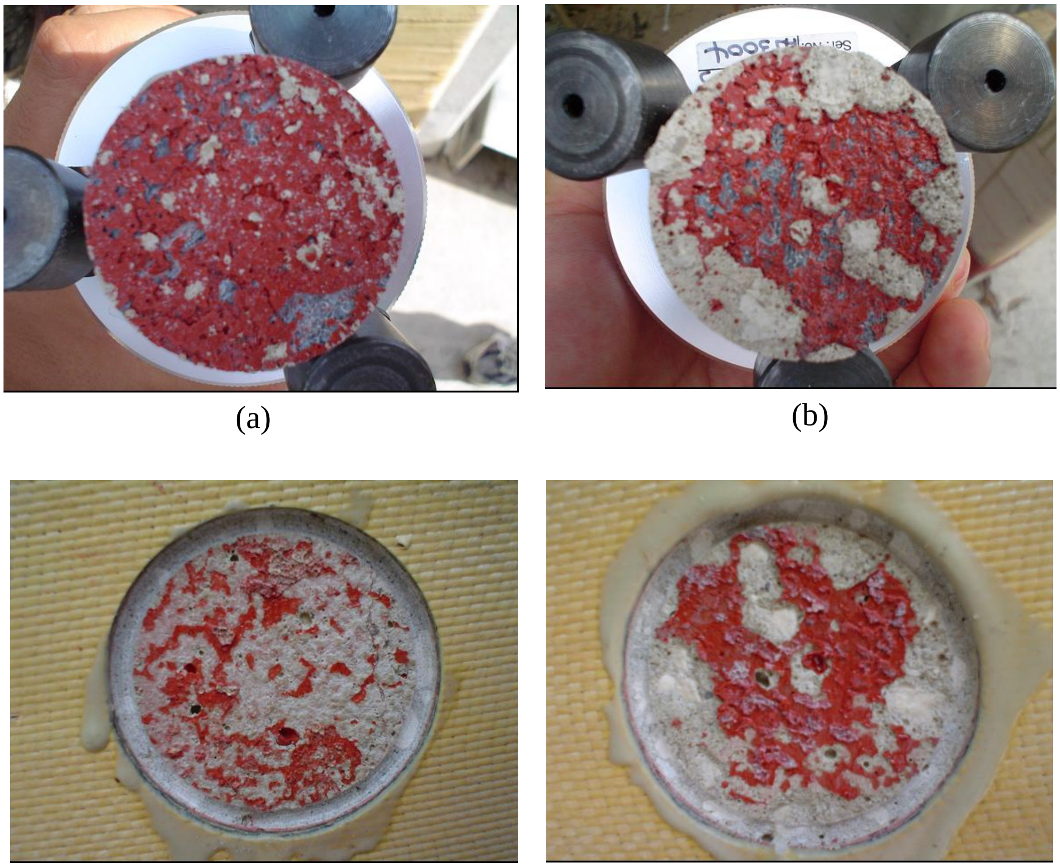
i. FRP wrap on the specimen was scored with a diamond drill bit operated by a magnetic drill. Scoring was performed on two different faces with a 1¾ in. of diameter.

ii. The surfaces of the scored FRP were cleaned with coarse sand paper and acetone.

iii. After completely drying the scored surfaces, the same amount of part A and B of Sikadur 32 hi-mod epoxy were mixed using a low speed drill for 3minutes.

iv. The mixed epoxy was applied to the surface of the dollies and bonded on the scored FRP surfaces using a wooden block and tie wrap.

v. After completely curing for 7 days, the pull out test was performed using the Elcometer.



(a) (b)

(c) (d)

Figure 8.25 Failure Mode of Concrete

The results of the tests are summarized in Table 8.12. The average bond was found to 0.13 ksi. Fig. 8.25 shows the failure mode. The FRP debonded indicating that the dolly adhesive strength exceeded the bond between the FRP and concrete surface. However, this low bond strength did not impair the ability of the FRP to strengthen the corroded columns (see Tables 8.8 and 8.9).

Table 8.12 Result of Pull Out Test

	Strength (ksi)
Test 1	0.12
Test 2	0.14
Average	**0.13**

8.7 Conclusions

Based on the information included in this chapter, following conclusions may be drawn.

1. Underwater FRP wrap are effective in increasing the capacity of corrosion damaged elements (see Table 8.13).
2. Accelerated corrosion regimes are less effective for wrapped specimens. Metal losses were significantly lower compared to unwrapped controls (see Table 8.11).
3. The bond between FRP and concrete was poor (Table 8.12) though it did provide significant strengthening in spite of this (see Table 8.13).

Table 8.13 Result Summary of Eccentric Load Test

	0%	**25%**		**50%**	
	Ultimate Load (kips)	**Ultimate Load (kips)**	**Increase (%)**	**Ultimate Load (kips)**	**Increase (%)**
Unwrapped	126.7	88.6	-30.0	79.6	-37.2
Wrapped	NA	137.6	+8.7	151.3	+19.5

References

8.1 Mullins, G., Sen, R, Torres-Acosta, Goulish, A., Suh, K., Pai, N. and Mehrani, A. "Load Capacity of Corroded Pile Bents". Final Report submitted to the Florida Department of Transportation, September 2001.

8.2 Fisher, J., Mullins, G. and Sen, R. "Strength of Repaired Piles". Final Report submitted to the Florida Department of Transportation, March 2000.

9. ALLEN CREEK BRIDGE FIELD STUDY

9.1 Introduction

This chapter provides details on a field study carried out in which full-sized prestressed piles located in tidal waters were wrapped. Two disparate systems were evaluated one in which cofferdam construction was used and one where the wrap was directly wrapped in water. The wrap using cofferdam construction was carried out by SDR Engineering. Their report is included as Appendix C. This chapter focuses on the underwater wrap where cofferdam construction was not used and piles were directly wrapped in water using a recently developed water-activated resin. A description of the test program is presented in Section 9.2. Information on the pre-repair assessment is included in Section 9.3 while instrumentation details are contained in Section 9.4. The procedure for underwater wrapping is presented in Section 9.5 while preliminary results on the corrosion measurements are summarized in Section 9.6. Preliminary conclusions are contained in Section 9.7.

9.2 Test Program

A field demonstration study was conducted to assess the feasibility of two alternate systems (1) a "dry" wrap requiring cofferdam construction, and (2) a "wet" wrap that could be directly carried out in water. The structure selected for this field demonstration project was the Allen Creek Bridge (#150036) located in Clearwater, Florida (Fig. 9.1). It met critical access requirements, e.g. shallow waters, proximity to the university, yet provided an aggressive environment with a long history of severe substructure corrosion problems in piles.

Figure 9.1 Allen Creek Bridge

Allen Creek Bridge is located on the busy US 19 highway 1.5 miles north of SR 686. Originally constructed in 1951, it was supported on 20 in. x 20 in. reinforced concrete piles. In 1982, the bridge was widened to accommodate additional traffic lanes. The widened section is supported by 14 in. x 14 in. piles that were prestressed by eight ½ in. Grade 270 stress relieved strands.

Fig. 9.2 shows the plan view of the Allen Creek Bridge. All piles are spaced 15 ft apart in the North-South direction and 6.5 ft apart in the East-West direction. A total of ten 14 in. ×14 in. prestressed piles located on the East side of the bridge were selected for the study. Details are summarized in Table 9.1 and their location identified in Fig. 9.2. Two piles of the ten piles B1 and G1 were used as controls. Another four piles E1, E2, F1, F2) were wrapped using the Aquawrap system of AirLogistics Inc. The remaining four piles C1, C2, D1, D2 were repaired by MAS2000 CFRP wrap system developed by SDR Engineering. Details of their study may be found in Appendix C.

Table 9.1 Specifications of Test Piles

Name	Type	Wrap	Layers	Instrumentation
B2	Control	No	No	3
C1	Mas2000	CFRP	2	3
C2	Mas2000	CFRP	2	0
D1	Mas2000	CFRP	4	4
D2	Mas2000	CFRP	2	0
E1	Aquawrap	GFRP	4	0
E2	Aquawrap	GFRP	4	4
F1	Aquawrap	CFRP	2	4
F2	Aquawrap	CFRP	2	0
G1	Control	No	No	4

An elevation view of the bridge is shown in Fig. 9.3. The waters from the creek flow east into Old Tampa Bay that in turn joins the Gulf of Mexico to the south. The environment is very aggressive; all the reinforced concrete piles from the original construction have been rehabilitated several times. At low tide, the water level in the deepest portion of the creek was about 2.6 ft. Maximum high tide is about 6 ft. This shallow depth meant that the underwater wrap would not require divers and could be carried out on a ladder.

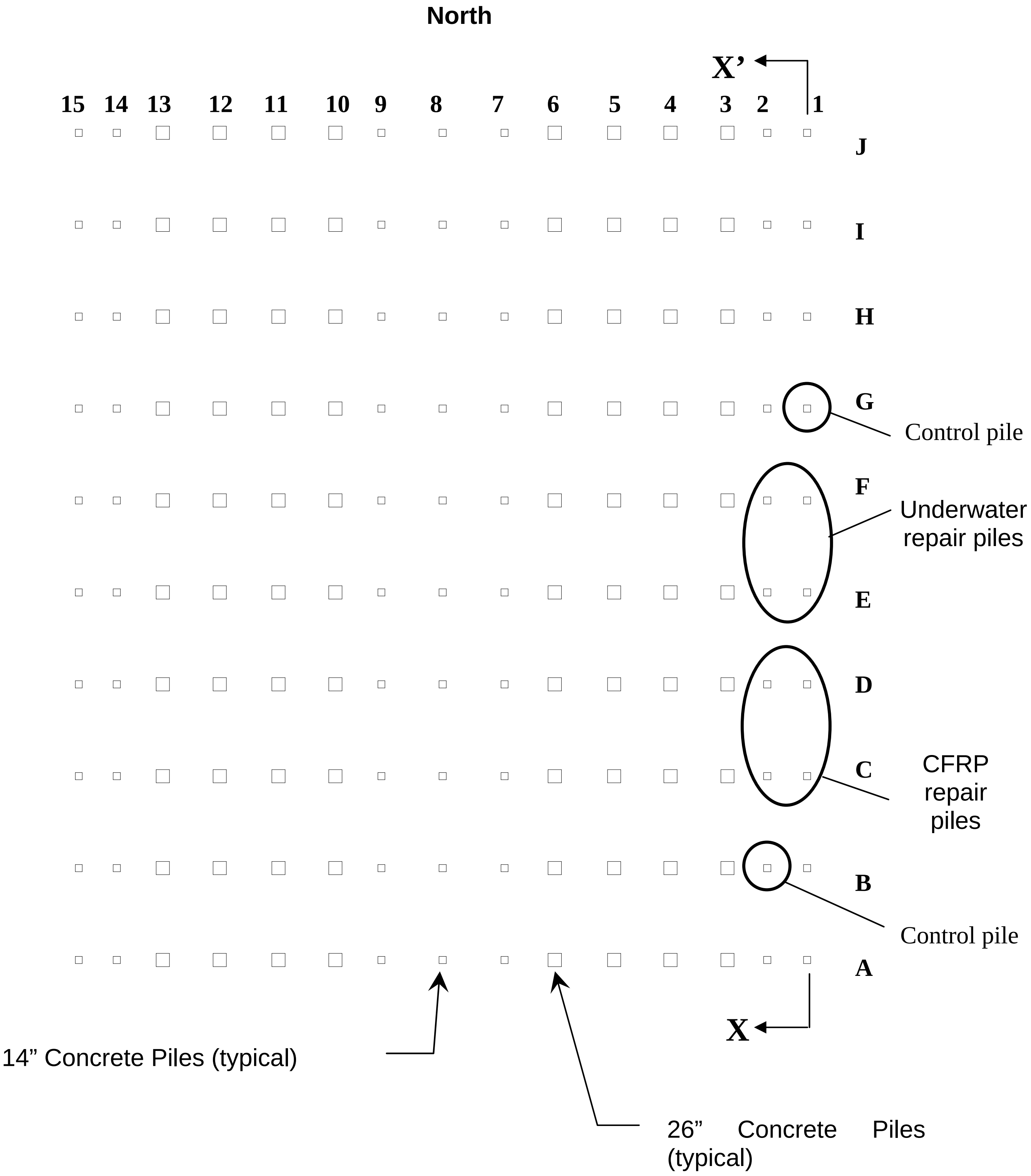


Figure 9.2 Plan View of Allen Creek Bridge

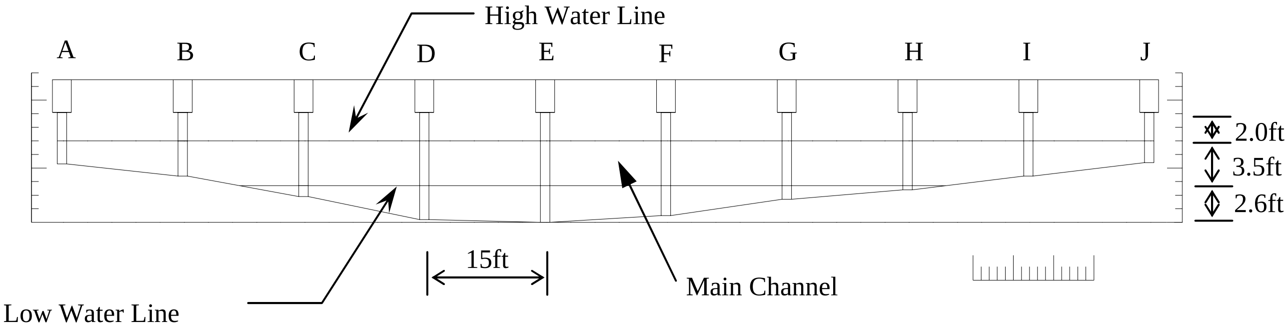


Figure 9.3 Allen Creek Bridge Looking West.

9.3 Initial Condition

A preliminary inspection of the piles revealed no visible signs of corrosion. To evaluate the internal corrosion status of the piles, delamination tests was performed using a hammer (Fig. 9.4). However, no hollow sound could be detected in any of the piles. To evaluate the initial corrosion state, several piles were instrumented to allow half-cell potential and the corrosion rate to be assessed.

Field instrumentation for long term monitoring is never a simple task. It is especially so for piles because of accessibility problems at high tide and the aggressive marine environment. Particular attention must be paid to ensure that electrical contacts are not corroded and moisture is not trapped inside junction boxes. Moreover, accessibility of instrumentation ports makes them vulnerable to vandalism. Thus, making instrumentation as unobtrusive as possible is a basic requirement for successful long-term monitoring. In the study, a system suggested by Sagues 2002 was used. It is simple to install and does not require any wiring or corrosion protection (Fig. 9.5).

Figure 9.4 Checking for Delamination

Two concrete cores were taken to the level of the steel to determine the chloride variation. The first sample was at the elevation corresponding to high tide. The second was 3 ft above high tide. The total chloride was determined at every inch down to the level of the steel by Florida Department of Transportations's State Materials Office. Results that they provided are summarized in Table 9.2 [9.2].

Inspection of Table 1 indicates that the chloride threshold for corrosion was easily exceeded at the high tide location. The chloride level varied between 6.71-5.59 lb/cy. Values were much greater 3 ft above high tide where it was 12.53 lb/cy in the initial inch of cover and reducing to 0.86 lb/cy in the vicinity of the prestressing steel. This is typical of chloride variation observed in specimens exposed to tidal waters – it is always much higher above the high tide region. This information is useful in assessing the extent of the pile wrap above the high water line. Since encapsulation of chloride contaminated regions sometimes leads to an upward shift in the corrosion cells [9.3].

Table 9.2 Chloride Profile in Allen Creek Bridge

Pile	Elevation	Depth	Test #1	Test #2	Test #3	Avg. Cl^-	Cl^- Range
C-2	0 AHT	0-1"	6.842	6.505	6.780	6.71	0.337
C-2	0 AHT	1-2"	6.215	5.982	6.049	6.08	0.233
C-2	0 AHT	2-3"	5.708	5.479	5.594	5.59	0.229
C-2	3 AHT	0-1"	12.254	12.531	12.809	12.53	0.555
C-2	3 AHT	1-2"	4.589	4.753	4.738	4.69	0.164
C-2	3 AHT	2-3"	0.846	0.866	0.866	0.86	0.020
D-1	0 AHT	0-0.75"	9.493	9.459	9.459	9.47	0.034
D-1	0 AHT	0.75-1.5"	6.506	6.449	6.506	6.49	0.057
F-1	0 AHT	1.5-2"	**2.939**	**2.883**	**2.880**	**2.90**	**0.059**
F-1	3 AHT	1.5-2"	**5.581**	**5.401**	**5.603**	**5.53**	**0.202**

AHT - Above High Tide
Chloride content units = lb/yd^3 of concrete
Chloride range is a test calibration value
Chloride content represents total chloride

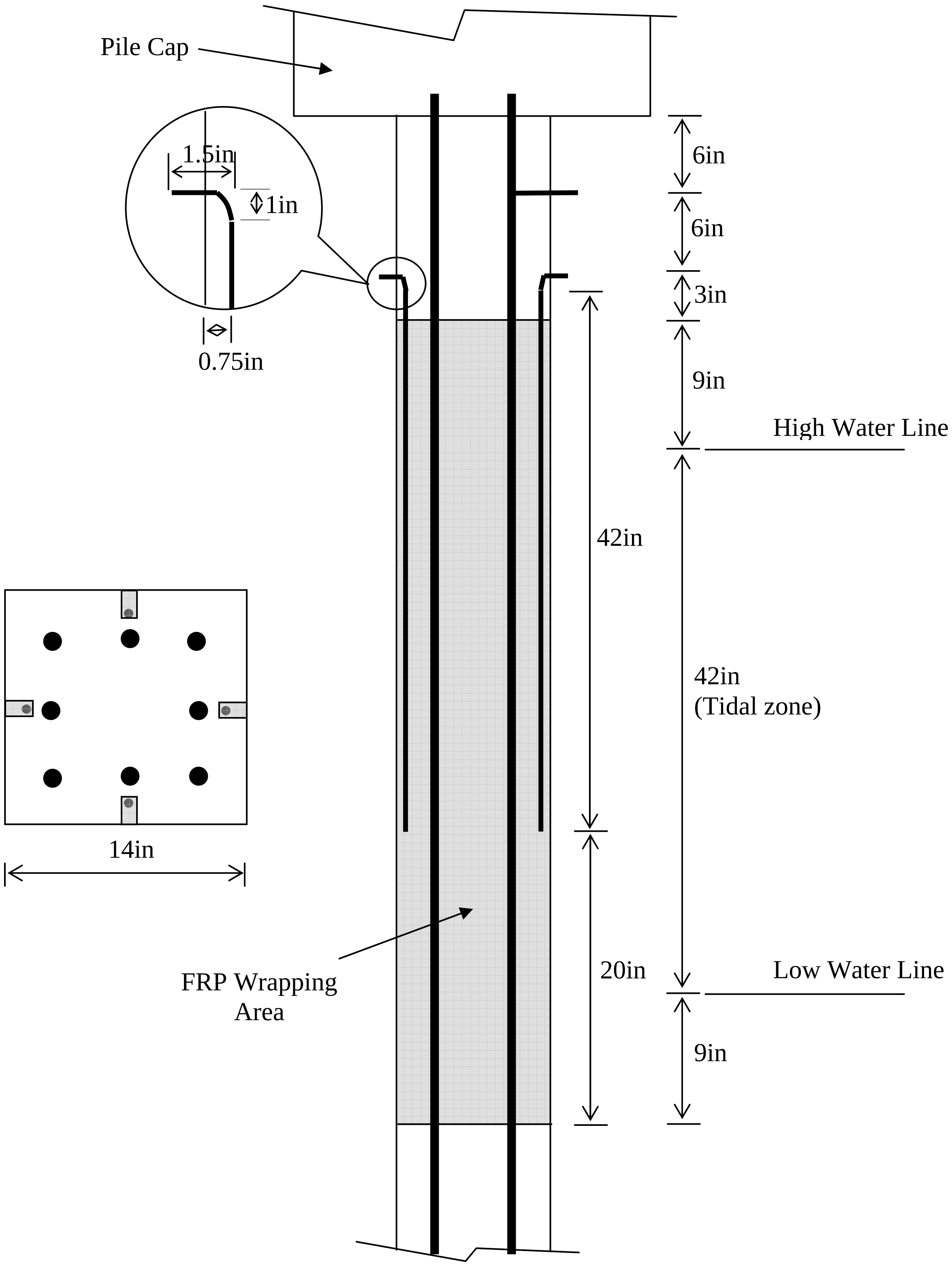


Figure 9.5 Instrumentation Details

9.4 Instrumentation

A total of six piles (B2, C1, D1, E2, F1, G1) were instrumented for monitoring potential and corrosion rate using the system shown in Fig. 9.5. To meet criteria described in the previous section, a simple system was used. In this system, 42 in. long, 3/16 in. diameter, 316 stainless steel bars were embedded in each pile face. A ¼ in × 45 in × 7/8 in groove was made using a grinder on the four surfaces of the piles that were instrumented. The stainless steel bars were inserted in the groove and mortar was used to close the groove (Fig. 9.6).

Figure 9.6 Installing Stainless Steel Rods

Although all strands could be physically connected to one another by ties, four grounding bars were installed on each of the four faces of the piles to ensure electric continuity. A 2 in. diameter, 3 in. deep hole was cored on the surface of the concrete using a center-hole drill. A four inch length of 316 stainless steel bar were brazed on to the strand. The holes were filled with mortar. Details are shown in Figure 9.7.

The instrumentation system allowed linear polarization and half cell potential measurement to be carried out. These were performed once a week prior to wrapping and once every two weeks after wrapping. To measure the half-cell potential, the positive terminal of a high impedance voltmeter was connected to the all strands (via alligator clips connecting to the installed grounding rod). A copper-copper sulfate reference electrode was connected to the negative terminal of the voltmeter. Potential measurements were made along each face using a 6 in. grid (Fig. 9.8). After wrapping, potential measurements could only be made in the 12 in. unwrapped portion of the pile below the pile cap.

Figure 9.7 Strands Grounding

Figure 9.8 Measuring Half-Cell Potential

Figure 9.9 Performing Linear Polarization Measurement

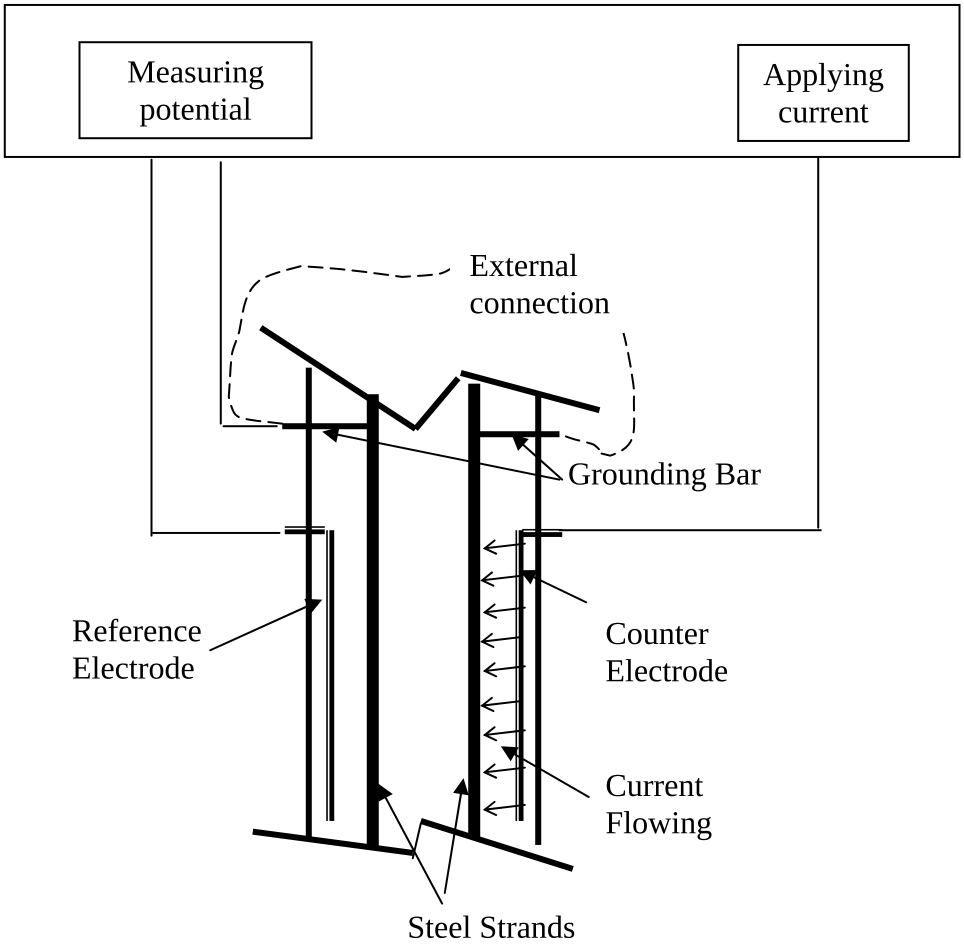


Figure 9.10 Schematic Setup for Linear Polarization Measurements Using USF Probe

The PR Monitor manufactured by Cortest Instrument Systems, Inc. was used to perform linear polarization measurements using a three-electrode probe comprising a reference, working, and counter electrode. PR monitor measures the polarization resistance of electrochemical system in the pile. As the polarization resistance is inversely proportional to the corrosion rate, the corrosion rate of steel can be estimated.

Concrete resistivity was measured using a soil resistance meter (Nilson 400). One of four stainless steel bars embedded was used as a reference electrode and the other three were used as counter electrodes. Therefore, the polarization test was conducted four times per pile changing the reference electrode in turn and averaging the data (Fig. 9.9 and 9.10). Assuming that only the same length of prestressed steel strand with counter electrode was polarized (46 in. length), the polarized steel area was calculated as 4534 cm^2. Details about calculating the corrosion rate are shown in appendix **.

9.5 Aquawrap Repair System (AirLogistics)

Before wrapping, holes and chipped concrete were filled using hydraulic cement (Fig. 9.11). Depressions on the pile surface were filled with cement paste and sharp edges were rounded using a air pressure operated grinder (Fig. 9.12). All dust and debris were cleaned by pressure washer.

Figure 9.11 Filling Holes with Hydraulic Cement

The Aquawrap is a pre-preg system developed by Air Logistics Inc. It uses a unique water activated urethane resin that allows FRP wrapping in water. Properties of all materials used by the Aquawrap Repair System are summarized in Chapter 3.

Two types of fibers - carbon and glass - were used. In this system, both unidirectional and bi-directional fibers were used. The unidirectional fibers were applied to increase axial capacity and the bi-directional fibers were used to add both longitudinal and transverse capacity. The capacities of the fibers were selected to match those used for the alternate repair using a cofferdam system (in Appendix C).

Figure 9.12 Grinding Edges

Two layers of carbon fiber were applied to pile F1 and F2. The wrapping procedure is given below (Fig. 9.13):

i. Mix base primer parts A and B.
ii. Apply mixed primer to the concrete surface by hand.
iii. Apply one layer of a 12 in. × 60 in. unidirectional carbon fiber pre-preg strip longitudinally on each face.
iv. Apply mixed primer to the surface of the unidirectional carbon fiber.
v. Apply bi-directional pre-preg carbon fiber spirally for two continuous layers without overlap.
vi. Apply one layer of a glass fiber veil to provide a smooth finish.
vii. Apply a plastic wrap and make tiny holes to allow gaseous products formed during curing to escape.
viii. Remove plastic wrap after curing for one day and apply mixed primer to the surface of glass fiber veil to provide UV protection.

Four layers of GFRP were applied to E1 and E2. The procedures used was identical excepting that greater number of GFRP layers were required. It is described below (Fig. 9.14):

i. Mix base primer parts A and B
ii. Apply mixed primer to the concrete surface
iii. Apply two layers of 12 in × 60 in unidirectional glass fiber pre-pregs to each of the four faces of the pile
iv. Apply mixed primer to the surface of the unidirectional glass fiber.
v. Apply 12 in, wide bi-directional glass fiber pre-preg spirally for four continuous layers.
vi. Apply one layer of glass fiber veil to provide a smooth finish.
vii. Apply a plastic wrap and puncture holes to allow gaseous products to escape.
viii. Remove plastic wrap after curing for one day and apply mixed primer to the surface of glass fiber veil to provide UV protection.

Figure 9.13 Underwater Wrapping (CFRP)

Figure 9.14 Underwater Wrapping (GFRP)

9.6 Test Results

9.6.1 Pre-Wrap Results

Half cell potential measurements using copper-copper sulfate reference electrodes were made once every two weeks. Stainless steel bars brazed to the steel strands were electrically connected using alligator clips and connected to a positive terminal of voltmeter. A reference electrode placed on the concrete surface was connected to a negative terminal of voltmeter. Potential reading were performed on each face using a 6 in grid.

Fig. 9.15 shows the variation of the half-cell potential (vs CSE) measured 4 ft below the pile cap before wrapping. All piles showed values that were more negative than –350mV indicating there was a 90% probability that corrosion was occurring (ASTM C-87-91). Fig. 9.16 shows the distribution of the initial half-cell potential values measured on the south face of each pile. Values are lower near the top but increase with depth.

Pre-wrap corrosion rates were also determined from linear polarization. Rates were small (less than 0.2 mils per year). They are shown with the post-wrap corrosion rate plots later for convenience.

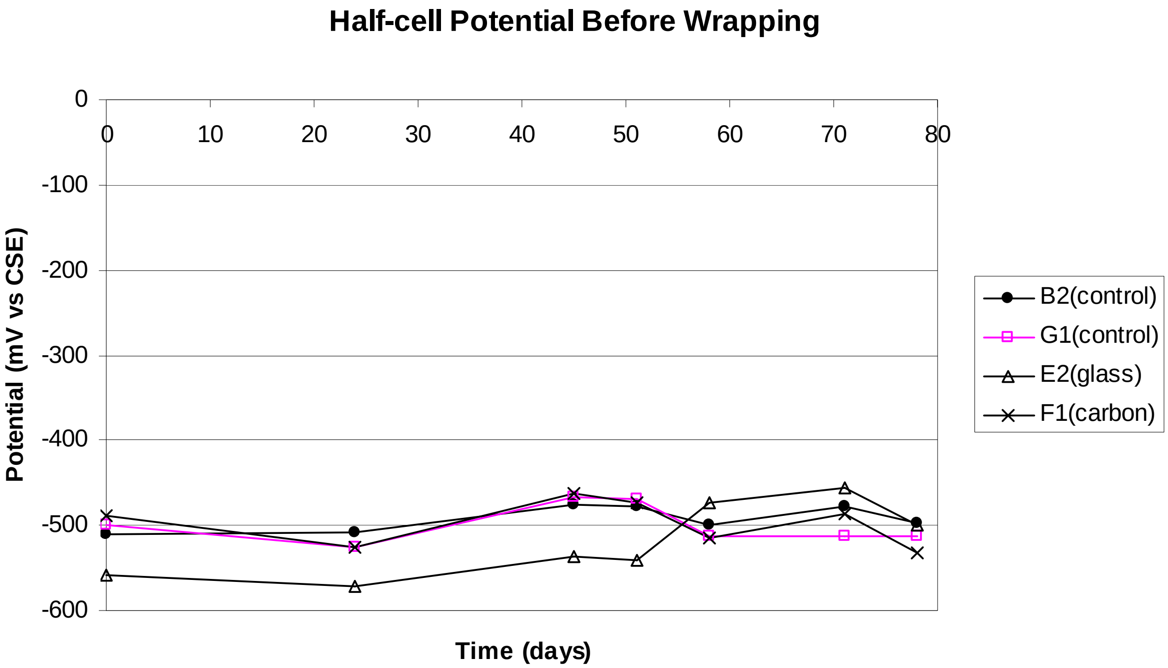


Figure 9.15 Half-Cell Potential 4 ft From the Pile Cap Before Wrapping

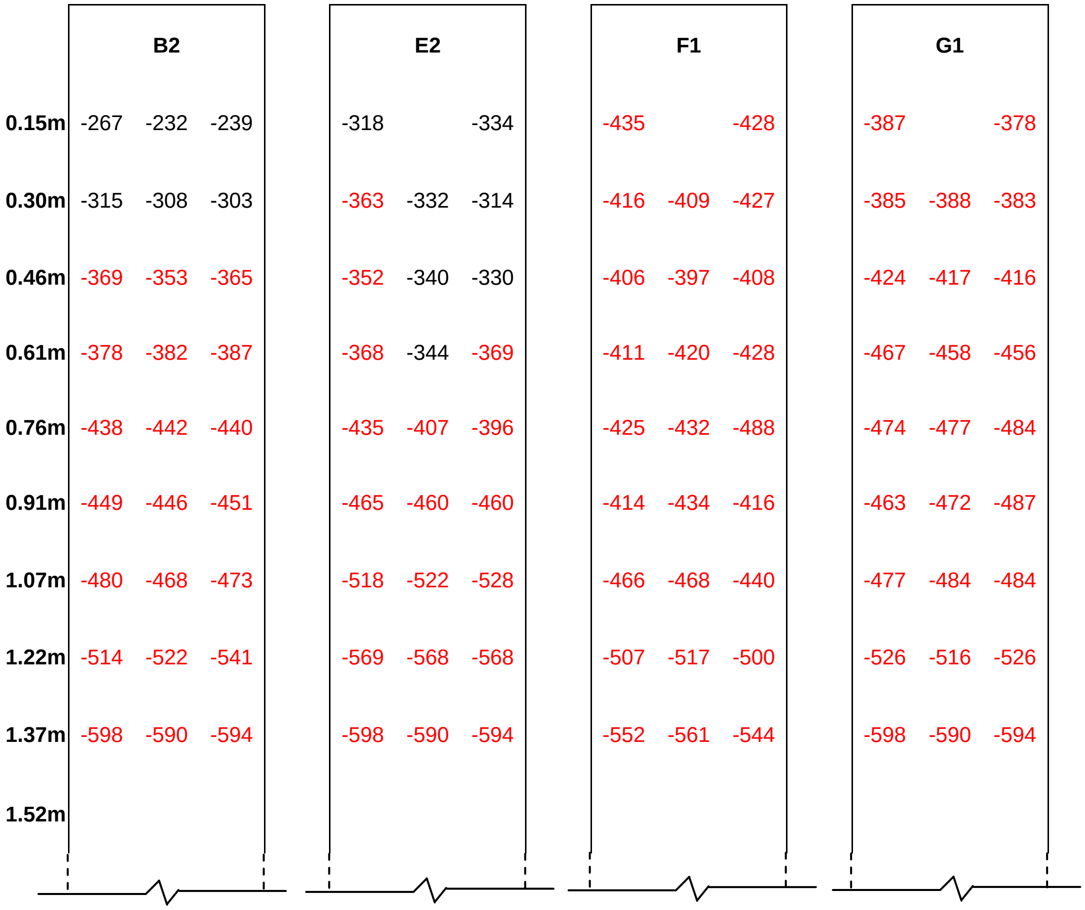


Figure 9.16 Initial Half-Cell Potential Map For South Face (units: mV)

9.6.2 Post Wrap Results

An overview of the results of monitoring the corrosion rate in the instrumented piles over 900 days is shown in Fig. 9.17-18. These compare the corrosion rates in the dry-wrapped (SDR - Appendix C) system (Fig. 9.17) and the wet-wrapped (Aquawrap) system (Fig. 9.18) with that of the controls. In both cases, the variation in the ambient temperature at the time of the reading is shown in the same plots.

Inspection of Fig. 9.17 shows that while all the piles were at a similar initial corrosion state, the corrosion rate (in mils – 0.001 in per year) in the dry-wrapped piles were consistently lower following wrapping. The readings have remained stable and there was no significant difference in the performance of the piles wrapped using two or four carbon layers. However, the corrosion rate for both controls showed a great deal of fluctuation that did not seem closely related the variation in the ambient temperatures. For the wet-wrapped system (Fig. 9.18), the corrosion rates for carbon matched those for the dry-wrapped system in Fig. 9.17. However, the performance of the fiberglass was much poorer and comparable to that of the unwrapped controls. This is shown clearly in Fig. 9.19 and 9.20 that compare the two systems with each other and the controls.

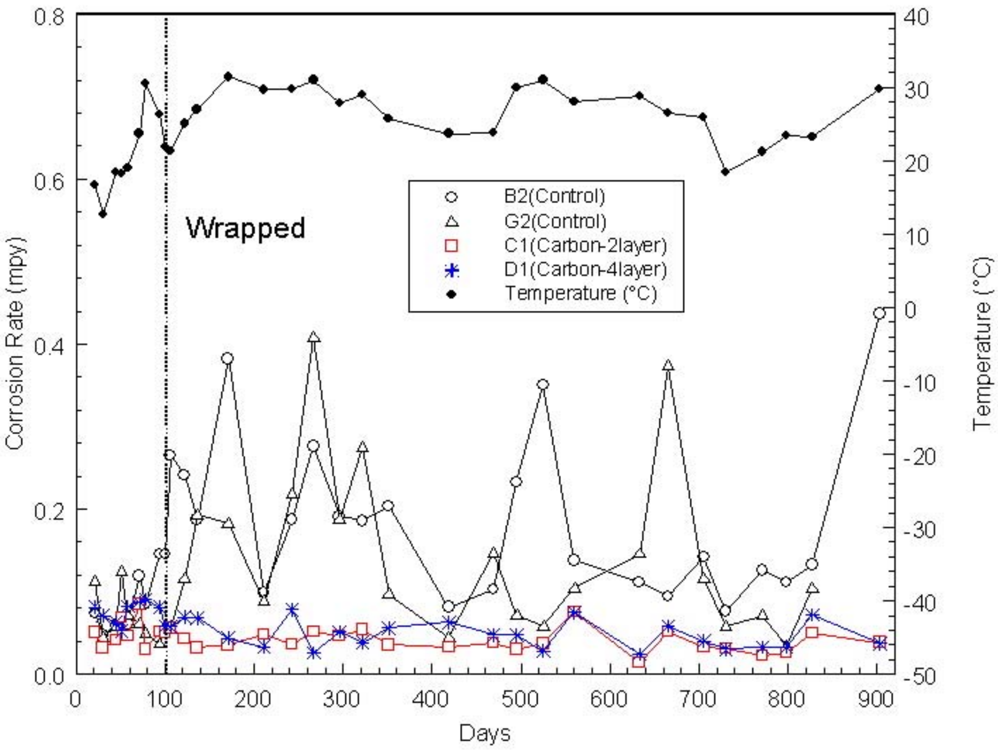


Figure 9.17 Corrosion Rate Measurements in Dry-Wrapped Piles

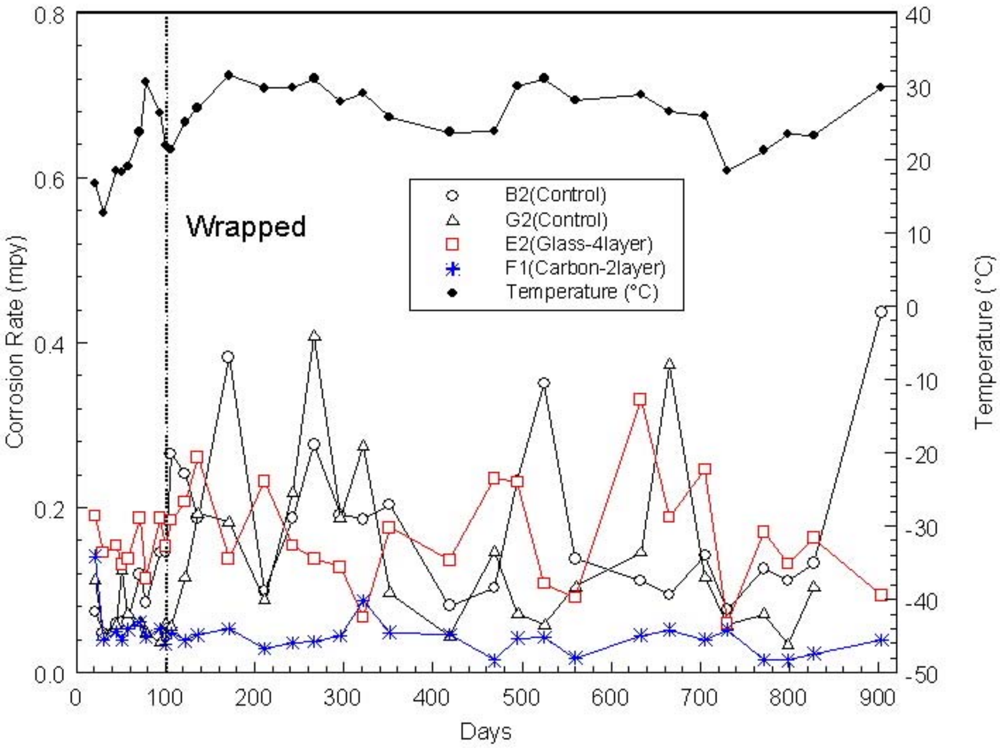


Figure 9.18 Corrosion Rate Measurements in Wet-Wrapped Piles

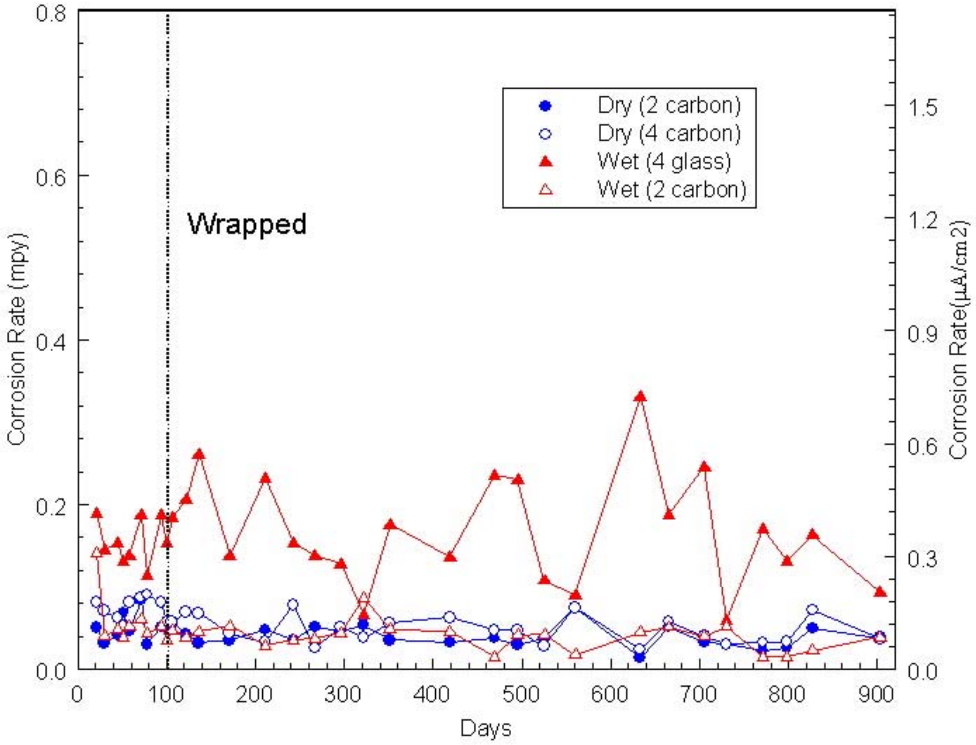


Figure 9.19 Comparison of Dry and Wet-Wrapped Systems

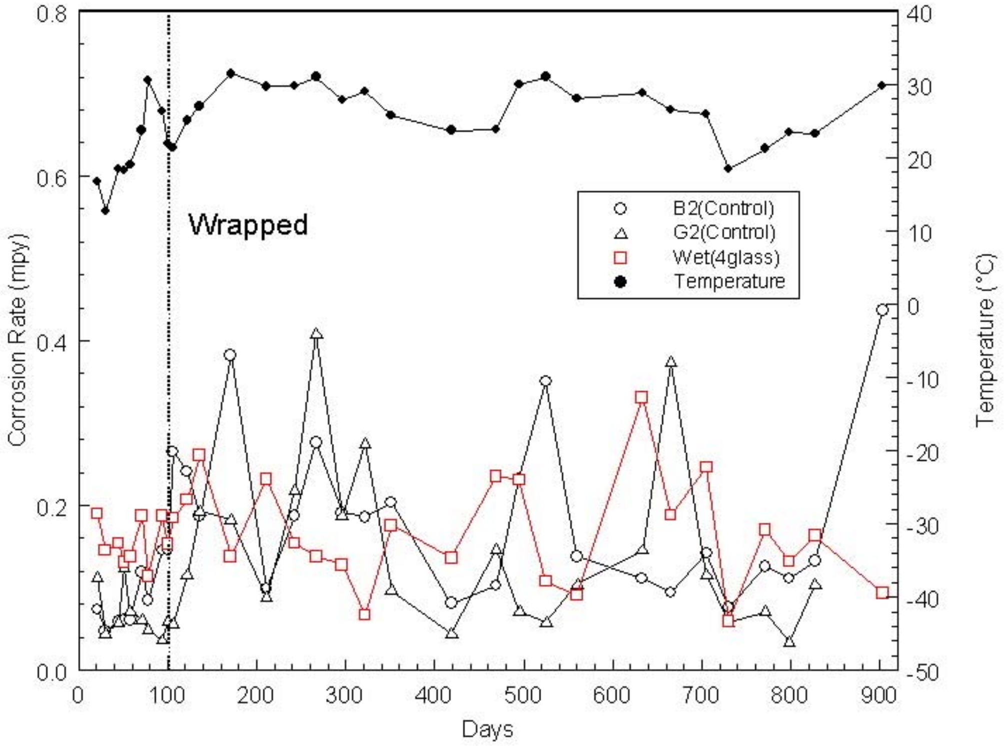


Figure 9.20 Comparison of Corrosion Rate of Wet-Wrap Glass and Controls

The corrosion rate in the wet-wrapped fiberglass pile was slightly higher initially compared to the others. Over the last hundred days (summer 2005) however, the rates were reducing though they are still higher than the other wrapped piles. It should be noted that the rates for all the wrapped and the unwrapped piles are still small. According to Clear [9.3], these rates signify that corrosion is possible in 10-15 yrs time.

9.6.3 Bond Tests

Two series of pullout tests were conducted to evaluate the FRP/concrete bond. In both series, an Elcometer106 adhesion tester was used in conjunction with a 1.456 in diameter dolly.

In the initial series, on-site pull-out tests were performed on witness panels in two piles (H1, I1) (Fig. 9.21). These specially created 2 ft wide panels were located in the dry upper part of the pile during the original wrapping operation using both carbon and glass fiber. The second series was conducted 26 months after the piles had been wrapped.

Figure 9.21 Pull-Out Test on Witness Panels.

9.6.3.1 Witness Panel Tests

The FRP witness panels on the east and west faces of the piles were scored using a 1¾ in. diameter diamond drill bit operated by a magnetic drill. The surfaces of the scored FRP were cleaned using coarse sand paper and the dust was removed with clean water. Sikadur 32 Hi-Mod was used for bonding dollies to the surface of the scored FRP. The same amount of Part A and B were mixed for 3 minutes using a low speed drill. A pre-drilled wood block into which the dolly could be fitted and a tie wrap were used to secure the dollies in place as the epoxy cured. Bond tests were carried out after the epoxy had cured for 7 days.

Table 9.3 summarizes the results of the pullout tests. The bond of FRP to the concrete substrate was found to be very poor. The scored FRP debonded by itself from concrete surface in three of the four test regions. In the other test area, there was debonding between fibers. The results are shown in Table 9.3.

Table 9.3 Summary of Bond Test Result on Witness Panel (unit:psi)

Pile	East	West
H1 (glass)	0	0
I1 (carbon)	0	72.5

Figure 9.22 Wrapped Piles Covered with Marine Growth.

9.6.3.2 Long Term Bond Test

Since marine growth developed within a few months of the wrap, a second series of tests were conducted in May 2005, more than 26 months after the original wrap using the same sized dollies and the Elcometer 106 Adhesion Tester. A total of four piles were selected for the test to encompass both the “dry” (SDR – Appendix C) and “wet” wraps.

For testing the dry-wrap repair, piles C2 (2 layer carbon) and D1 (4 layer carbon) were selected. For testing the wet-wrap systems, piles E1 (4 layer glass) and F2 (2 layer carbon) were selected. The tests were conducted on two faces per pile at two different levels – in the dry and the tidal region (Fig. 9.23). Instead of Sikadur 32 Hi-Mod epoxy used in the first series, a faster curing epoxy (Power-Fast+) manufactured by Powers Fasteners, Inc. was used for bonding the dollies to the FRP. This took 15 minutes to dry and cured in 24 hours to provide a maximum bond strength of 3000 psi.

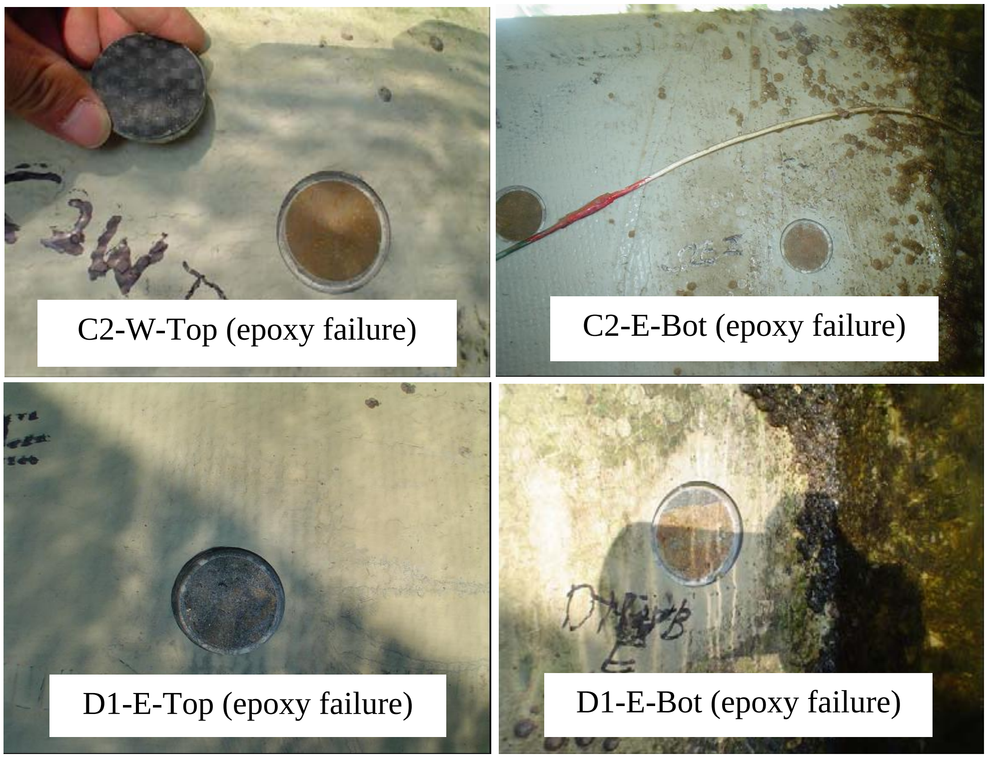
The results of the tests are summarized in Table 9.4. Of the sixteen tests conducted, 13 were epoxy failures and the remaining 3, layer failures. Epoxy failures refer to failures where the dolly separates from the concrete at its interface. This type of failure occurred in the dry system (Fig. 9.24). In layer failure, one FRP layer separates from its adjoining layer indicating that the bond between the FRP layers was poorer than its bond to concrete. Such failures only occurred in the wet system. Both types of failures are indicative of poor bond in systems that are referred to as ‘bond-critical’, i.e. where the performance of the FRP relies exclusively on its bond to the concrete substrate.

Figure 9.23 Bond Test in Progress.

Table 9.4 Summary of Bond Test Result (unit:psi)

Name	**Type**	**Face**	**Top**	**Bottom**
C2 Dry	2 layer Carbon Mas2000	East	188.5 Epoxy	58.0 Epoxy
		West	188.5 Epoxy	174.0 Epoxy
		Average	**188.5**	**116.0**
D1 Dry	4 layer Carbon Mas2000	East	304.4 Epoxy	145.0 Epoxy
		West	362.4 Epoxy	43.5 Epoxy
		Average	**333.4**	**94.2**
E1 Wet	4 layer Glass Aquawrap	East	72.5 Epoxy	29.0 Layer failure
		West	29.0 Layer failure	29.0 Layer failure
		Average	**50.7**	**29.0**
F2 Wet	2 layer Carbon Aquawrap	East	116.0 Epoxy	29.0 Epoxy
		West	145.0 Epoxy	0.0 Layer failure
		Average	**130.5**	**14.5**

Note: Term epoxy, layer refers to failure mode
Epoxy failure refers to separation of the FRP from the concrete indicating poor bond
Layer failure refers to separation of FRP layers indicating the inter-layer bond was poorer than the FRP concrete bond.



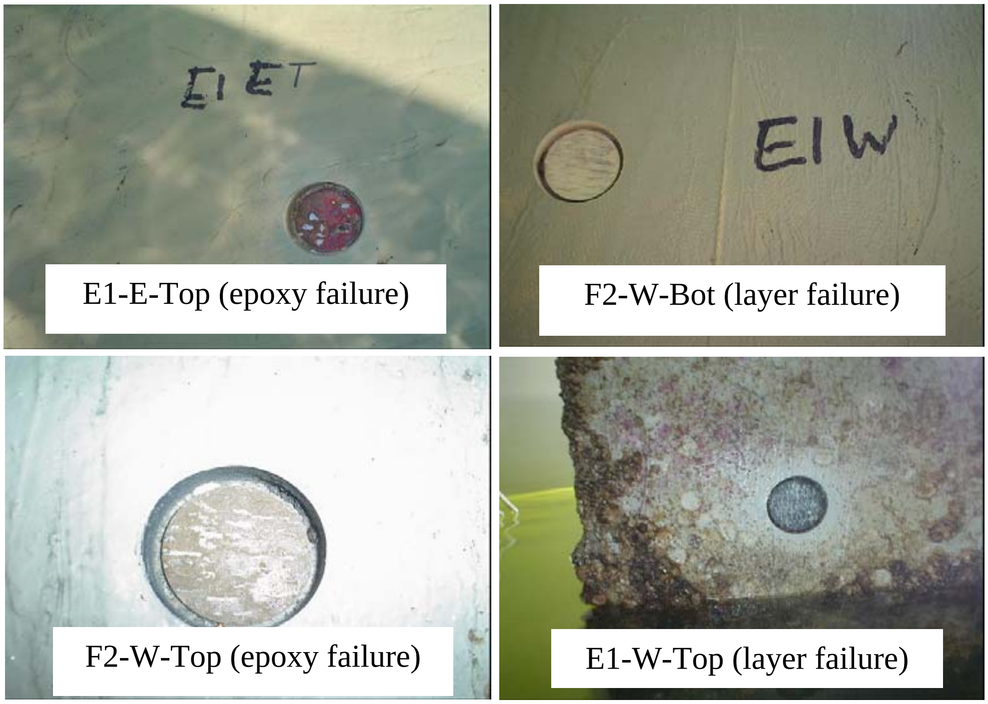
C2-W-Top (epoxy failure)

C2-E-Bot (epoxy failure)

D1-E-Top (epoxy failure)

D1-E-Bot (epoxy failure)

Figure 9.24 Bond Tests at Dry-Wrap Repaired Piles



E1-E-Top (epoxy failure)

F2-W-Bot (layer failure)

F2-W-Top (epoxy failure)

E1-W-Top (layer failure)

Figure 9.25 Bond Tests at Wet-Wrap Repaired Piles.

Inspection of Table 9.4 shows that the performance of the dry system was vastly superior compared to the wet system. Compared to the maximum 145 psi failure bond stress at the top of the pile in the wet system, the corresponding maximum for the dry system was 362.4 psi. The results also show considerable variation at the bottom, e.g. 58 psi and 174 psi.

Such wide variation was not observed in specimens cast for the ambient exposure (Table 6.3) where the same epoxy was used and specimens exposed for a longer period of nearly 3 years. In similar tests, the average ultimate bond values were also much higher (275 psi and 289 psi). This suggests that the epoxy may not have cured properly.

The bond values for the wet wrap system were quite poor especially at the bottom where the value was 29 psi and in one case, zero. The latter case was for carbon. Nonetheless, the corrosion rate measurements for the carbon wrap from both dry and wet systems were comparable (see Fig. 9.19). This suggests that in corrosion mitigation applications where the FRP is continuous over the circumference, the level of bond required is smaller compared to that needed for flexural strengthening in beams and slabs where it is only applied to one surface.

The results in Table 9.4 are plotted in the form of bar diagrams in Fig. 9.26-27. The former shows the average bond values while the latter shows the maximum values. This is included because of the large scatter in the measured data. Inspection of Fig. 9.26 shows that the average bond stresses from the wet wrap are a small fraction of that for the dry wrap. This difference is somewhat smaller when the maximum values are compared as in Fig. 9.27.

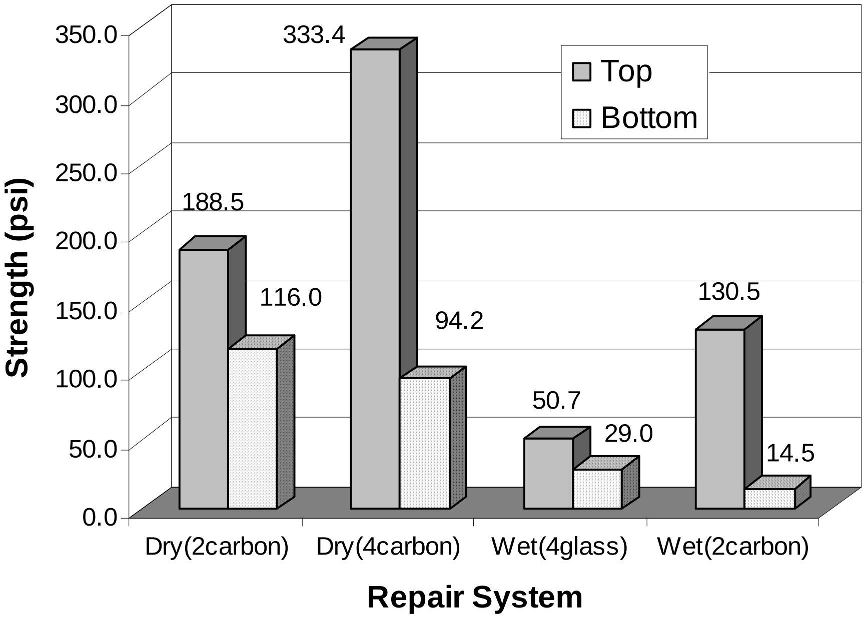


Figure 9.26 Average Bond Strength After 26 Months.

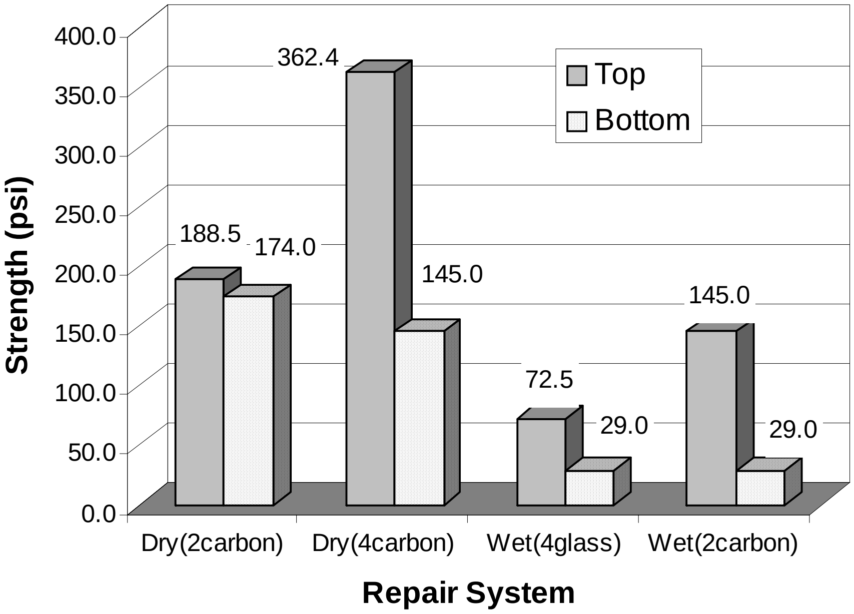


Figure 9.27 Maximum Bond Strength After 2 Years.

9.7 Conclusions

Based on the information presented in this chapter the following conclusions may be drawn:

1. Underwater FRP wrapping is viable. Surface preparation, especially grinding sharp corners under water can be problematic. Pre-preg systems with water-activated epoxies allowed piles to be wrapped in under an hour (Fig. 9.12 – 9.14).

2. The innovative instrumentation system installed for monitoring the corrosion performance of the piles works well. However, it takes time to be stabilized before the test (Fig. 9.6 – 9.10)

3. Corrosion rate measurements over 2 years indicate the corrosion rate was lower for FRP wrapped piles compared to unwrapped controls (Fig. 9.17-9.18). Readings were carbon from for both “dry” (coffer dam system in Appendix C) and “wet” wrap (underwater epoxies) were comparable (Fig. 9.19). The performance for 2 and 4 layer wraps were similar. Results from the wet wrap using fiberglass fluctuated over time and was not as good (Fig. 9.20).

4. The bond between FRP and concrete was much better for the dry wrap compared to the wet wrap (Table 9.4, Fig. 9.26-9.27). For the wet wrap, the carbon bonded better to the concrete surface than the fiber glass. It was better in the region that

was usually dry than in the submerged regions. However, all failures were epoxy failures that occurred at the FRP/concrete interface or interlayer failures (Fig. 9.24-25). Despite the much poorer bond for the wet system, the corrosion rate measurements over two years indicated that the performance of the dry and wet systems for the carbon wrap were comparable (Fig. 9.19). This suggests, bond may not be as all-important in corrosion mitigation applications where the role of the FRP is to serve as a barrier against intrusion of deleterious materials and also to contain the expansive forces set up due to corrosion.

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10. GANDY BRIDGE FIELD STUDY

10.1 Introduction

The field study on the Allen Creek Bridge was carried out in shallow waters where piles could be accessed using ladders. In view of this a second field study was undertaken in which piles supporting the Gandy Bridge (# 100300) located in the deeper waters of Tampa Bay were wrapped. In this case, only epoxies that cured underwater were used and no cofferdam construction was used. Details on the test program are presented in Section 10.2 with results of preliminary inspection performed before wrapping are described in Section 10.3. Instrumentation details are provided in Section 10.4 and information on the procedures used for the underwater wrapping is described in Section 10.5. Results on corrosion measurements obtained to date are summarized in Section 10.6 and findings from bond tests in Section 10.7. Finally, Section 10.8 contains the conclusions from this chapter.

10.2 Test Program

The Gandy Bridge provides an east-west link across Tampa Bay between Pinellas (St. Petersburg) and Hillsborough (Tampa) County. Three bridges referred to here as north, middle and south were built at different times. The north bridge now called the Friendship Trails Bridge was built in the 1950’s and is used as a recreation trail. The south bridge built in 1970’s, the subject of the FRP repair, and the middle bridge built in 1990’s are for eastbound and westbound traffic crossing Tampa Bay respectively.

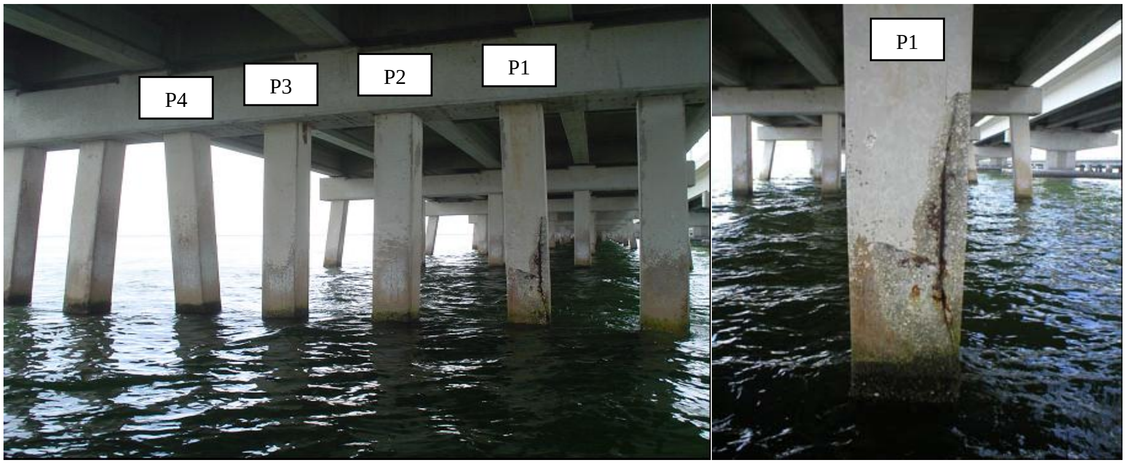


Figure 10.1 View of Pier 208 at Gandy Bridge

A preliminary survey of the south bridge was performed to select piles suitable for a demonstration project. This bridge has approximately 300 piers mostly consisting of common pile bents with five or eight prestressed concrete piles. A preliminary inspection of the bridge showed that only twenty of more than 1500 piles had cracks caused by active corrosion damage. Based on this inspection, pier 208 with the worst damaged pile (P1) was selected for this study (Fig. 10.1). `

Table 10.1 Test Program

Pile Name	Repair System	Type	Instrumentation
P1	Aquawrap® Repair System	CFRP 1+2 layers*	Yes
P2	Aquawrap® Repair System	CFRP 1+2 layers	Yes
P3	Tyfo® Wrap System	GFRP 2+4 layers	Yes
P4	Control	N/A	Yes

* signifies number of layers in the longitudinal and transverse directions respectively

Pier 208 is composed of eight 20 in. x 20 in. concrete piles prestressed by eight ½ in. Grade 270 stress relieved strands. Concrete cover was approximately 3 in. Four of the eight piles in the middle were selected for the study. Details of the four piles in pier 208 identified as P1, P2, P3 and P4 are summarized in Table 10.1. A schematic diagram showing the relative position of the piles in the piers is shown in Fig. 10.2.

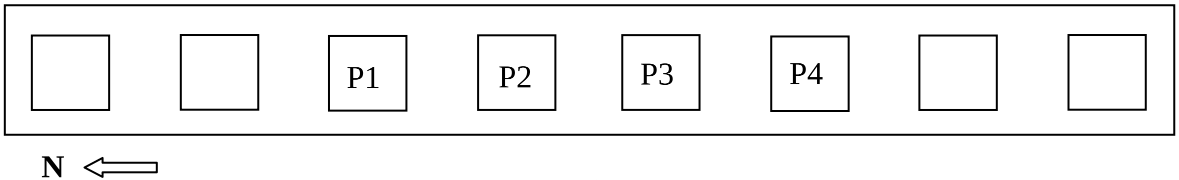


Figure 10.2 Plan View of Pier 208 at Gandy Bridge

All four piles were instrumented using two different types of probes – a rebar probe developed by FDOT and commercially available probes developed by Concorr, Inc to monitor the corrosion state. Each pile was instrumented using four rebar probes (RP-A, B, C, D) and two commercial probes (CP-T, B). One pile was used as a control and the other three instrumented piles were wrapped using two different underwater wrapping systems. The two piles (P1 and P2) were wrapped using the same carbon wrap system used in Allen Creek Bridge (# 150036) developed by Air Logistics. This comprised one layer of unidirectional fabric for axial capacity and two layers of bi-directional fabric for transverse capacity. The third pile, (P3) was wrapped using a TYFO® fiberglass system developed by Fyfe Co. LLC. This required two layers in the axial and four layers in the transverse directions to provide equivalent strengthening. All three piles were wrapped to a 6 ft length that extended 28in. above the high water line (Fig. 10.3). Details on the material properties used in this study are summarized in chapter 3.

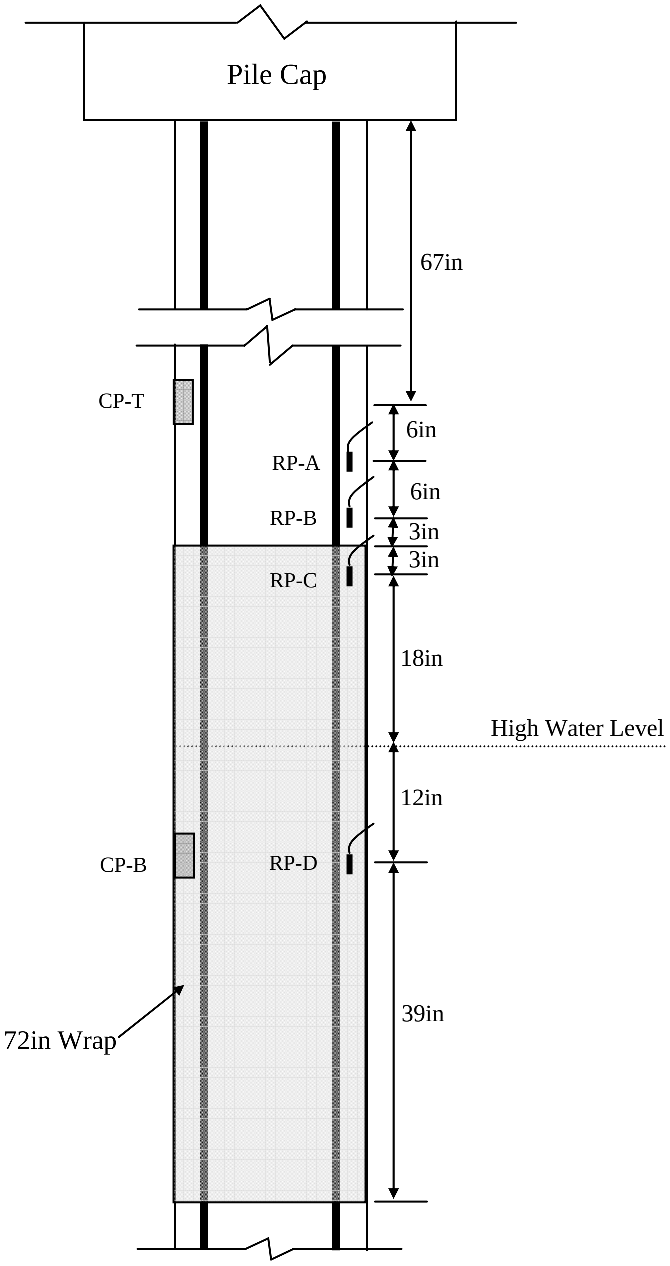


Figure 10.3 Wrap and Instrumentation Detail

10.3 Pre-Wrap Test

Pile P1 (Fig. 10.1) was severely damaged due to corrosion. There was spalling of concrete on the north-east corner and a severely corroded strand was exposed. And several cracks were found on every face excepting the south face. There were, however, no visible signs of corrosion in other three piles.

To evaluate the internal corrosion state of the piles, cores were taken to conduct a chloride content analysis. Half-cell potential measurement were made to map the corrosion potential. Furthermore, to evaluate the corrosion state, several piles were instrumented to allow the initial corrosion current and the corrosion rate to be assessed.

10.3.1 Chloride Content Analysis

Four two in. diameter, 3 in. deep cores were taken from each of the four piles for installing the rebar probes at the four different levels A, B, C, and D shown in Fig. 10.3. Using these sixteen concrete cores and additional cores, chloride content analysis was performed at the Florida Department of Transportation’s State Materials Laboratory in Gainesville. The Florida Method FM 5-516 [10.1] was used to determine chloride content.

Sample Preparation

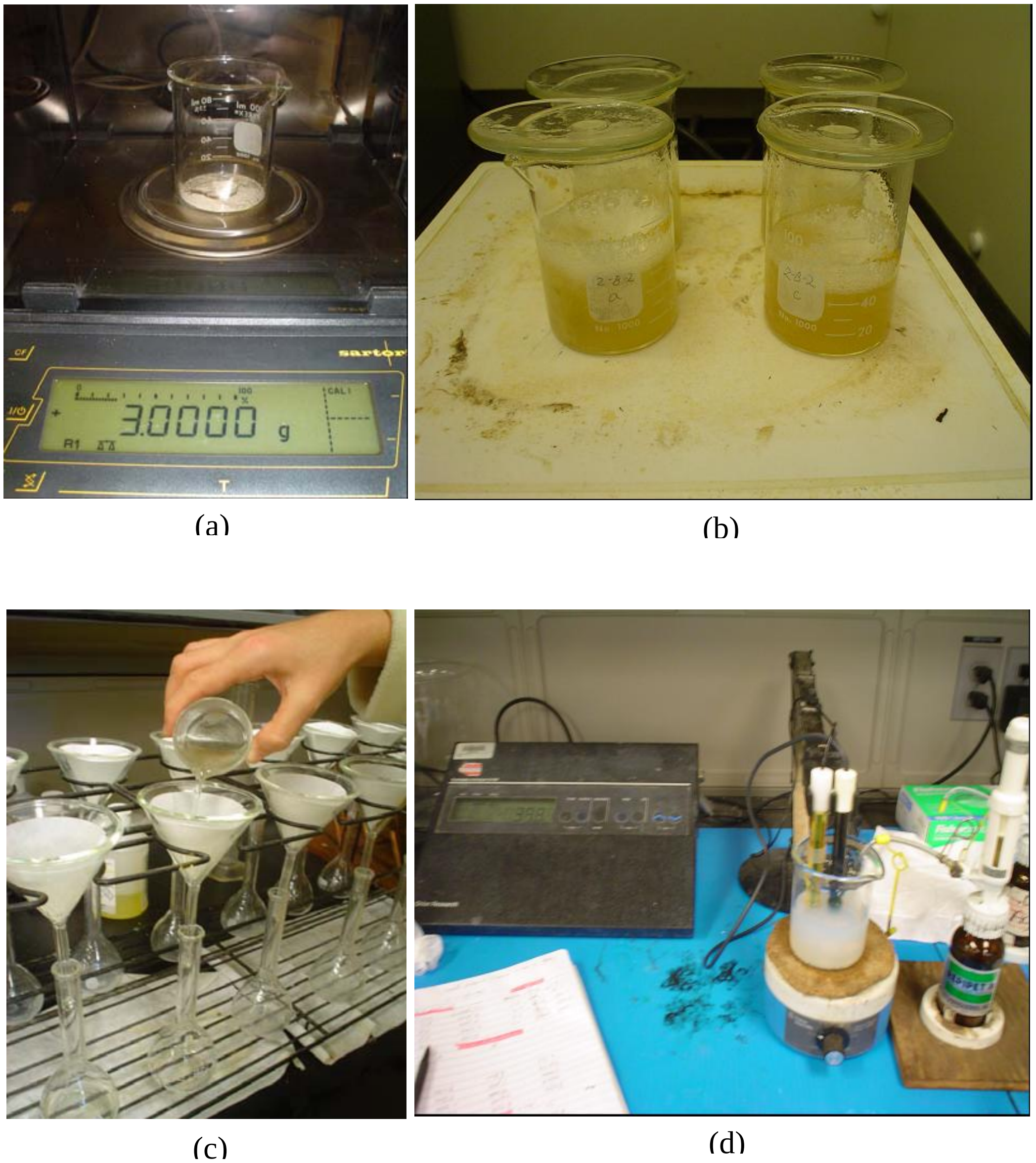
The 3 in. deep concrete cores were cut into three one inch thick pieces using a chop saw and stored in labeled plastic bags. All forty-eight samples were then separately crushed and totally dried for three hours in an oven at 230°F. After drying, all samples were pulverized until the material could pass a 50 sieve (Fig. 10.4).

(a) (b)

Figure 10.4 (a) Samples cut into 1 in. thickness; (b) Crushed and Pulverized Samples

Test Procedures

The procedures specified in the Florida Method FM5-516 [10.1] were followed to obtain the total chloride content. Three tests were performed on each sample. Fig. 10.5 shows the various steps involved in the procedure. The results are summarized in Table 10.2.



(a) (b)

(c) (d)

Figure 10.5 (a) Measuring 3 g Sample; (b) Heating Sample Solution After Adding Nitric Acid Solution; (c) Filtering Heated Sample Solution; (d) Monitoring Potential While Adding $AgNO_3$

Test Results

As shown in Table 10.2, the total chloride varied between 4.43 – 31.3 lb/cy at the highest level (location A – Fig. 10.3) and between 12.82 – 40.86 lb/cy at the lowest level (location D – Fig. 10.3). At all the locations, the chloride threshold limit (1 lb/cy) was exceeded.

Generally, the chloride content was higher close to the sea water level and close to the concrete surface excepting in pile P1. The chloride content in pile P1 was higher in the deeper concrete (1 - 2 in. depth) than near the surface (0 – 1 in. depth). The peculiar result for pile P1 could be attributed to chloride intrusion through the cracks formed on the three surfaces.

Table 10.2 Result of Chloride Content Analysis in Gandy Bridge

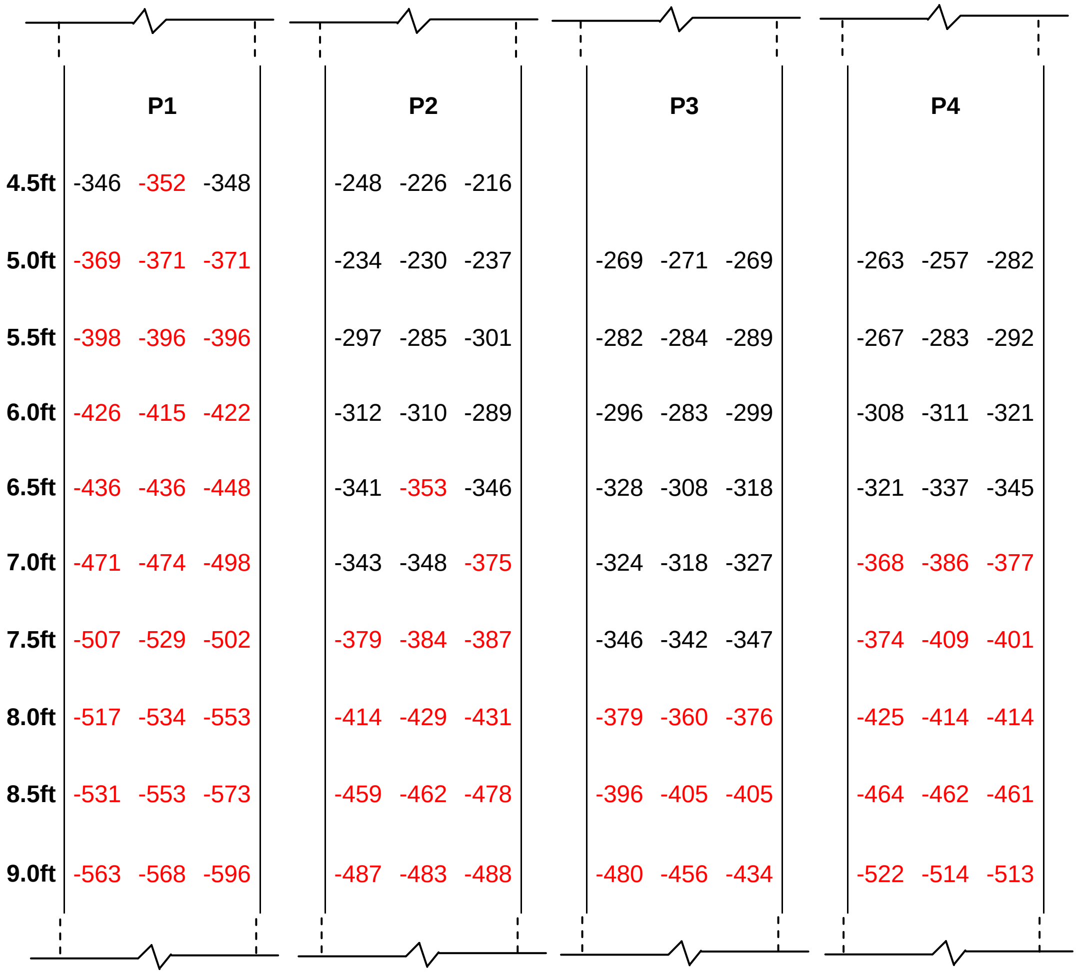
Pile Name	Location*	0 - 1 inch (lb/cy)	1 - 2 inch (lb/cy)	2 - 3 inch (lb/cy)
P1	A	12.34	**31.30**	18.81
	B	17.11	**18.81**	15.41
	C	22.25	21.72	N/A
	D	23.62	**25.48**	24.24
P2	A	12.40	9.03	4.48
	B	14.78	9.12	5.87
	C	15.65	13.02	7.52
	D	**40.86**	26.98	16.66
P3	A	15.40	16.72	14.24
	B	16.71	16.29	12.03
	C	17.85	15.62	10.52
	D	**33.02**	18.89	13.37
P4	A	15.06	9.17	4.43
	B	17.93	12.02	7.48
	C	18.39	13.97	7.97
	D	**29.65**	20.27	12.82

Location*
A: 30 in. above the high water level
B: 24in. above the high water level
C: 21in. above the high water level
D: 12 in. below the high water level

10.3.2 Surface Potential Measurements

Half-cell potential distributions on the concrete surface were measured to evaluate the initial corrosion state of the embedded prestressed steel using a copper-copper sulfate reference electrode. Assuming all strands were electrically connected in concrete, one strand was exposed by coring and connected to the positive terminal of voltmeter whose negative terminal was connected to the reference electrode.

Fig. 10.6 shows the distribution of initial half-cell potential values measured on the east faces of all four piles. Measurement was performed from 4.5 - 9 ft below the pile cap with a 6 in. space. Most of the potential readings in corrosion damaged pile P1 were more negative than -350 mV indicating there was a 90% probability for corrosion (ASTM C-91 [10.2]).



	P1			P2			P3			P4		
4.5ft	-346	-352	-348	-248	-226	-216						
5.0ft	-369	-371	-371	-234	-230	-237	-269	-271	-269	-263	-257	-282
5.5ft	-398	-396	-396	-297	-285	-301	-282	-284	-289	-267	-283	-292
6.0ft	-426	-415	-422	-312	-310	-289	-296	-283	-299	-308	-311	-321
6.5ft	-436	-436	-448	-341	-353	-346	-328	-308	-318	-321	-337	-345
7.0ft	-471	-474	-498	-343	-348	-375	-324	-318	-327	-368	-386	-377
7.5ft	-507	-529	-502	-379	-384	-387	-346	-342	-347	-374	-409	-401
8.0ft	-517	-534	-553	-414	-429	-431	-379	-360	-376	-425	-414	-414
8.5ft	-531	-553	-573	-459	-462	-478	-396	-405	-405	-464	-462	-461
9.0ft	-563	-568	-596	-487	-483	-488	-480	-456	-434	-522	-514	-513

Figure 10.6 Initial Half Cell Potential on the East Face of Piles (mV vs CSE)

10.4 Instrumentation

To monitor the progression of corrosion in the test piles, rebar probes and commercial probes were installed in each of the four piles. Current flow due to the macro cell formed by corrosion of steel was measured using rebar probes. Linear polarization test was performed using commercial probes to measure the corrosion rate. Four rebar probes were installed on the west side of the pile and two commercial probes were embedded on the east side at specified heights. Two rebar probes and one commercial probe were positioned above the wrap and the other two rebar probes and a commercial probe were placed below the wrap (Fig. 10.3).

10.4.1 Rebar Probes

Rebar probes (Fig. 10.7) were developed by the Florida Department of Transportation who made them available for this study. They are composed of a 2 in. length of a #4 rebar with a copper wire connected to one end. The wire-rebar connection is sealed with epoxy and only 1 in. length of the rebar was exposed.

Figure 10.7 Rebar Probe Developed by FDOT

Since steels in the bridge pile are exposed to different environments according to their elevation, their corrosion propagation are likely to be different. For example, steel in the lower part of pile (near the water level) may be more corroded than in the upper area (near the pile cap) since conditions for corrosion such as water, oxygen and chloride are more favorable. Electrons released in the anodic region (lower part of steel in this case) are consumed in the cathodic area on the steel surface to preserve electrical neutrality, Fig 10.8 (a). Since probes are positioned close to the existing steel in the pile, it should be similarly impacted. Therefore, by monitoring the direction and magnitude of current flow between two rebar probes installed at two different levels in the bridge pile, the shift of corrosion activity in the bridge pile may be determined.

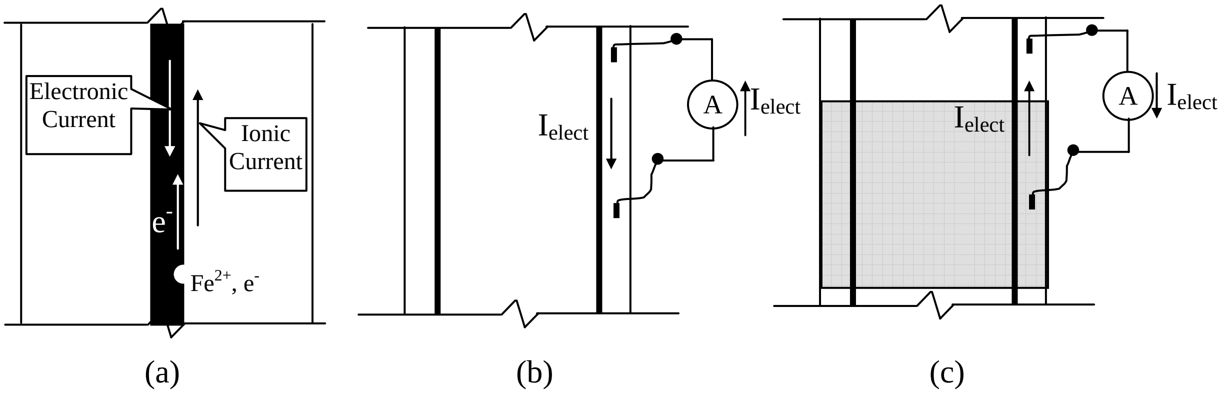


Figure 10.8 (a) Current Flow in Corroded Steel; (b) Current Flow Between Two Rebar Probes in Normal Pile; (c) Current Flow Between Two Rebar Probes in Wrapped Pile

When two probes are electronically connected in the pile, electronic current flow may look like Fig. 10.8 (b). After wrapping, however, the magnitude of current will be decreased or the direction of current flow reversed since corrosion action will be decreased in the wrapped area due to reduced supply of moisture, oxygen and chlorides, Fig. 10.8 (c). Monitoring this change would allow assessment of the performance of the wrap in controlling macro-cell corrosion.

10.4.2 Concorr Probes

To monitor the corrosion rate of steel, commercial probes manufactured by Concorr, Inc. were installed in the piles. As shown in Fig. 10.9, the probes are a 2.4 in. × 2.4 in. × 5 in. mortar block with two cables at one end. One cable is a ground wire for connection to a working electrode (steel) and the other is the data cable with a six-pin connector for connecting a PR monitor. Reference electrode and counter electrode are embedded in the mortar connected to the data cable.

Two commercial probes were installed in each pile. One probe (CP-B) was positioned at 1 ft below the high water level and the other (CP-T) was installed at 3 ft above the high water level (Fig. 10.3).

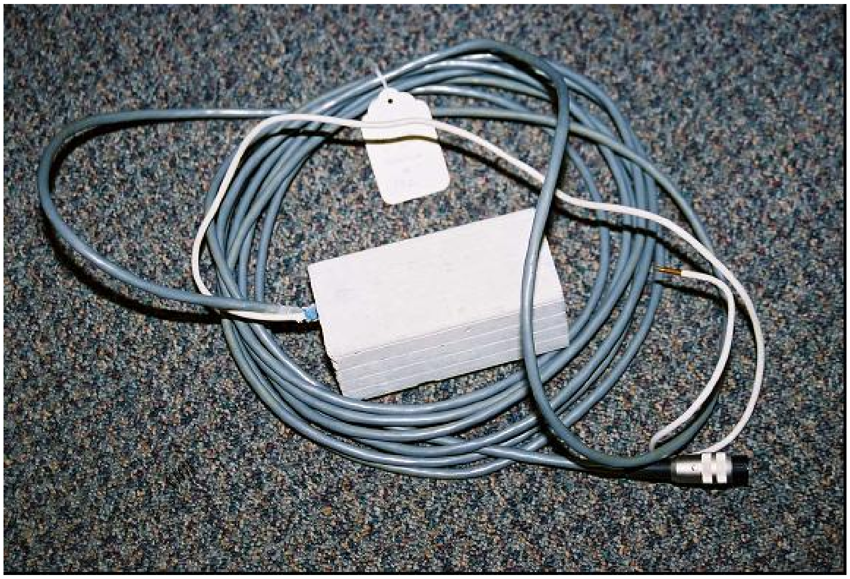


Figure 10.9 Commercial Probe Manufactured by Concorr, Inc.

10.4.3 Instrumentation Procedures

Coring

Four 2 in. diameter holes with a 3 in. depth were cored at four locations using a hollow core drill on the west face of the pile for the installation of the rebar probes. The cored concrete samples were carefully stored and used later for determining the chloride profile in the pile at those depths (Fig. 10.10).

Figure 10.10 Coring Holes for Rebar Probe Installation

To install commercial probes, two 3 in. x 6 in. opening with a 3 in. depth were made at two locations by drilling six 2 in. diameter holes on the east face of the pile (Fig. 10.11). Additionally, another two holes were cored 18 in. away from the commercial probes to make a grounding connection between the probe and steel.

Figure 10.11 Coring for Commercial Probe Installation

Installing Probes

The surface of all the rebar probes were sand blasted right before their installation to remove dirt on the surface and to increase corrosion activation. A mortar paste was filled to about a third of the hole and was pressed firmly to install the rebar probe. The probe was then positioned parallel to the main steel (in the longitudinal direction) and pressed firmly against the mortar paste placed earlier. The remainder of the core hole was then filled with the mortar paste to restore the original concrete surface (Fig. 10.11).

The chloride content in the mortar was pre-designed and mix water determined in the laboratory to make its chloride content similar to that of initial surface concrete of piles. The results of the chloride content analysis shown in Table 10.2 were used for this purpose. The chloride content was taken to be the average of the three measured values over each inch of cover, e.g. for P1 at location A it was (12.34+31.3+18.81)/3=20.8 lb. Assuming the cement content to be 600 lb/cy, the percentage chloride ratios to the weight of cement in every core hole were calculated and are shown in Table 10.3. The water/cement ratio of mortar used for mixing was 0.25.

Figure 10.12 Rebar Probe Installation

Table 10.3 Chloride Content in Mortar Used for Probe Installation
(Percent chloride to weight of cement)

	A	B	C	D
P1	3.47	2.85	3.67	4.08
P2	1.44	1.65	2.01	4.69
P3	2.58	2.50	2.44	3.63
P4	1.59	2.08	2.24	3.49

For the installation of the commercial probes, regular mortar (sand, cement and freshwater) with a 0.25 of water/cement ratio was used. The installation procedure followed the manufacture’s instructions (Fig. 10.13). A four-inch length of 316 stainless steel bar were brazed on to the strand exposed in the core hole for ground connection. Grounding wire from the commercial probe was attached to the stainless steel rod with a stainless steel clamp and then the junction was coated with epoxy to prevent corrosion (Fig. 10.14). Finally, the hole was filled with silicone and smoothed with mortar.

Figure 10.13 Commercial Probe Installation

Figure 10.14 Connecting Grounding Cable to Steel

Junction Boxes

Junction boxes were installed below the pile cap on the west face of the four instrumented piles to protect the wiring from corroding and to allow the data measurements to be performed easily. Four wires coming out from the rebar probes were connected to stainless steel rods fixed in the junction box that was bonded to the concrete surface. And two data cables coming from commercial probes were brought in (Fig. 10.15) to this box. All exposed wiring and cables were inserted in groves cut on the surface by an electric saw and sealed with hydraulic cement (leak stopper) and epoxy.

Figure 10.15 Installing Junction Box and Wiring

10.5 Wrap Details

10.5.1 Wrap Design

To design the number of wrapping layers required for restoring capacity loss due to corrosion, a parametric study was conducted using both proposed wrap repair systems. Because of the lack of information on the properties of the piles selected in the study, several assumptions were made. The ultimate strength and elastic modulus of the prestressed strand were assumed to be 270 ksi and 27,500 ksi respectively. And its yield strength was taken as 85% of its ultimate strength. Additionally, it was assumed that the strands were initially tensioned to 75% of its ultimate strength and prestress losses were 25%. The same procedures used for designing the wrap for the Allen Creek Bridge were followed (Appendix C).

Fig. 10.16 shows the interaction diagram for the 20 in. × 20 in. prestressed piles for a steel loss of 0% and 20% assuming the concrete compressive strength as 4 ksi. The graph shows that Aquawrap® Repair System developed by Air Logistics, Co. using one layer of uni-directional and two layers of bi-directional carbon wrap was sufficient to restore the original load capacity. And a similar result was assessed with Tyfo® Wrap

System manufactured by Fyfe Co. LLC using 2 layers of axial and 4 layers of transverse glass wrap.

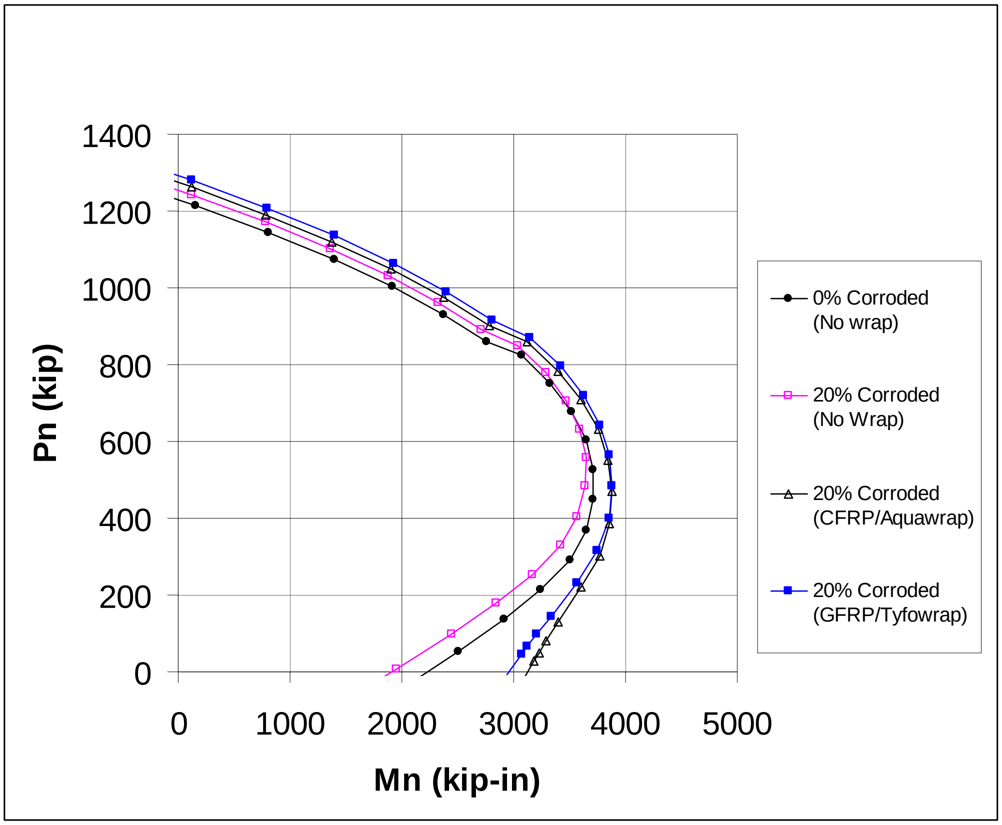


Figure 10.16 Interaction Diagram of 20in. x 20 in. Prestressed Pile

10.5.2 Preparatory Work

Since the selected piles were located in deep waters, a sturdy and simple scaffolding system was required to perform the repair work safely. A scaffold was built using ¾ in. #9 expanded steel mesh and 2 in. x 2 in. x ¼ in. steel angles. The scaffolding system was in two parts and designed to be fitted around a pile. The two parts were assembled in the field and scaffold was suspended over the pile cap using steel chains (Fig. 10.17).

10.5.3 Patching Lost Concrete Section

Due to corrosion of steel, there was delamination and spalling of the surface concrete in pile P1. Prior to the application of FRP wrap, the lost concrete section was reformed using Tyfo® PUWECC manufactured by FYFE Co. LLC. Tyfo® PUWECC is a cement-based patching material designed to be worked in water. After the exposed surface of the steel and concrete were cleaned by sand blasting, fresh water was applied to the surface to make them damp for achieving proper bond. Tyfo® PUWECC paste mixed with fresh water was poured into a wooden mold attached on the targeted corner by a clamp (Fig. 10.18). All procedures followed manufacture's instructions. Material properties are presented in Chapter 3.

Figure 10.17 Installing Scaffolding Around a Pile

Figure 10.18 Patching Damaged Pile (P1)

10.5.4 Surface Preparation

The marine growth on the surface of all the piles was removed with a scraper and the surface cleaned with a sand blaster and a grinder operated by air pressure. Projecting parts of concrete surface were chipped using a hammer and chisel, and depressions were filled with hydraulic cement. All four corners were chamfered and were ground to a ¾ in.

radius using a grinder. Just prior to wrapping, all surfaces were pressure washed using fresh water to remove all dust, debris, and remaining marine growth (Fig. 10.19)

Figure 10.19 Surface Preparation of Piles

10.5.5 Aquawrap® Application (AirLogistics Co.)

Two piles (P1 and P2) were wrapped using Aquawrap® Repair System developed by Air Logistics, Co. Both piles were wrapped using one layer of unidirectional carbon fiber and two layers of bi-directional carbon fibers. The procedures for wrapping the piles using Aquawrap® Repair System were as follows (Fig. 10.20).

i. Mix the base primer composed of a grey colored part A and a clear brown colored part B.
ii. Apply the primer to the prepared pile surface by hand.
iii. Attach eight 10 in. × 72 in. unidirectional carbon fiber strips longitudinally on the four faces.
iv. Place the bi-directional carbon fiber spirally applied over the unidirectional carbon layer in two continuous layers without overlap.
v. Place one layer of a 10 in. wide glass fiber veil over the bi-directional carbon fiber with a 2 in. overlap to consolidate the wrap and provide a better finish.
vi. Place an orange-colored plastic stretch film over the veil with tiny openings to allow gases that develop during curing to escape.
vii. Remove the stretch film after curing completely and paint the same base primer over the veil for UV protection

Excepting for step (vi), this was the same procedure used for installing the FRP wrap on the Allen Creek Bridge.

(a) (b) (c)

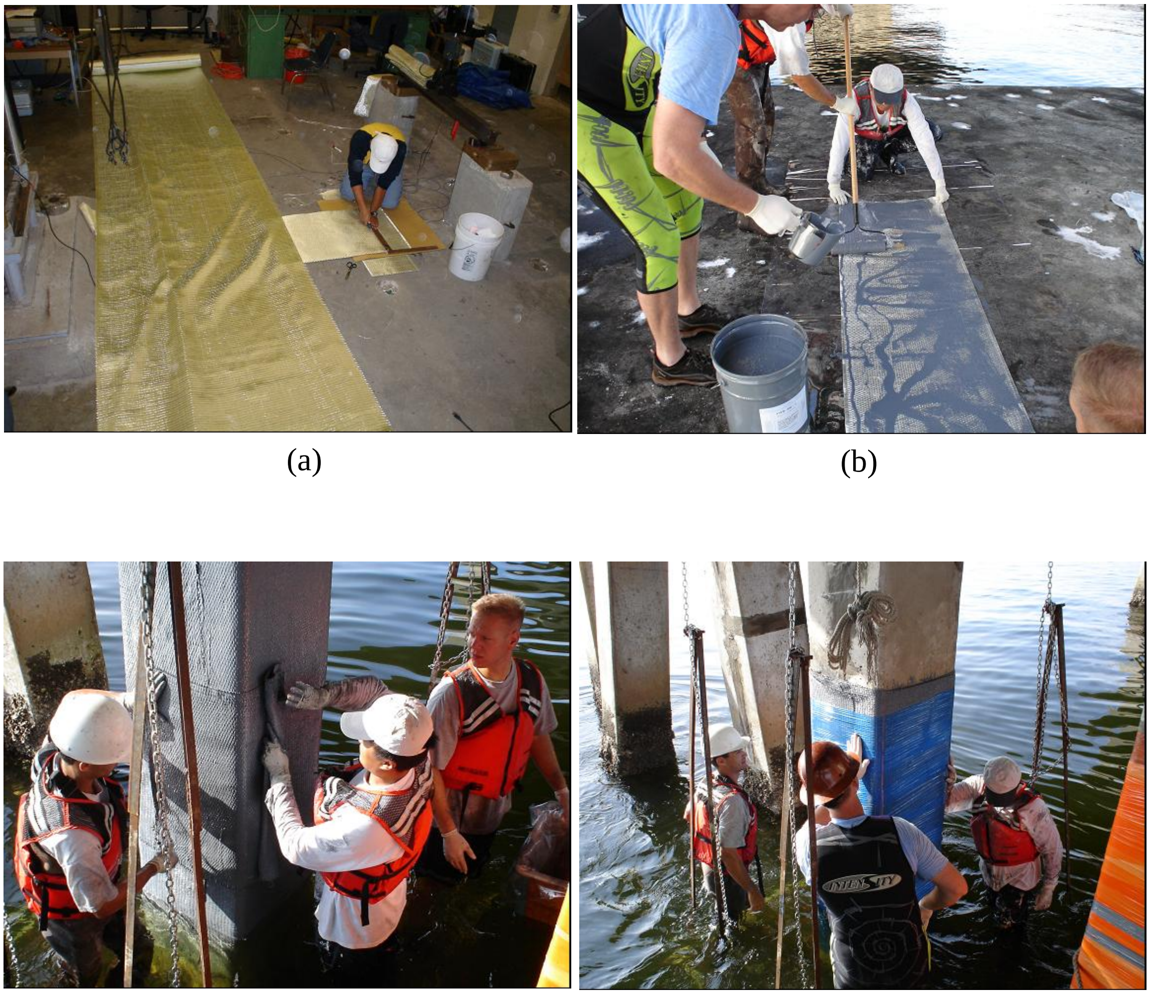
(d) (e) (f)

Figure 10.20 (a) Applying Primer; (b) Longitudinal Wrap; (c) Transverse Wrap; (d) Applying Glass Fiber Veil; (e) Applying Shrinkage Plastic; (f) UV Protection Coating

10.5.6 Tyfo® Wrap Application (Fyfe Co. LLC)

Only one pile (P3) was wrapped using the Tyfo® Wrap System manufactured by Fyfe Co. LLC. It comprised a SEH-51A fiberglass fabric, SEH-51AR fiberglass fabric and Tyfo® SW-1 epoxy. Both SEH-51A fiberglass and SEH-51AR fiberglass are uni-directional glass fiber having exactly same material properties excepting that their weave directions have a difference of 90°. Tyfo® SW-1 is a two component epoxy developed for underwater use. Details on their material properties are summarized in Chapter 3. In this study, two layers of SEH-51AR fabric were applied for axial capacity and four layers of SEH-51A were applied for transverse capacity. Unlike Air Logistics' System that was a 'pre-preg', the fibers had to be impregnated with resin on site. The procedures for wrapping the piles using Tyfo® Wrap System were as follows (Fig. 10.21).

i. Tyfo® SEH-51A fabric was cut into twelve 24 in. × 90 in. pieces for transverse capacity.
ii. Tyfo® SEH-51AR was cut into four 24 in. × 90 in. pieces and two 36 in. × 90 in. pieces for axial capacity.
iii. Tyfo® SW-1 was mixed using a low speed drill for 5 minutes.
iv. Half of the Tyfo® SEH-51A fabric cut in step (i) and half of Tyfo® SEH-51AR fabric cut in step (ii) were saturated with Tyfo® SW-1 epoxy. Two quarts of epoxy was used for saturating one piece of fabric and each piece was made into a roll afterwards for easy transport and application.
v. Three 24 in. × 90 in. Tyfo® SEH-51A fabric pieces were applied laterally without overlap for transverse capacity.
vi. Another three 24 in. × 90 in. Tyfo® SEH-51A fabric pieces were applied laterally without overlap for transverse capacity.
vii. Two 24 in. × 90 in. pieces and one 36 in. × 90 in. piece Tyfo® SEH-51AR fabric were applied laterally with 6 in. overlap for axial capacity.
viii. Repeat steps from iii to vii.
ix. Place a plastic film over the wrap to protect wrap during curing.


(a) (b)

(c) (d)

Figure 10.21 (a) Cutting GFRP; (b) Saturating GFRP; (c) Wrapping; (d) Applying Shrinkage Plastic

10.6 Result of Corrosion Monitoring

Fig. 10.22 shows the view of wrapped piles and unwrapped control. Four sets of corrosion measurements were taken before application of the FRP wrap to assess the initial corrosion state of the piles. Corrosion monitoring included the measurement of the current flow between the rebar probes using an ammeter and a linear polarization test using the embedded commercial probes and a PR monitor. After wrapping, five additional sets of data were taken.

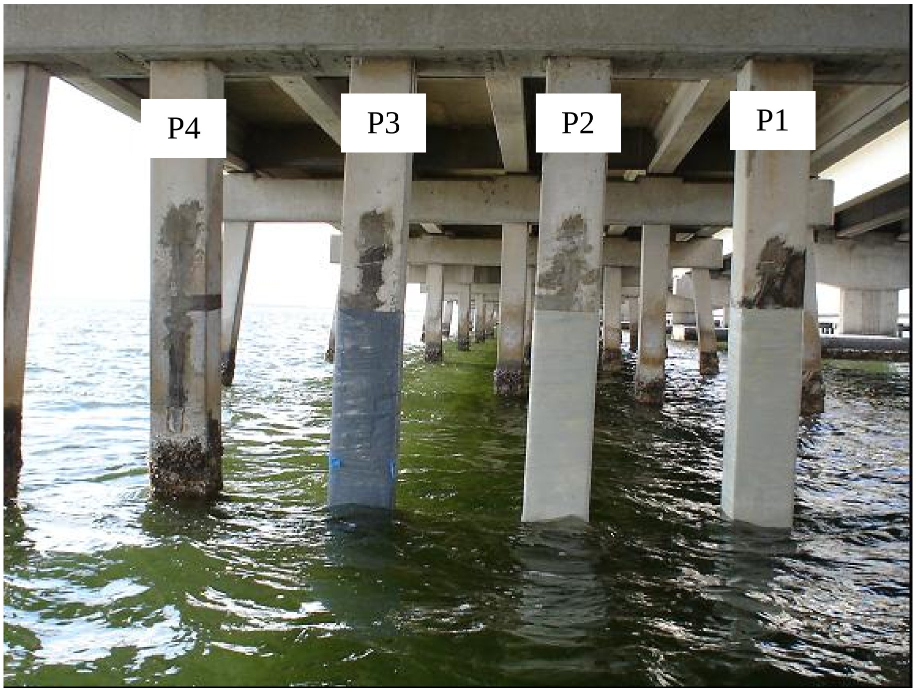


Figure 10.22 View of Unwrapped Control and Wrapped Piles

Figure 10.23 Current Flow Measurement (L) and Linear Polarization Test (R)

10.6.1 Current Measurement

As explained in Section 10.4, the magnitude and direction of the current flowing between the two probes embedded at different elevation may provide information on the change in corrosion in the pile. Fig. 10.24 shows the variation of current flow between rebar probeA (RP-A) and rebar probeD (RP-D) in all four piles. The RP-A is located in the unwrapped area and RP-D is embedded in the wrapped concrete (Fig. 10.3). RP-A was connected to the negative terminal of the ammeter and RP-D was connected to the positive terminal.

Since the lower region in the pile might be more corroded than the upper region at the initial stage, the current was expected to flow from RP-D to RP-A showing a positive value on the ammeter. Fig. 10.3 shows that initially the current flow in Pile1 and Pile2 was positive meaning that RP-D was more active in corrosion than RP-A. After wrapping, however, the magnitude of current flow decreased and finally the direction of current flow reversed as expected. On the other hand, there was little change in the direction of current flow in Pile3 and Pile4.

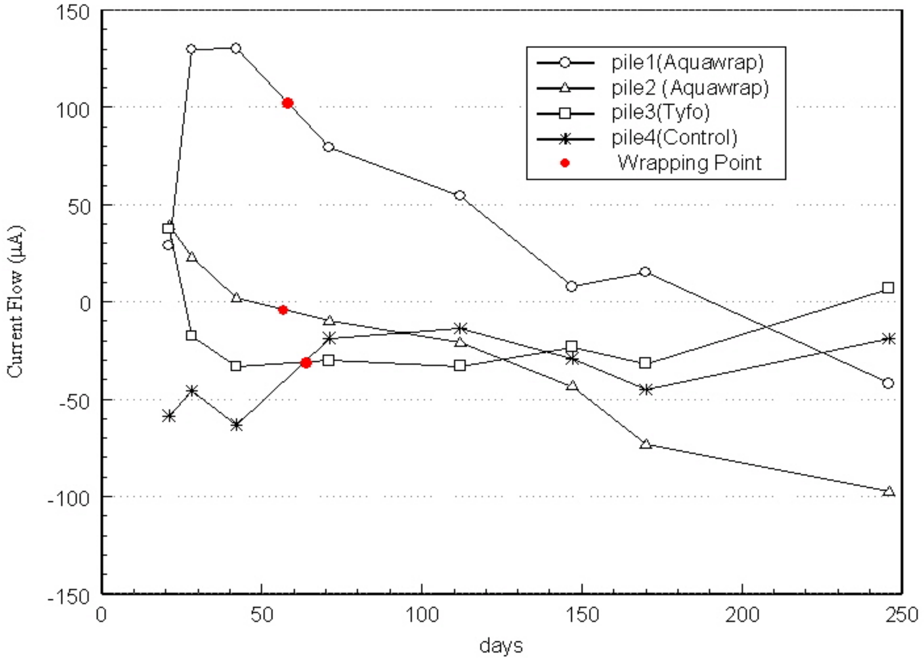


Figure 10.24 Current Flow Measurement Between PR-A and PR-D

10.6.2 Linear Polarization Test

Linear polarization tests were performed using the commercial probes installed at the top (CP-T) and bottom (CP-B) of the piles. CP-T was embedded 3 ft above the high water level and CP-T was located 1 ft below the high water level.

The result of the corrosion rate measurements using CP-T is shown in Fig. 10.25. As expected, the variation in the corrosion rate in the top part of the piles was very small. Since seawater could not reach this area, corrosion might not be active. The rate was highest in Pile1 that had been severely damaged.

Fig. 10.26 shows the variation of the corrosion rate in the tidal zone of the piles. The corrosion rate in the bottom part was much higher than in the top part for every pile, especially the previously damaged pile. After wrapping, however, the values showed a tendency to be stabilized in Pile2 and Pile3 while it was still unstable in Pile1. There has been no difference in corrosion rate between the wrapped and unwrapped piles.

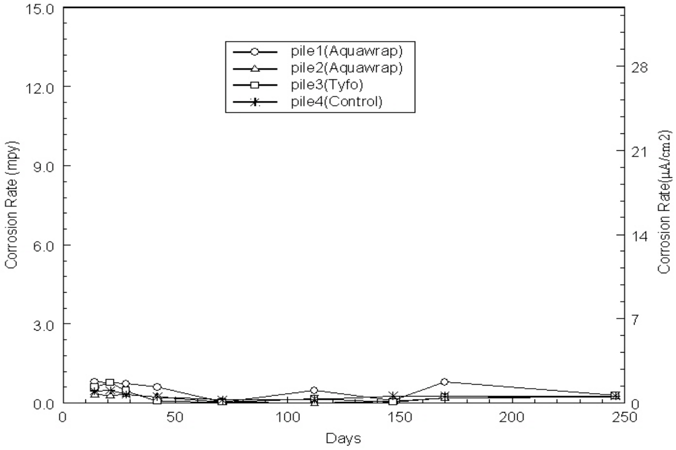


Figure 10.25 Variation of Corrosion Rate at the Top of the Piles

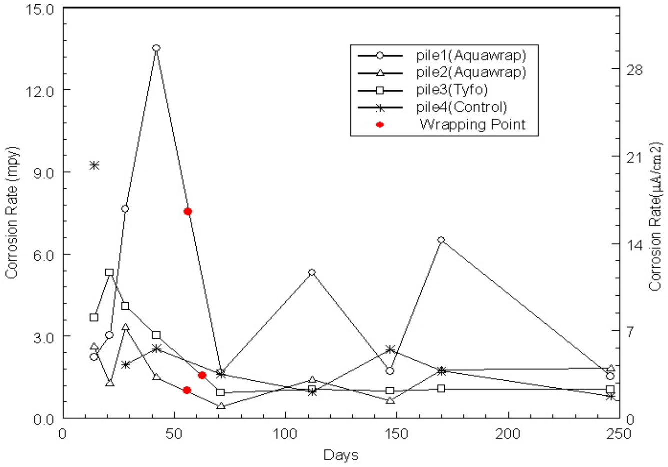


Figure 10.26 Variation of Corrosion Rate at the Bottom of the Piles

10.7 Bond Test

A total of twelve tests were carried out to evaluate the FRP/concrete bond in May 2005 nearly 6 months after the application of the FRP wrap. Tests were conducted on two piles (Pile2 - Aquawrap and Pile3 - Tyfo), two faces (north and south) and at three different elevations as shown in Fig. 10.27. Three 1.75 in. diameter holes were scored on the two FRP surfaces using a diamond drill bit. Since the test area was exposed to tide changes, a fast curing epoxy (Power-Fast+) manufactured by Powers Fasteners, Inc. was used to bond the dollies to the FRP. It took 15 minutes to dry completely and took 24 hours to cure to provide a maximum bond strength of 3000 psi. The test was performed using an Elcometer 106 adhesion tester 7 days after the installation of the dollies.

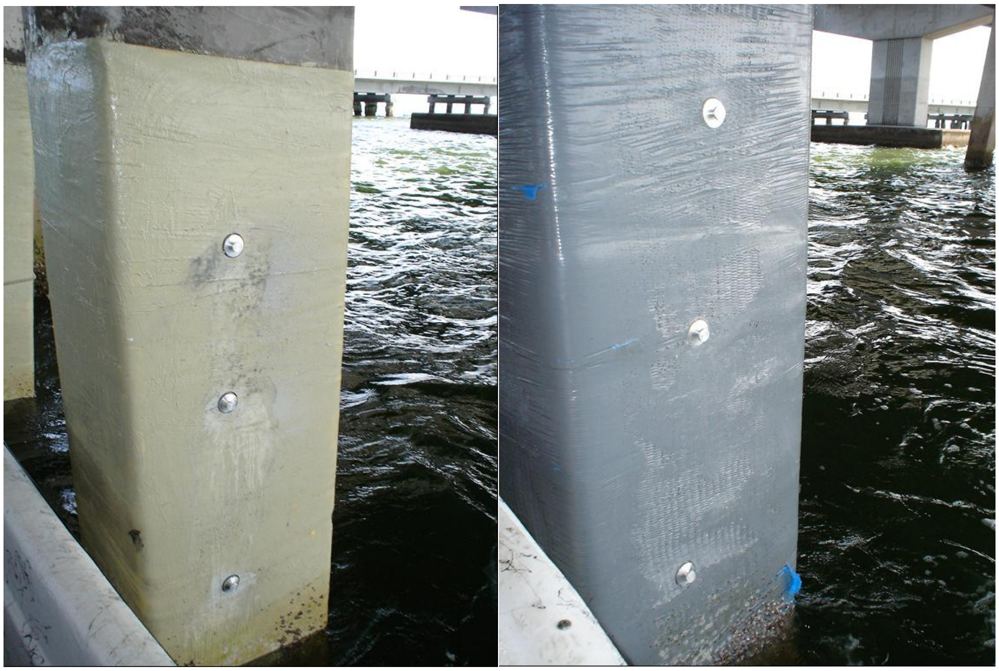


Figure 10.27 Installed Dollies on Pile2 (L) and Pile3(R)

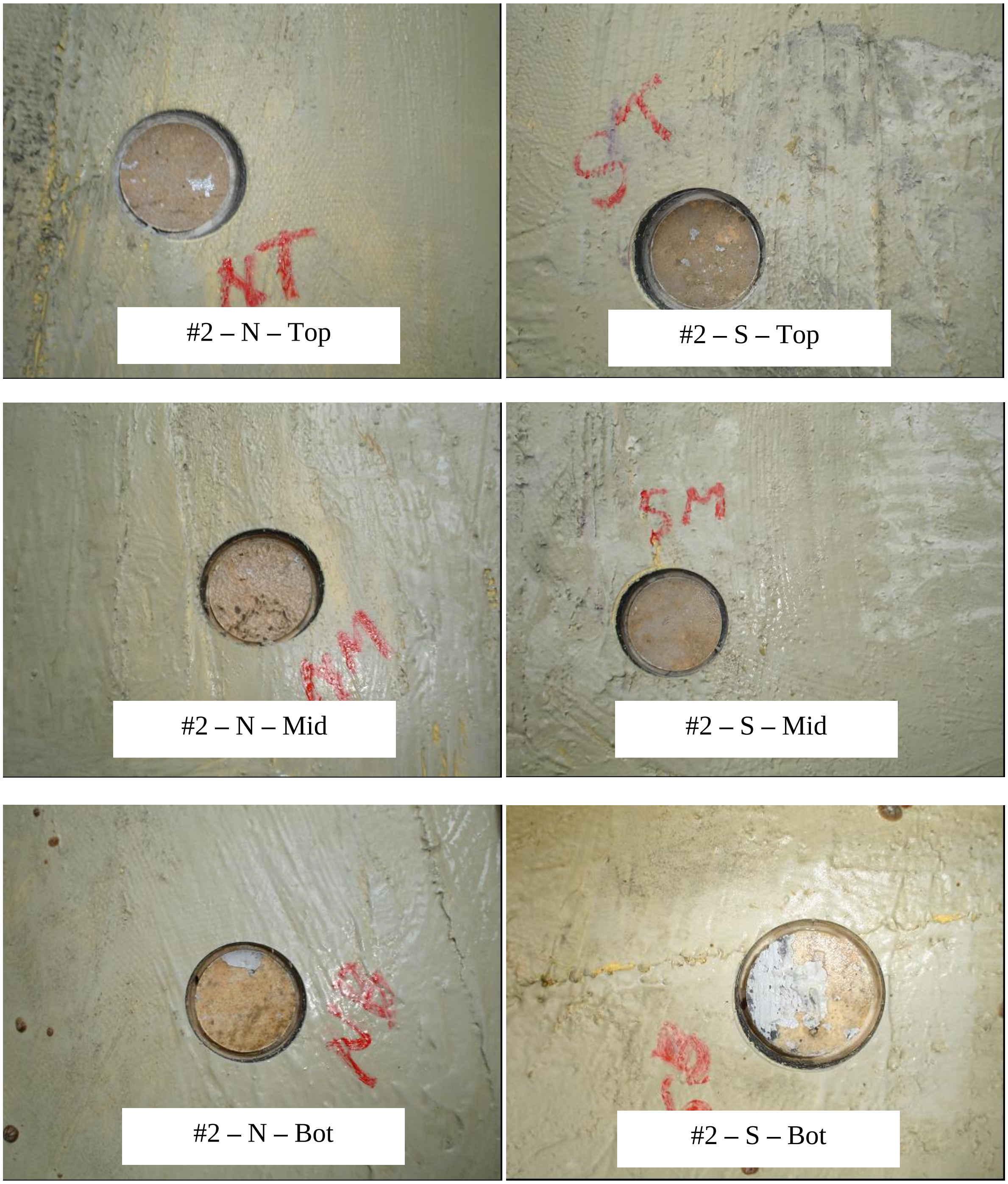
The results of the tests are summarized in Table 10.4. As with the bond tests conducted on the wrapped piles in the Allen Creek Bridge reported in the previous chapter, none of the tests led to failure in the concrete. However, there were no similar layer failures. Instead, all the failures were epoxy failures in which the dolly separated from the concrete at its interface (Fig. 10.28 – 29).

The ultimate bond stress values were higher for the Aquawrap system compared to that in Allen Creek Bridge (Table 9.4). There was also less scatter. The epoxy used in the Fyfe system (Tyfo fiberglass) was much better and gave significantly higher strength values particularly at the middle and bottom. Surprisingly it was very low at the top where there it appears that there insufficient epoxy.

Table 10.4 Bond Strength between FRP and Concrete (unit: psi)

Name	**Repair**	**Face**	**Top**	**Middle**	**Bottom**
Pile2	Aquawrap (Carbon)	North	116.0 (epoxy)	29.0 (epoxy)	87.0 (epoxy)
		South	145.0 (epoxy)	72.5 (epoxy)	58.0 (epoxy)
		Average	**130.5**	**50.7**	**72.5**
Pile3	Tyfo (Glass)	North	0.0 (epoxy)	87.0 (epoxy)	87.0 (epoxy)
		South	72.5 (epoxy)	289.9 (epoxy)	203.0 (epoxy)
		Average	**36.2**	**188.5**	**145.0**

Note: Epoxy failure refers to separation of the FRP from the concrete indicating poor bond



#2 – N – Top

#2 – S – Top

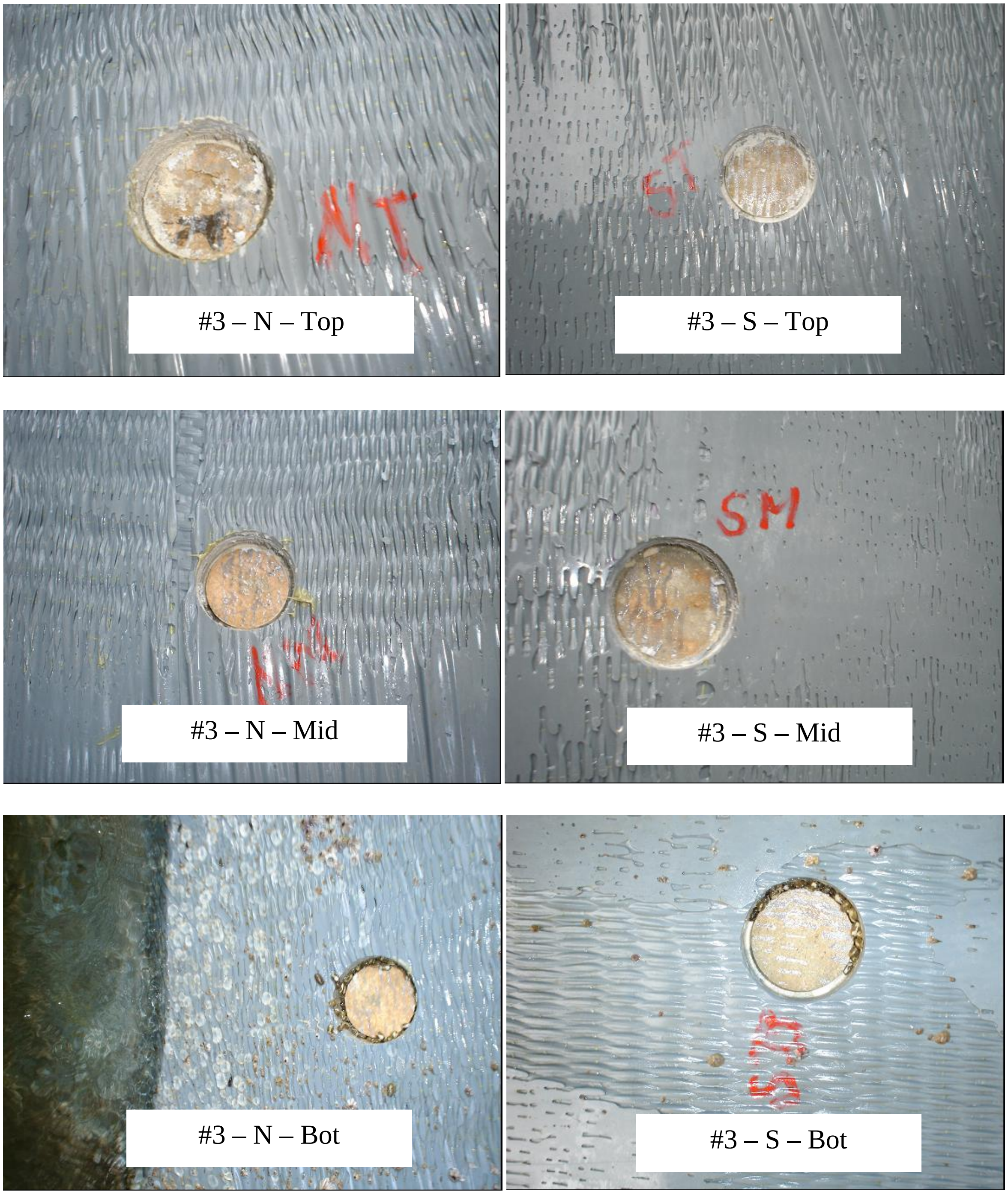
#2 – N – Mid

#2 – S – Mid

#2 – N – Bot

#2 – S – Bot

Figure 10.28 Bond Test on Pile2 (all epoxy failure)



#3 – N – Top

#3 – S – Top

#3 – N – Mid

#3 – S – Mid

#3 – N – Bot

#3 – S – Bot

Figure 10.29 Bond Test on Pile3 (all epoxy failure)

Table 10.4 shows that the bond strength varied from 29 psi to 145 psi for Aquawrap and from 0 to 290 psi for Tyfo wrap. In the Aquawrap system, the minimum strength was found at the middle of the north face and the maximum at the top of the south face. In the Tyfo system, the maximum strength was at the middle of the south face and the minimum was top of the north face. Interestingly, the minimum strength was not in the tidal zone (bottom) of the pile in either system. This indicated that the epoxies performed better under wet application. Based on Table 10.4, the performance of the Tyfo system was better than the Aquawrap repair system.

Fig. 10.30 shows the average bond values while Fig. 10.31 shows the maximum values. Inspection of Fig. 10.30 shows that the average bond stresses from the Aquawrap system are a fraction of that for the Tyfo wrap. This difference is somewhat smaller when the maximum values are compared as in Fig. 10.31.

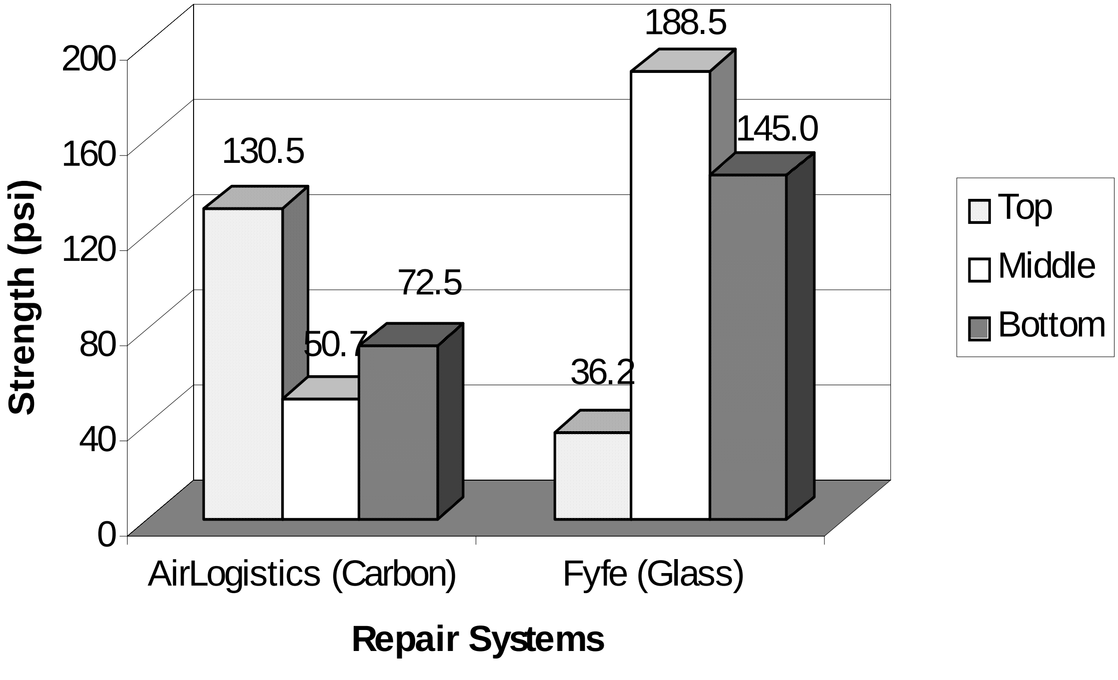


Figure 10.30 Averaged FRP-Concrete Bond Strength

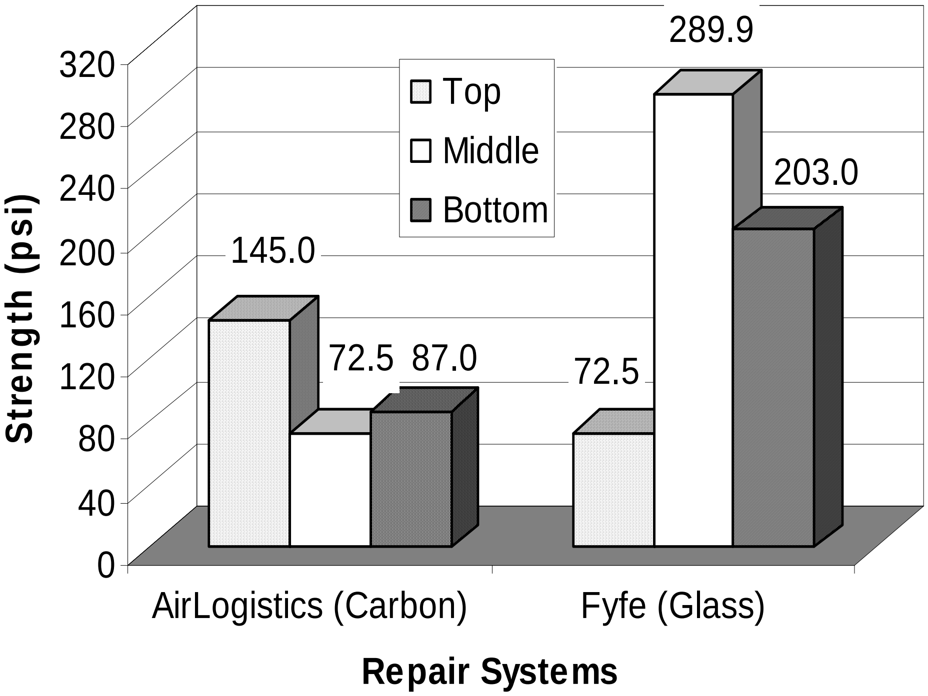


Figure 10.31 Maximum FRP-Concrete Bond Strength

10.8 Conclusions

The following conclusions may be drawn:

1. A new, modular, portable scaffolding system that could be assembled around the pile and suspended from the pile cap permitted FRP wrapping in the deeper waters. This scaffolding system (Fig. 10.17) worked well and was moved from pile to pile after the wrap was completed. With the scaffolding in place, it took about 40 minutes to wrap a pile after surface preparation.

2. Of the two wet-wrap systems, the pre-preg system developed by Air Logistics was easier to use since fibers are pre-impregnated. The Fyfe system requires on-site impregnation that can pose logistic problems. In this case, nearby access to above water foundations of the adjacent Gandy Bridge made it possible to carry out the wrap (Fig. 10.20 –21). Otherwise, it could have been a problem.

3. The bond strength of the Fyfe system was higher than the Aquawrap system particularly especially in the wet region. All bond failures were in the epoxy at the FRP/concrete surface (Fig. 10.28 – 29). Poor results for the Fyfe system at the top pile were most probably due to lack of sufficient epoxy. Despite the epoxy bond failure, the measured bond stress at two locations at the bottom using the Fyfe

system were 289 psi and 203 psi (Table 10.4) more than double that for the Aquawrap system (Fig. 10.30-31).

4. The two instrumentation systems appear to be working well. The rebar probe showed a change in the direction of current flow for the most severely corroded pile (Fig 10.24) but not in the other two. The linear polarization measurements were taken (Fig. 10.25-10.26) but it is too early to draw conclusions on the effect of FRP wrapping on corrosion of steel.

References

10.1 FDOT, (2000). Florida Method of Test for Determining Low-Levels of Chloride in Concrete and Raw Materials, Designation:FM 5-516.

10.2 ASTM C876-91 (1999). “Standard Test Methods for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete,” American Society for Testing and Materials, Philadelphia, PA.

11. SUMMARY AND CONCLUSIONS

11.1 Summary

Concrete piles exposed to tidal waters in southeastern United States are vulnerable to corrosion damage. Although pile jackets incorporating a sacrificial cathodic protection system can provide durable repairs they may not always be affordable. In such circumstances, fiber reinforced polymers have the potential to offer a lower cost, acceptable alternative.

This report presents results from laboratory and field studies to evaluate the effectiveness of FRP in strengthening piles in a marine environment. Laboratory studies investigated (1) expansion due to corrosion, (2) the effect of the fiber type and number of layers on the corrosion rate, (3) the effect of exposure on the FRP-concrete bond, (4) the role of surface preparation on FRP performance, and (5) strengthening effectiveness of underwater wrap using a newly developed water-activated resin. The demonstration studies evaluated two disparate FRP systems – two wet wrap and a dry wrap system using coffer dam construction (undertaken by SDR Engineering and described in Appendix C). Piles were instrumented to allow determination of the corrosion rate and on-site pullout tests conducted to evaluate the FRP-concrete bond.

11.2 Review of Findings – Laboratory Investigations

Important findings from the laboratory investigations are presented in this section. Additional conclusions on these studies may be found at the end of Chapters 5-8.

11.2.1 Expansion Study

Maximum measured expansion ranged from 0.85% for 15.66% steel loss to 2.1% for 25.72% (Table 5.9). These values provide an upper bound on corrosion expansion in FRP wrapped piles since (1) the expansion was unrestrained and (2) a "dry" accelerated corrosion scheme was used that did not permit corrosion products to be washed away.

11.2.2 Effectiveness of FRP - Ambient Exposure

FRP significantly slows down the corrosion rate. In tests conducted outdoors, identical unwrapped and wrapped (28 days after fabrication) specimens were subjected to simulated tidal cycles (the tide was changed every 6 hours to allow 2 cycles a day) for nearly 3 years. Strands and ties were retrieved from these specimens and metal loss determined by measuring the weight loss after cleaning. The average weight loss in the strands was 6.6% in unwrapped controls vs 3.3% in the CFRP and 3.4% in the GFRP wrapped specimens. For ties, the corresponding weight losses were 10.1% vs 6.6% (CFRP) and 6.3% (GFRP) (Table 6.8). The effectiveness of the FRP was not linked to the number of FRP layers. Best results were obtained when only two CFRP or GFRP layers were used (Fig. 6.40-6.41).

11.2.3 Effectiveness of FRP – Accelerated Exposure

In the ambient exposure study, the performance of newly cast specimens with cast-in chlorides was compared. The accelerated exposure study evaluated the practical case where piles had corroded prior to wrapping. Specimens were first corroded to a targeted 25% metal loss and then wrapped using CFRP. The exposure for the wrapped and unwrapped specimens was identical to that for the ambient case (Fig. 6.12), i.e. simulated tidal cycles excepting that the temperature of the salt water was 140F (60C) and the tank was covered (Fig. 7.16). Metal loss was evaluated identically through gravimetric testing by retrieving the strands and ties, cleaning and accurately measuring their weight.

The maximum incremental metal loss in the unwrapped controls was found to be 64.2% vs from 1.1% to 12.1% in CFRP wrapped specimens (Table 7.13, Fig. 7.50) after exposure to the same environment for nearly two years. The study also found that performance was not necessarily enhanced if more than two CFRP layers were used or if the wrap extended over the entire length of the corroding specimen.

11.2.4 FRP/Concrete Bond

A total of 48 pull out tests (24 each for the CFRP and GFRP specimens) were conducted to evaluate the FRP/concrete bond on selected specimens exposed for nearly 3 years to simulated tidal cycles in the ambient exposure study (Section 11.2.2). The bond was evaluated at the top (always dry), middle (wet or dry) and bottom (always wet). The average measured bond stresses rounded *down* from the values in Tables 6.2-6.3 are summarized in Table 11.1.

Table 11.1 Average Bond Stress Comparison (psi)

Description	**Item**	**Top** (dry)	**Middle** (wet or dry)	**Bottom** (submerged)
CFRP	Highest	347	347	289
	Lowest	145	145	203
	Average	284	264	268
GFRP	Highest	434	333	405
	Lowest	217	174	145
	Average	295	260	300

The highest values reflect concrete failure and the lowest values reflect epoxy failure (see Fig. 6.28-6.29). The bond was lowest when a single layer was used (Fig. 6.30-6.31). In terms of the largest number of concrete failures, the epoxy used for the CFRP wrap gave the best results.

11.2.5 Strengthening Using Water Activated Resin

Columns specimens corroded to 25% targeted steel loss were wrapped underwater with minimal surface preparation. Immediate ultimate eccentric load tests were conducted to assess the effectiveness of the underwater strengthening. Additionally, wrapped specimens corroded to 25% were further corroded to a targeted 50% metal loss and tested identically under eccentric loading. The tests showed that underwater FRP strengthening was effective and the section gained strength that exceeded its original capacity (Table 8.13). However, limited bond pullout tests indicated that the bond between FRP and the concrete was poor (Table 8.12).

11.2.6 Effect of Surface Preparation

The accelerated exposure (Section 11.2.3) was also used to investigate the role of surface preparation on the effectiveness of the FRP wrap from both strength and gravimetric testing. Variables investigated included (1) full vs minimal repair (2) partial vs complete wrap and (3) sealing vs non-sealing the concrete surface.

Eccentric column tests were conducted on 2-layer CFRP strengthened specimens that had been pre-corroded to a targeted 25% metal loss and then exposed to the accelerated environment for nearly 2 years. The results of the tests showed that surface preparation had little effect on the restored strength. The performance of specimens that had full or epoxy repair (Fig. 7.9-10) was very similar (Table 7.6). This was also reflected in the gravimetric testing where measured metal losses in the strands and ties were, in general, comparable (Table 7.13).

Thus, the benefits of conducting full repair in which all deteriorated concrete was removed, the steel cleaned, corrosion inhibitors applied and section re-formed was marginal. Epoxy repair that simply sealed the cracks gave similar results.

11.3 Field Demonstration Studies

11.3.1 Design

A new iterative method of designing the FRP wrap has been proposed in which strength and expansion requirements are uncoupled. Strength is provided based on interaction relations and the expansion strain that can be accommodated by this strength is assessed using a confinement model. The procedure is fairly simple and gives results that are less conservative than available alternative methods. Complete details are given in SDR’s report in Appendix C. This procedure was used for designing the wrap in both the field studies.

11.3.2 Allen Creek Bridge

A full-scale demonstration study was conducted in which eight 14 in. x 14 in. prestressed piles supporting the Allen Creek Bridge, Clearwater (#150036) were wrapped using FRP. Six of the piles including two unwrapped controls were instrumented to allow measurement of the post-wrap corrosion rate.

Two disparate wrap systems were tested: (1) an underwater system in which the FRP could be bonded to the wet concrete surface directly using a new water-activated urethane Aquawrap resin system, and (2) a dry wrap system requiring cofferdam construction. The water-activated resin system used both carbon and fiberglass whereas the cofferdam system only used carbon. The underwater system is a pre-preg and is very efficient in terms of time taken to complete the wrapping. The cofferdam system is a wet-lay up in which the fibers are saturated with resin on site. It takes more time. Details may be found in Chapter 9 and Appendix C.

Post-wrap corrosion measurements taken over more than 2 years indicate that the measured corrosion rate in the wrapped piles was lower than that of the unwrapped controls (Fig. 9.17-18). The performance of the CFRP wraps for both the dry and wet wrap systems was comparable but that for the GFRP from the wet wrap was somewhat poorer (Fig. 9.19). The instrumentation system used held up well throughout.

Bond Tests

A total of 16 bond tests (8 each on the dry and wet wrap systems) were conducted 26 months after the piles had been wrapped. The bond was evaluated at the top (always dry) and at the bottom (wet or dry). The average measured bond stresses rounded *down* from the values in Table 9.4 are summarized in Table 11.2.

Table 11.2 Allen Creek Bridge Bond Stress Comparison (psi)

System	**Material**	**Item**	**Top** (dry)	**Bottom** (wet or dry)
Dry Wrap Mas2000	Carbon 2-layer	Highest	188	174
		Lowest	188	58
		Average	188	116
	Carbon 4-layer	Highest	362	145
		Lowest	304	43
		Average	333	94
Wet Wrap Aquawrap	Carbon 2-layer	Highest	145	29
		Lowest	116	0
		Average	130	14
	Glass 4-layer	Highest	72	29
		Lowest	29	29
		Average	50	29

All failures in the dry system occurred in the epoxy (Fig. 9.24). In the wet system failure additionally occurred between the FRP layers (Fig. 9.25). The ultimate bond values for the wet system were lower than that obtained from laboratory tests (Table 8.12). The values for the dry system were also surprisingly lower for the submerged region than laboratory values (268 psi in Table 11.1 vs 94 or 116 psi).

The values for the carbon using the water-activate resin were very poor yet its corrosion rate measurements were comparable to that for the dry system. This suggests that the adhesion needed for corrosion mitigation in which the FRP is wrapped completely around the element could be lower than that needed for strengthening beams or slabs where the FRP is applied to just one surface.

11.3.3 Gandy Bridge

This demonstration study (Chapter 10, #100300) evaluated two different wet wrap systems. Three piles were wrapped – two using the water-activated Aquawrap resin system used in the Allen Creek Bridge and a third pile using a wet wrap system developed by Fyfe. All three wrapped piles and an additional unwrapped control were instrumented using two different type of probes – a rebar probe developed by the State Materials Office and a commercially developed probe made by Concorr Inc. As the wrapping was completed recently, the corrosion readings are in still in the process of stabilizing (Fig. 10.24-26) and no conclusions can be drawn regarding possible reduction in the corrosion rate of wrapped piles at this time.

Bond Tests

A total of 12 bond tests (6 each on the two wet wrap systems) were conducted 6 months after the piles had been wrapped. The bond was evaluated at three levels - the top (always dry), middle and at bottom (wet or dry) (Fig. 10.27). The average measured bond stresses rounded *down* from the values in Table 10.4 are summarized in Table 11.3.

Table 11.3 Gandy Bridge Bond Stress Comparison (psi)

Wet Wrap System	**Material**	**Item**	**Top** (dry)	**Middle** (wet or dry)	**Bottom** (wet or dry)
Aquawrap	Carbon 2-layer	Highest	145	72	87
		Lowest	116	29	58
		Average	130	50	72
Fyfe	Glass 4-layer	Highest	72	289	203
		Lowest	0	87	87
		Average	36	188	145

All failures occurred in the epoxy (Fig. 10.28-29). The epoxy for the Fyfe system performed better than the Aquawrap system. At the bottom and the middle where the application was made on wet surfaces, its performance compares favorably with the dry wrap system (188 psi, 145 psi vs 116 psi and 94 psi in Table 11.2).

11.4 Conclusions

The goal of this study was to determine if FRP was effective in reducing the corrosion rate in wrapped specimens. Gravimetric testing convincingly demonstrated that FRP significantly slowed down the corrosion rate under both ambient (Table 6.8) and accelerated exposure (Table 7.13).

In the ambient exposure, new specimens were wrapped and exposed to an outdoor environment for nearly 3 years. The metal loss in strands of wrapped specimens was only 50% that of unwrapped controls (Table 6.8). In accelerated testing where specimens were pre-corroded to targeted 25% metal loss and then exposed to hot, salt water cycles for nearly 2 years, reductions were much greater. The maximum *increase* in steel loss in the strands for the unwrapped controls was 64.2% compared to 12.1% for the *worst* performing FRP wrapped specimen (Table 7.13). Strength tests conducted following exposure showed that the FRP had restored much of its original capacity (Table 7.6).

Field demonstrations showed that the technology could be readily transferred (Chapters 9 and 10). The USF team with access to limited resources and facilities were able to wrap piles in Allen Creek and in the much deeper waters of Tampa Bay using an innovative scaffolding system. The piles could be wrapped quite easily and in most cases individual piles took less than an hour to wrap.

Corrosion monitoring indicates that the corrosion rates in the wrapped piles were lower compared to controls (Fig. 9.17-9.18). Based on laboratory data it is likely there these specimens are corroding more slowly. Laboratory data showed that corrosion rates and actual measured weight loss corroborated reasonably well (Fig. 6.43).

Estimated Costs

Table 11.4 shows the estimated cost for repairing one pile with the Aquawrap system based on information provided by Air Logistics Inc. The cost of mobilization, fabricating scaffolding and labor is not considered.

Table 11.4 Cost Estimation of Air Logistics Wrap System (for one pile)

Item	**Allen Creek (14 in x 14 in , 5 ft)**		**Gandy (20 in x 20 in, 6 ft)**	
Type	**Carbon Wrap** 1-longitudinal 2-transverse	**Glass Wrap** 2-longitudinal 4-transverse	**Carbon Wrap** 1-longitudinal 2-transverse	**Glass Wrap** 2-longitudinal 4-transverse
Fiber	$859	$655	$1,469	$1,110
Adhesive	$200	$200	$250	$250
Paint	$150	$150	$150	$150
Total	$1,209	$1,005	$1,869	$1,510
Cost/linear ft	$242	$201	$312	$252

Table 11.5 is a cost estimate of the Tyfo Wrap system as provided by Fyfe for repairing one pile in the Gandy Bridge. This does not include surface preparation, mobilization, scaffolding and labor.

Table 11.5 Cost Estimation of Tyfo® Wrap System (for one pile)

Item	**Gandy (20 in x 20 in, 6 ft)**
Type	**Glass Wrap** 2-longitudinal 4-transverse
SEH 51A	0.72/SF
Adhesive	$1.73/SF
Tyfo51-SW1	$30.56/SF
Cost/linear ft	$204

Table 11.6 shows part of the item average unit cost posted on the website of the Florida Department of Transportation [11.1]. The statewide average for pile jacket system on a 20 in. pile is $725.25 per linear ft. Although the costs in Table 11.4-11.5 do not include supplementary cost such as labor and scaffolding, FRP wrapping may be competitive cost-wise with the pile jacket system.

Table 11.6 Item Average Unit Cost (2003 – 2005) [11.1]

Item	of	Average	Amount	Quantity	Meas	Obs?	Description
0457 70404	3	$704.17	$80,275.00	114.000	LF	N	PILE JACKET INTEGRAL(OTHER)(16")
0457 70405	5	$330.39	$193,280.00	585.000	LF	N	PILE JACKET INTEGRAL(OTHER)(18")
0457 70406	4	$725.25	$406,138.00	560.000	LF	N	PILE JACKET INTEGRAL(OTHER)(20")
0457 70408	2	$791.67	$11,875.00	15.000	LF	N	PILE JACKET INTEGRAL(OTHER)(22")
0457 70409	5	$1,088.32	$728,084.00	669.000	LF	N	PILE JACKET INTEGRAL(OTHER)(24")
0457 70411	1	$1,100.00	$5,500.00	5.000	LF	N	PILE JACKET INTEGRAL(OTHER)(30")

Based on the findings of this research project and cost data it appears that FRP is a viable repair method. Bond values from the field studies were low (Table 11.2-11.3) for strengthening. The ACI guide [11.1] requires the tension adhesion tests to "*exceed 200 psi and exhibit failure of the concrete substrate*" for bond critical applications, e.g. strengthening a beam or a slab where the FRP is applied to one face.

In corrosion mitigation application where the FRP encompasses the specimen completely, such limits may not necessarily apply. Laboratory tests showed that strengthening was unaffected by the poor bond (Table 8.13) while corrosion measurements showed corrosion rates for carbon from the dry and wet Aquawrap system to be comparable (Fig. 9.19). A lower value may therefore be appropriate in corrosion repair applications. In this connection it is worth noting the epoxy system developed by Fyfe gave bond values in excess of 200 psi in the Gandy Bridge. Its highest values were 289 psi and 203 psi in wet applications (Table 11.3) that exceeded those obtained from the dry wrap at the same locations (Table 11.2). This system may be suitable since it does not require coffer dam construction and can provide bond in excess of 200 psi. It will however, require on-site impregnation that makes it more difficult to use than the Aquawrap water-activated resin system.

11.5 Recommendations for Future Work

The field demonstration studies showed that FRP can be readily used in the field. Laboratory exposure showed that there was no upward migration of corrosion cells when the wrap extended beyond the chloride contaminated region. This practice should be followed in full-scale applications. Strength tests conducted in the laboratory also showed that bond was not as critical for wrapped specimens for restoring strength. Despite relatively poor bond (Table 8.12), ultimate capacity was not compromised (Table 8.13). Cost information provided by Fyfe and Air Logistics (Table 11.4, 11.5) suggest that there may be situations where FRP could provide a cost effective repair alternative. In view of recommendations are made that can further improve this method.

Bond Evaluation and Testing

The bond strength measured on site was found to be very variable. A systematic laboratory investigation needs to be conducted to establish exactly what is needed in underwater field repairs to ensure good bond. The Fyfe epoxy system seems to be the more promising since it gave the highest values under wet conditions. However, the ease with which the Aquawrap system can be applied suggests that it should also be considered in future studies. Thus, both systems should be the subject of future evaluations.

Localized Anodic Protection

The study shows that FRP cannot stop corrosion but it can slow it down. In view of this, it is possible to optimize the performance of the FRP wrap if it can be combined with a localized embedded cathodic protection system that can stop corrosion. Such systems have been successfully used for building repairs. Embedded anodes will enable regions of high corrosion to be protected and thereby extend repair life. They are easy to install and as it is a sacrificial cathodic protection system, it will not add significantly to the cost of the FRP repair. Moreover, as corrosion will be stopped, fewer FRP layers may be required thereby lowering the overall cost of the repair.

Field Trial

New field trials should be conducted using the improved procedure for ensuring good bond. Contrasting sites with piles corroding non-uniformly would be suitable for testing the localized embedded anode protection system. Piles will need to be instrumented to assess the effectiveness of the cathodic protection, preferably wirelessly as it will allow continuous monitoring. Provision should be made in these demonstration projects to allow bond testing, preferably non-destructively, after the wrap is applied so that problematic regions can be immediately fixed.

References

11.1 FDOT, http://www.dot.state.fl.us/estimates/TRNSPORT/PayItems.htm

11.2 ACI 440.2R-02 (2002). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, Farmington Hills, MI.

Appendix A

Crack Drawings for Surface Preparation Study

A.1 Crack Drawings of 5ft Specimens at 25% of Targeted Steel Loss

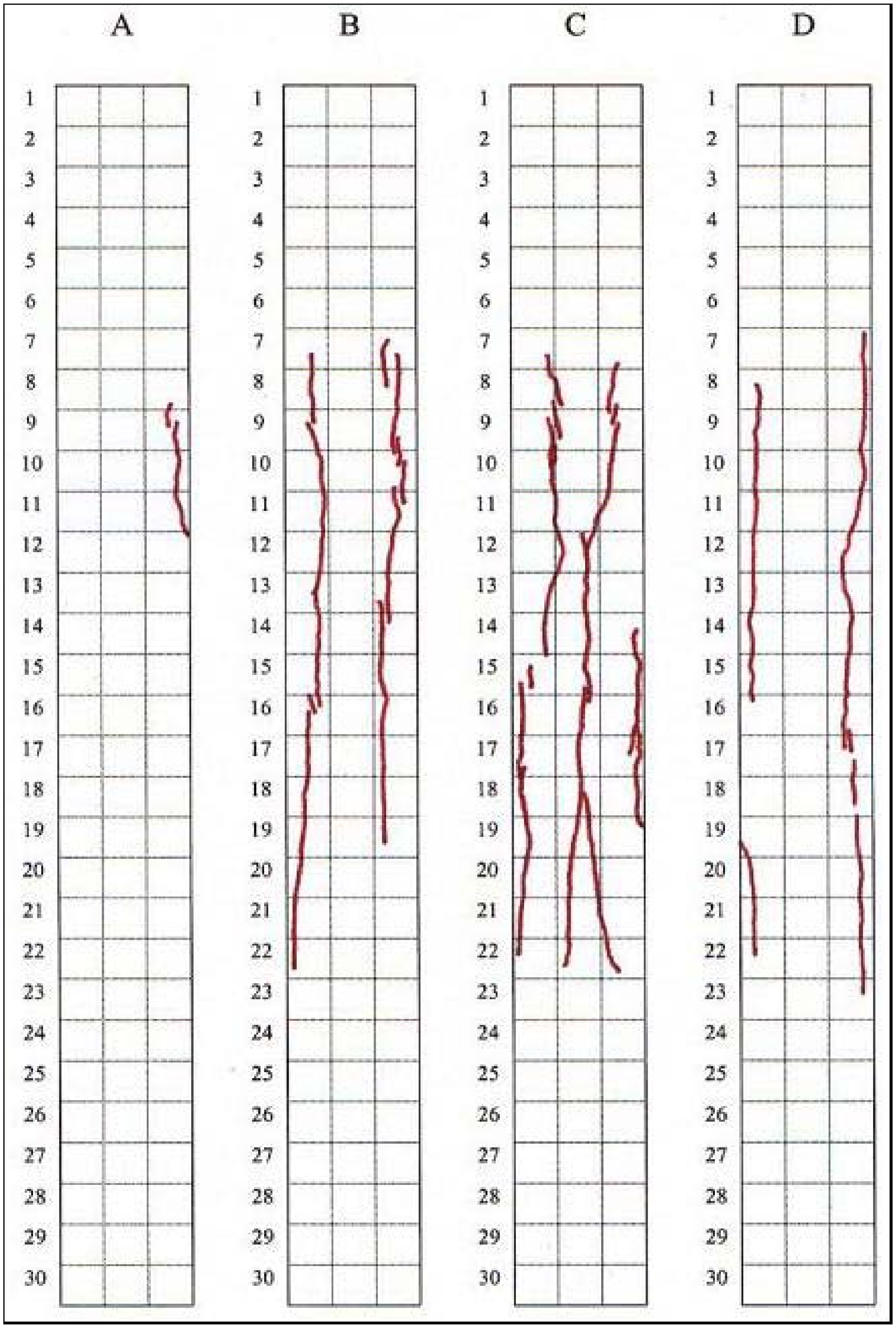


Figure A.1 Crack Mapping of #61 at 25%

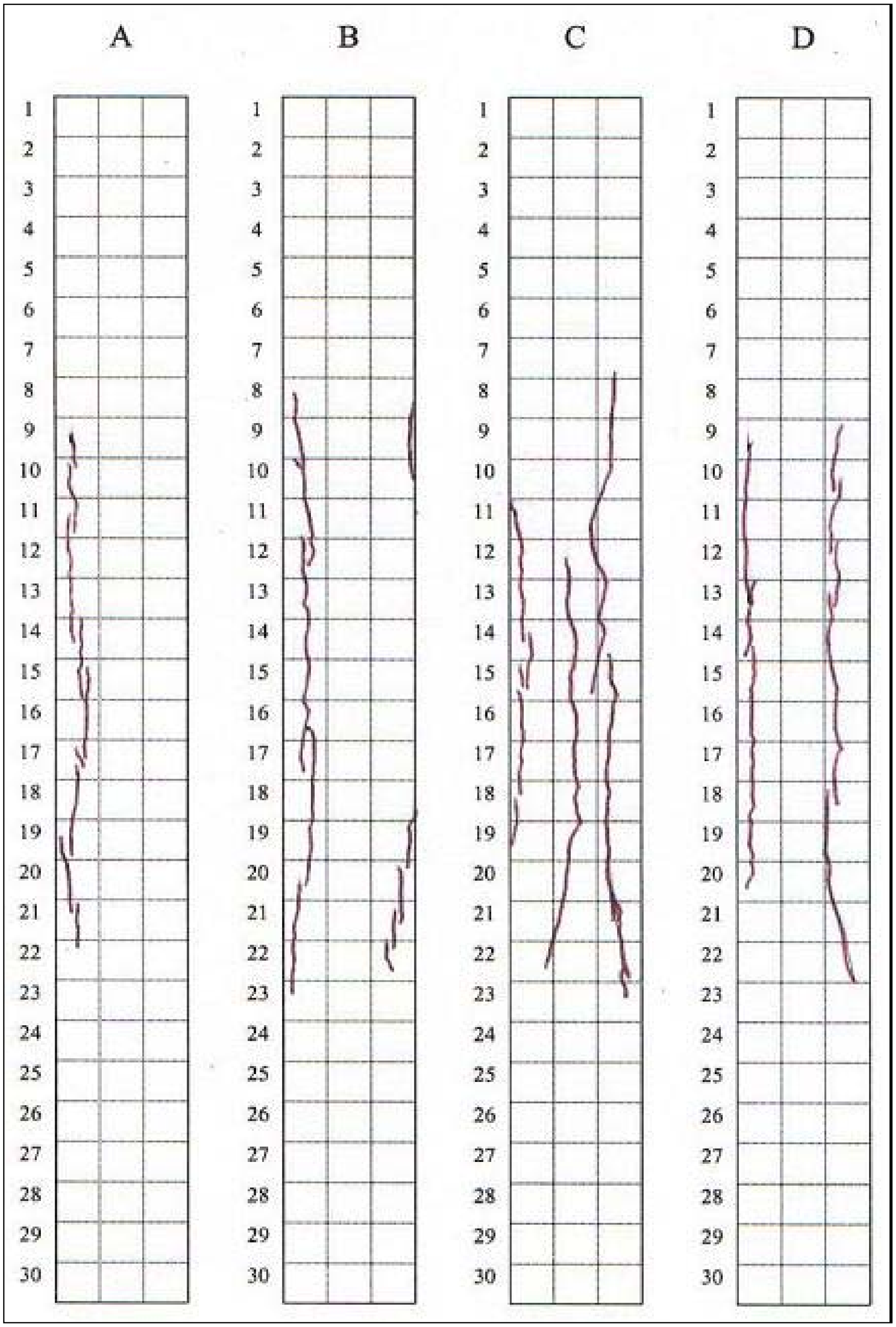


Figure A.2 Crack Mapping of #62 at 25%

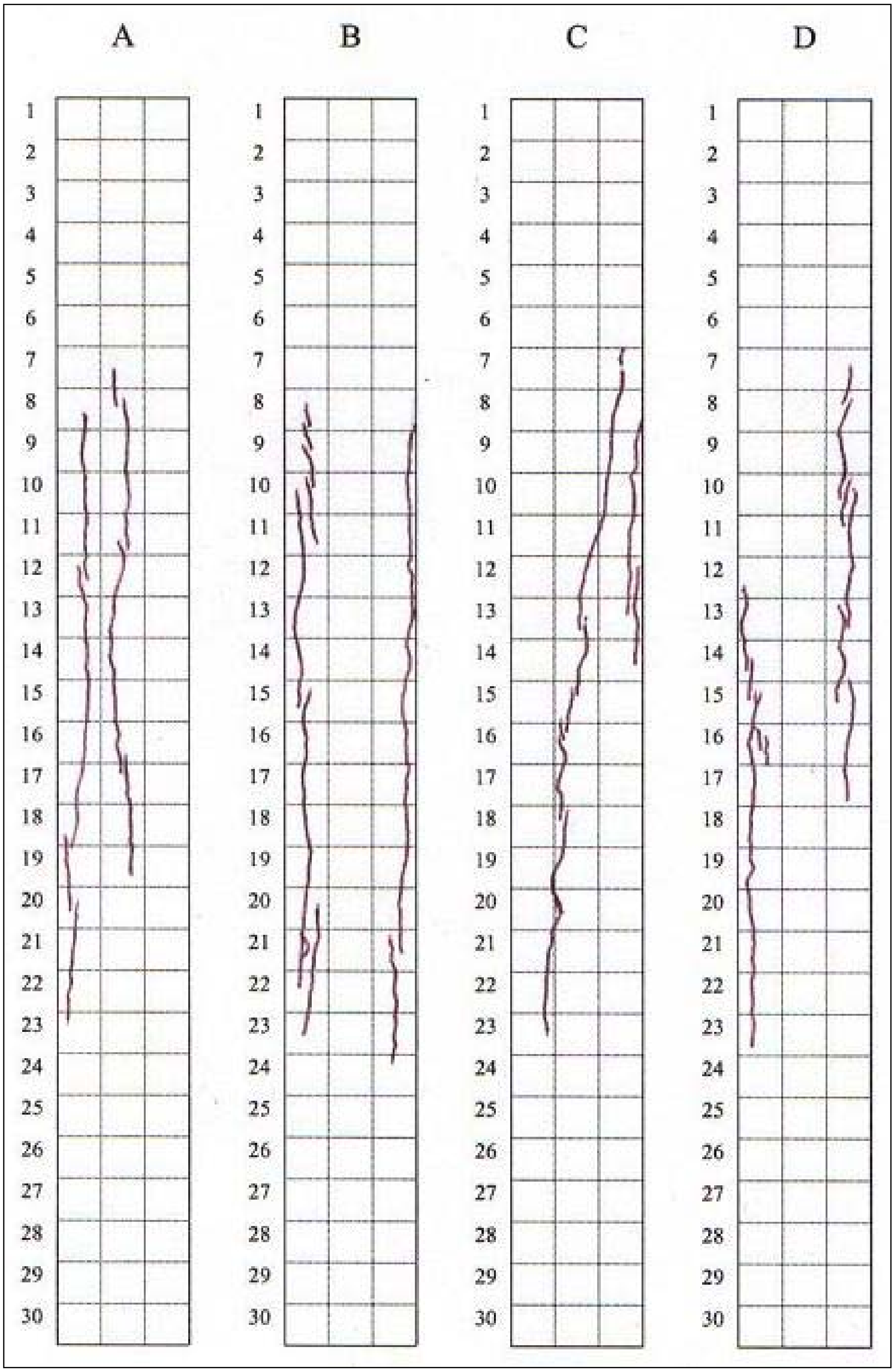


Figure A.3 Crack Mapping of #63 at 25%

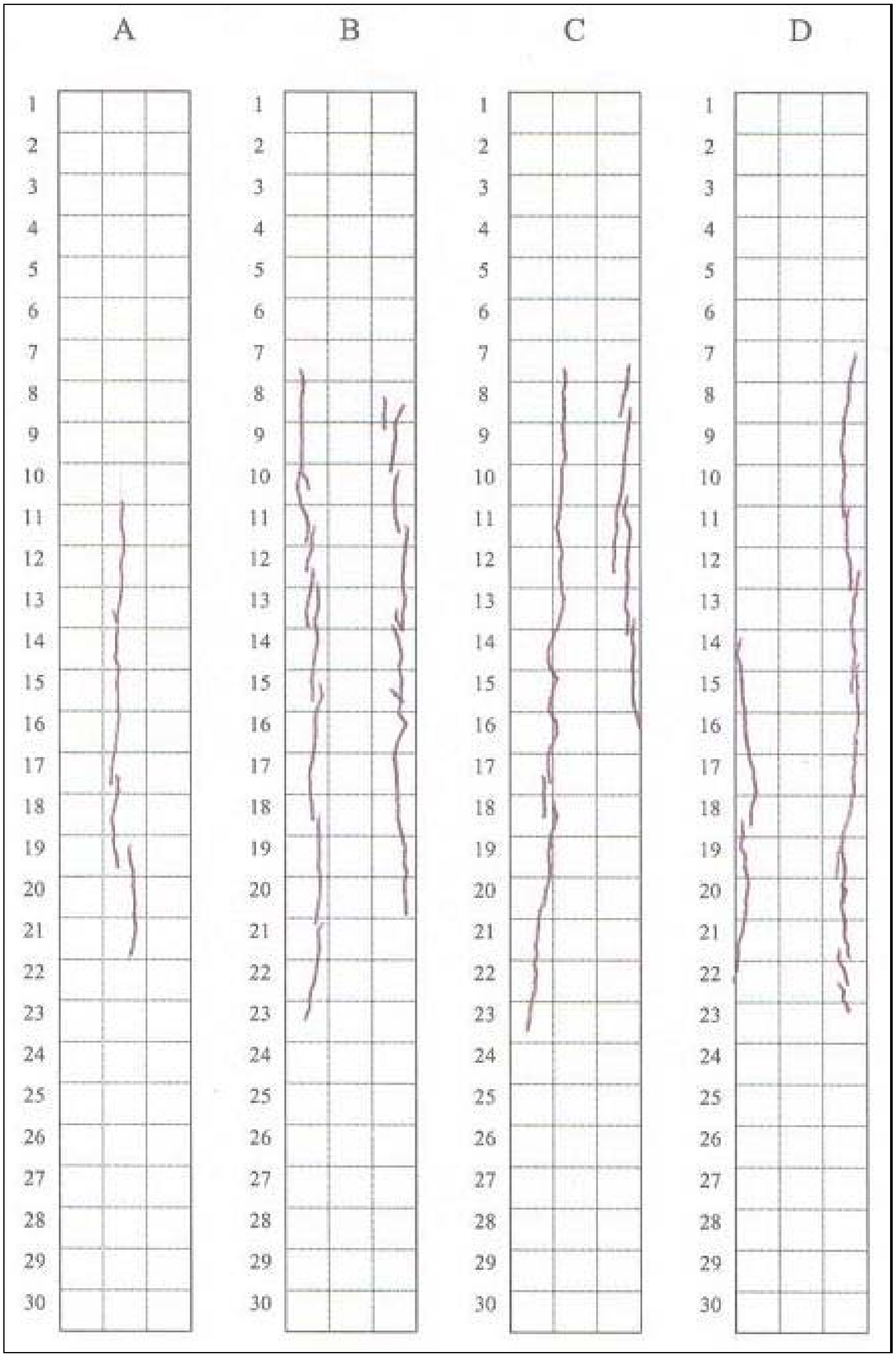


Figure A.4 Crack Mapping of #64 at 25%

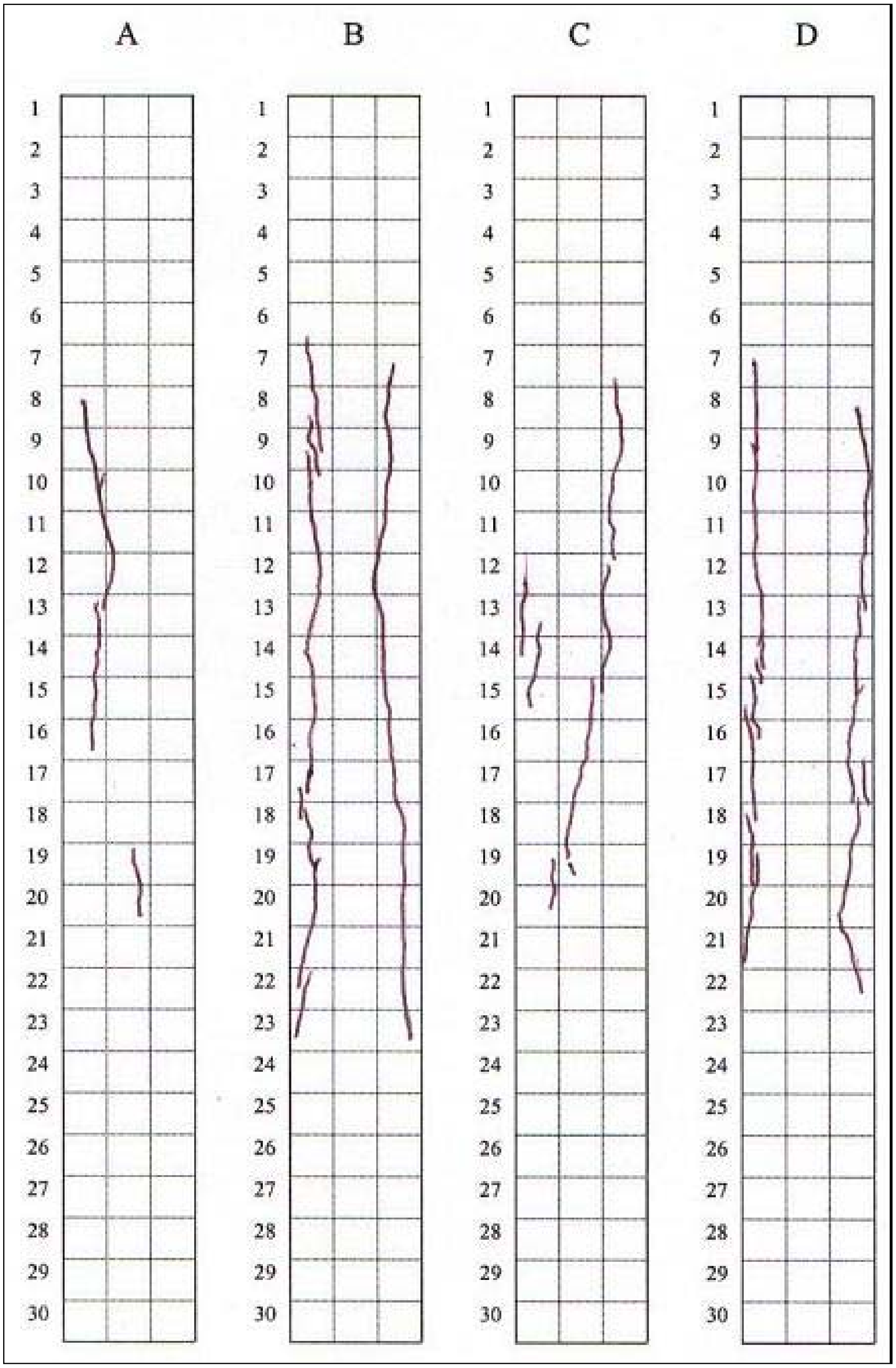


Figure A.5 Crack Mapping of #65 at 25%

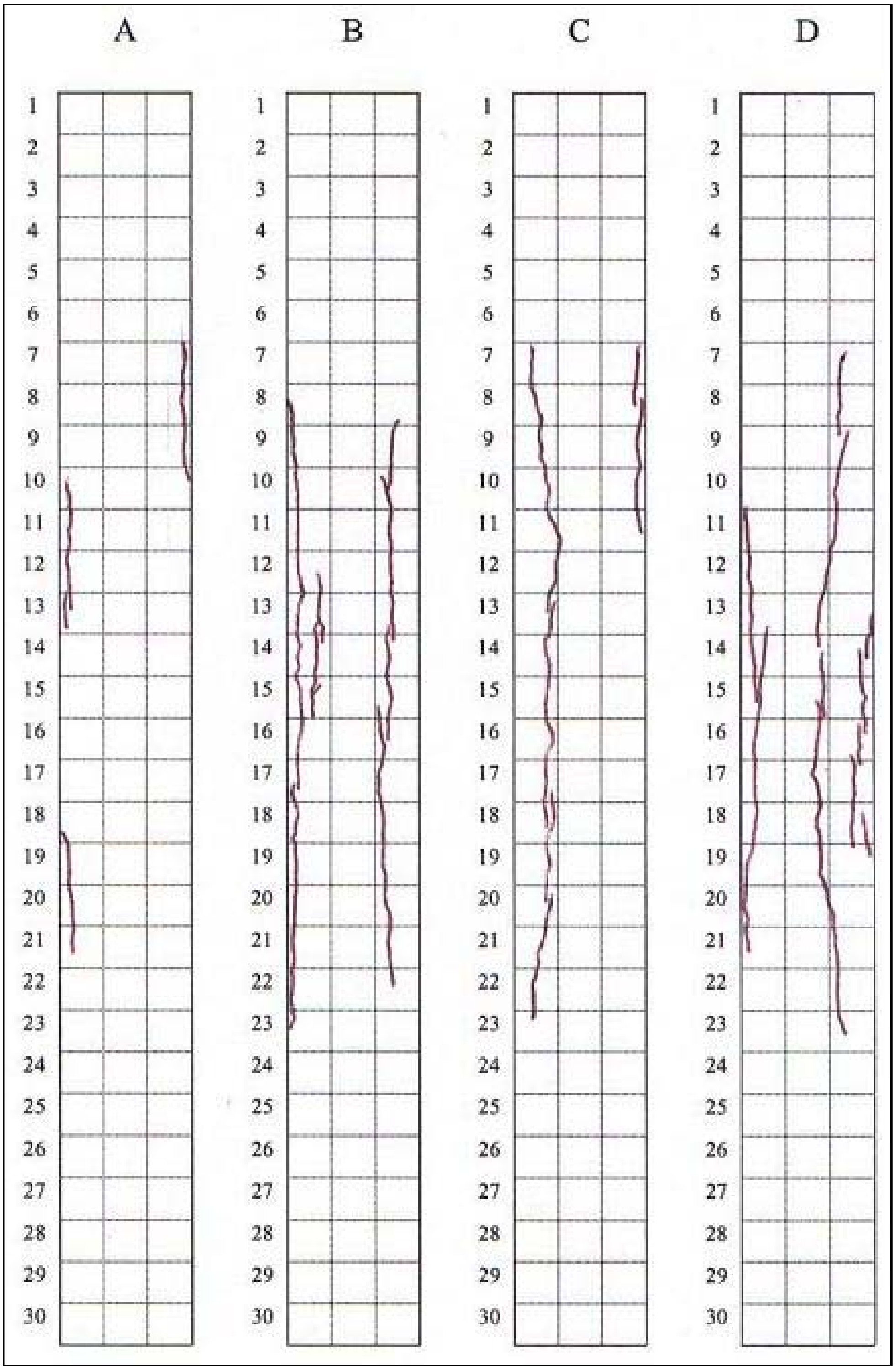


Figure A.6 Crack Mapping of #66 at 25%

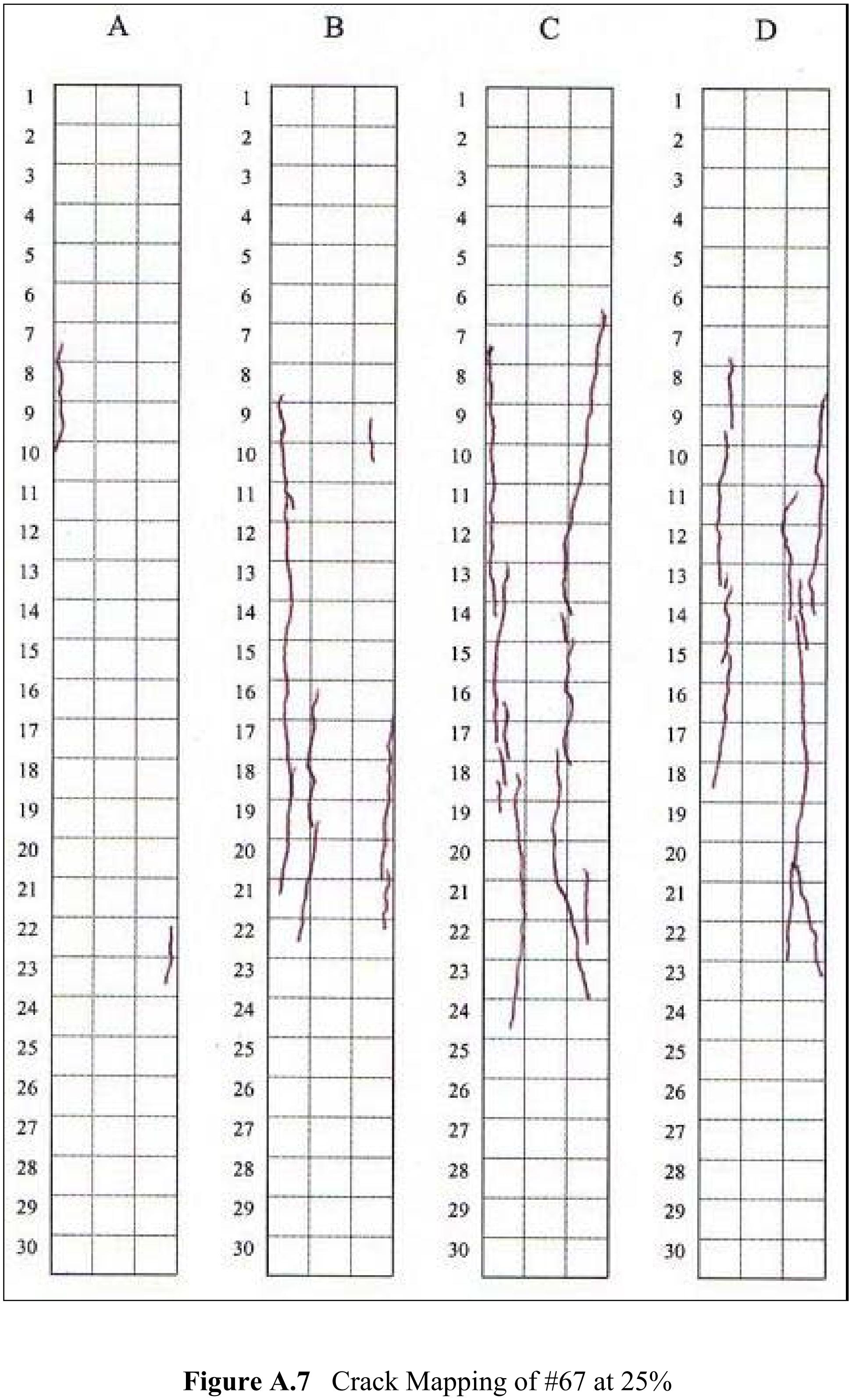


Figure A.7 Crack Mapping of #67 at 25%

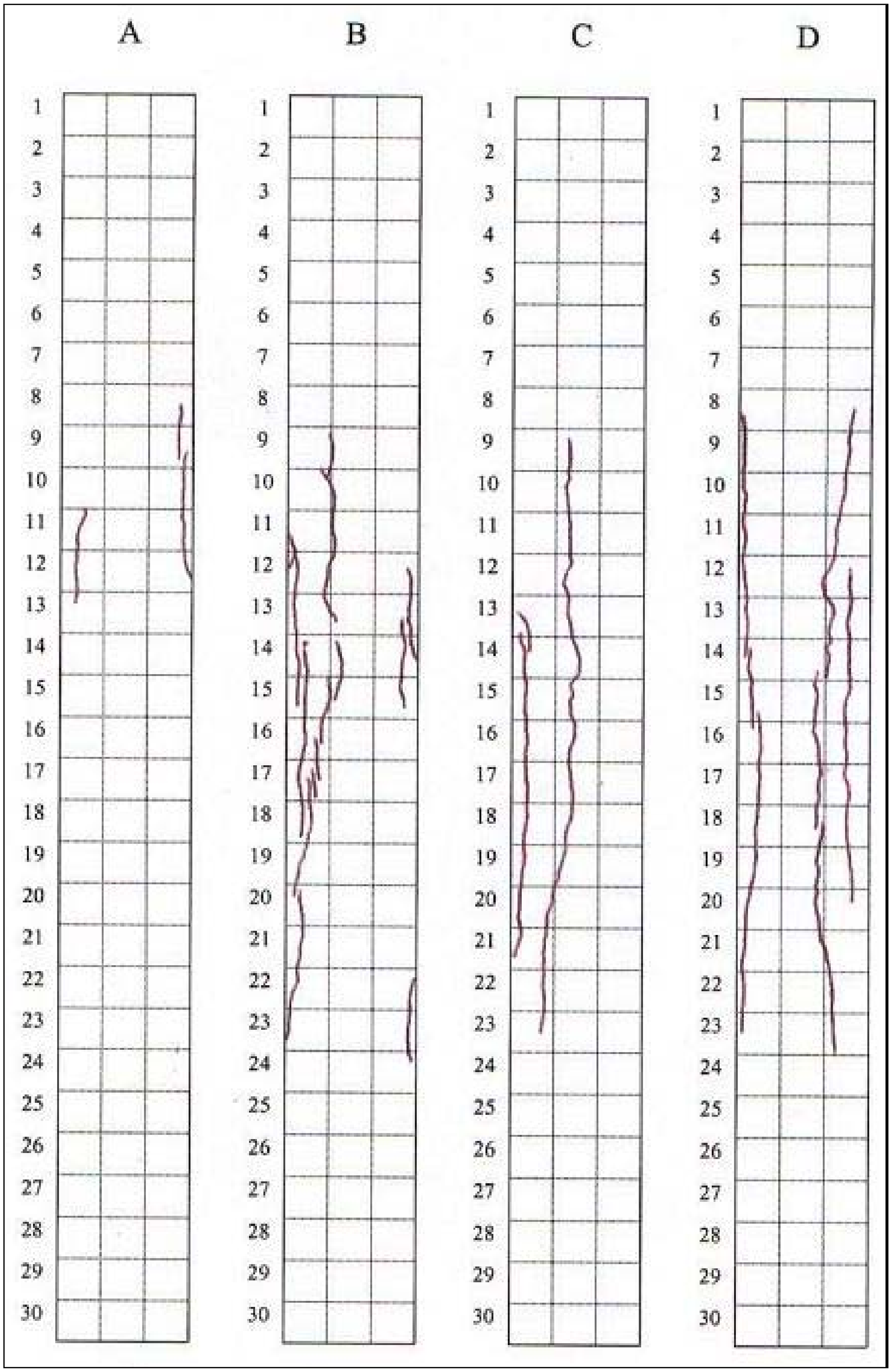


Figure A.8 Crack Mapping of #68 at 25%

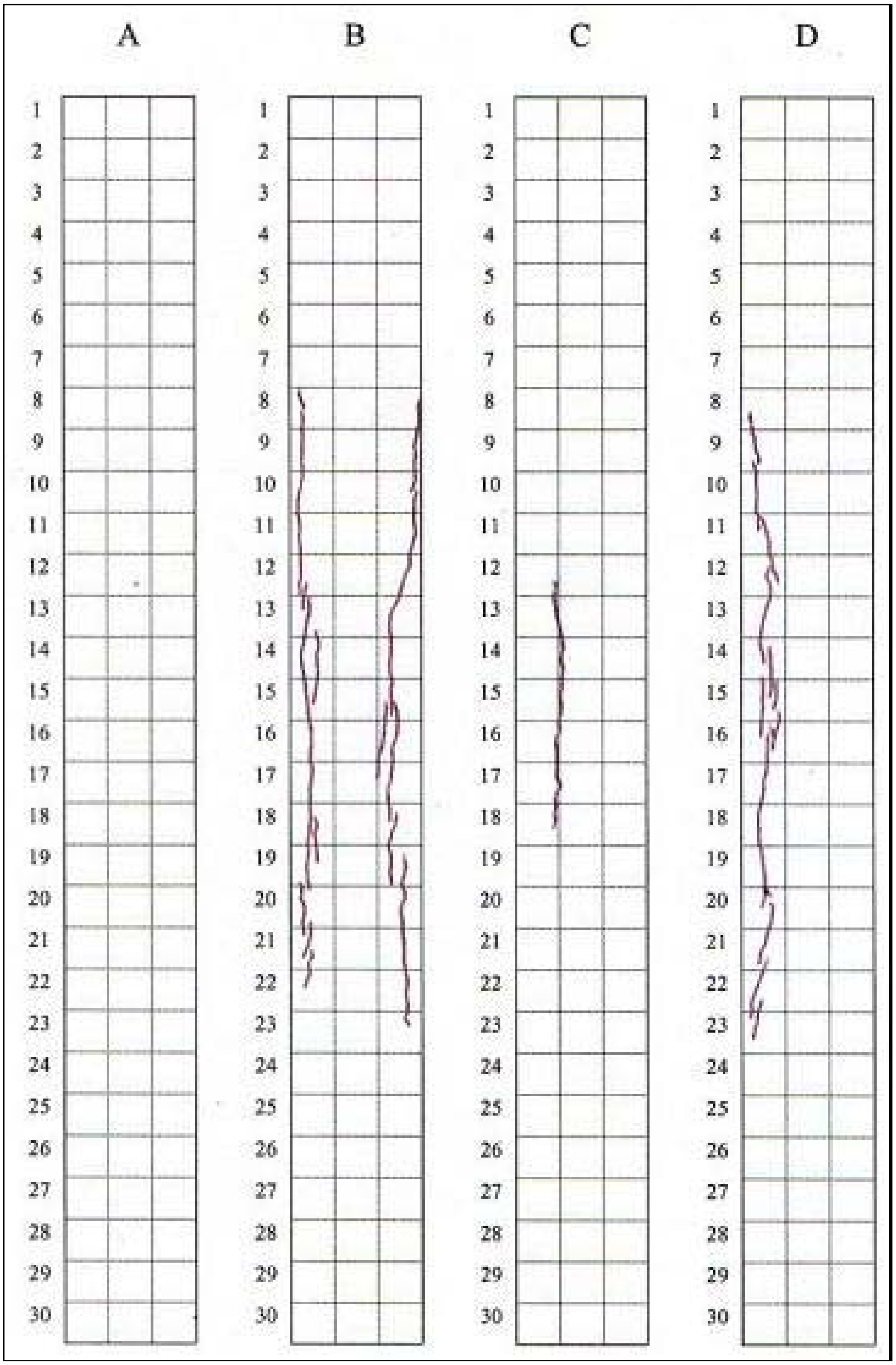


Figure A.9 Crack Mapping of #69 at 25%

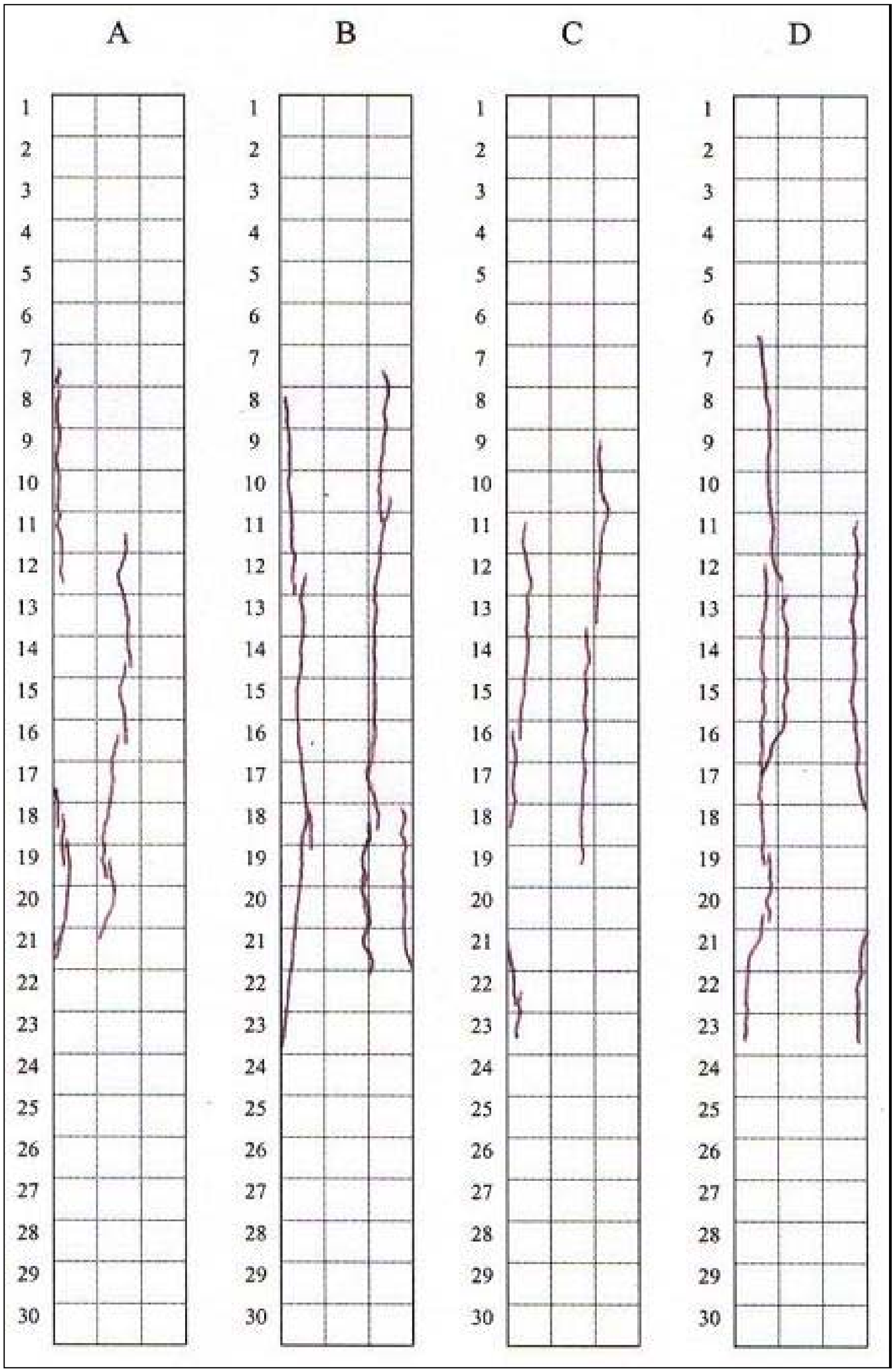


Figure A.10 Crack Mapping of #70 at 25%

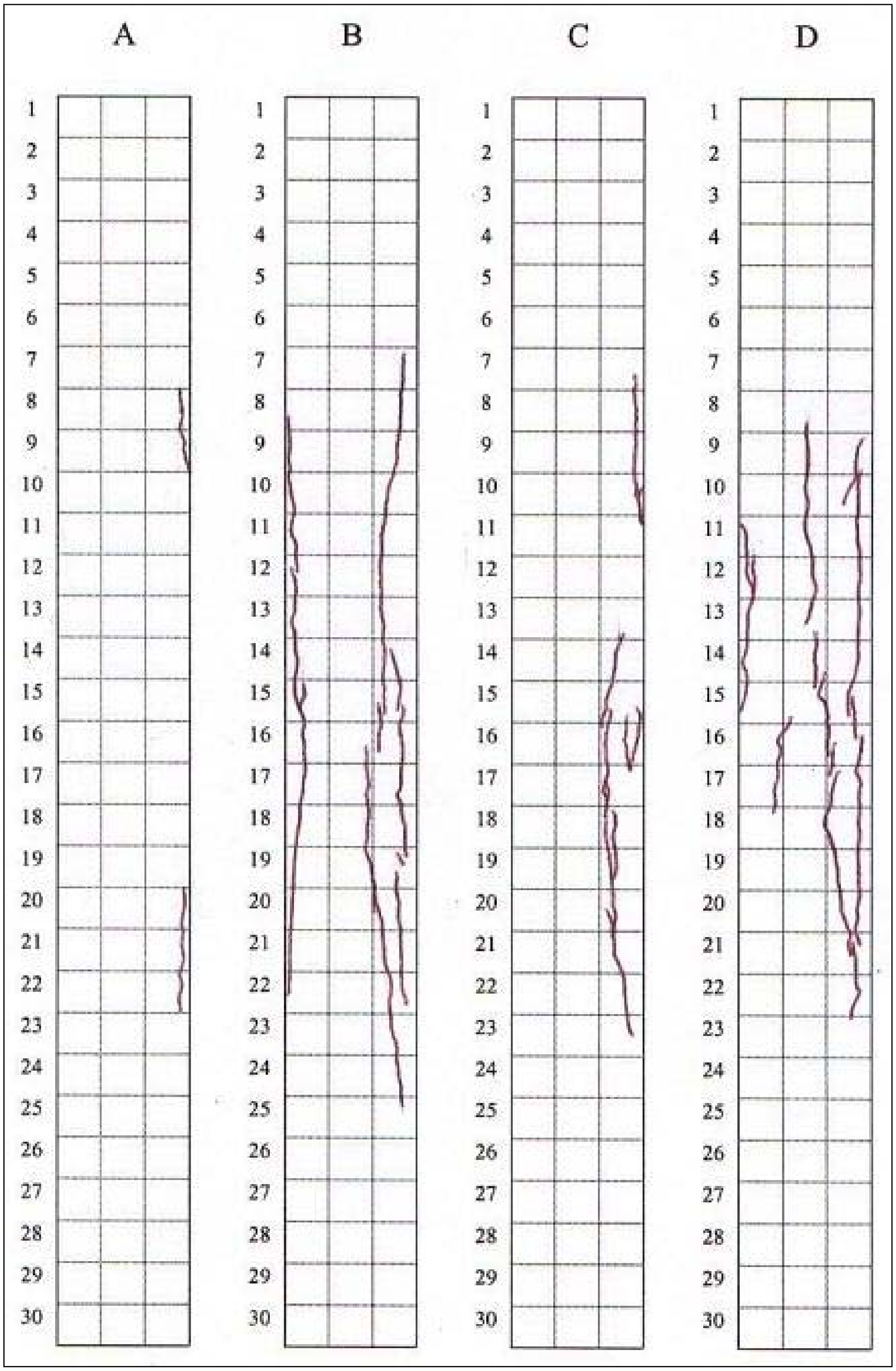


Figure A.11 Crack Mapping of #71 at 25%

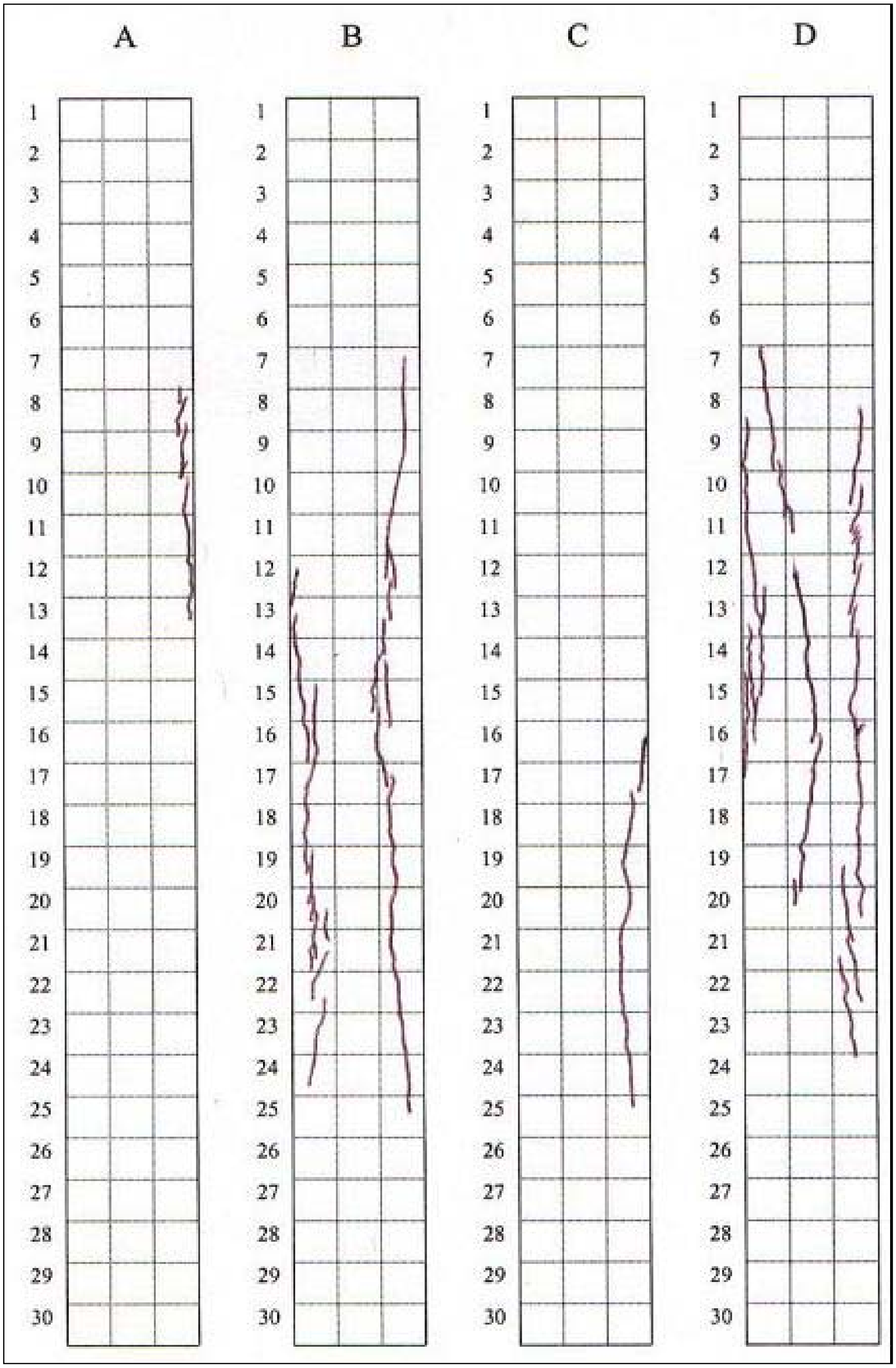


Figure A.12 Crack Mapping of #72 at 25%

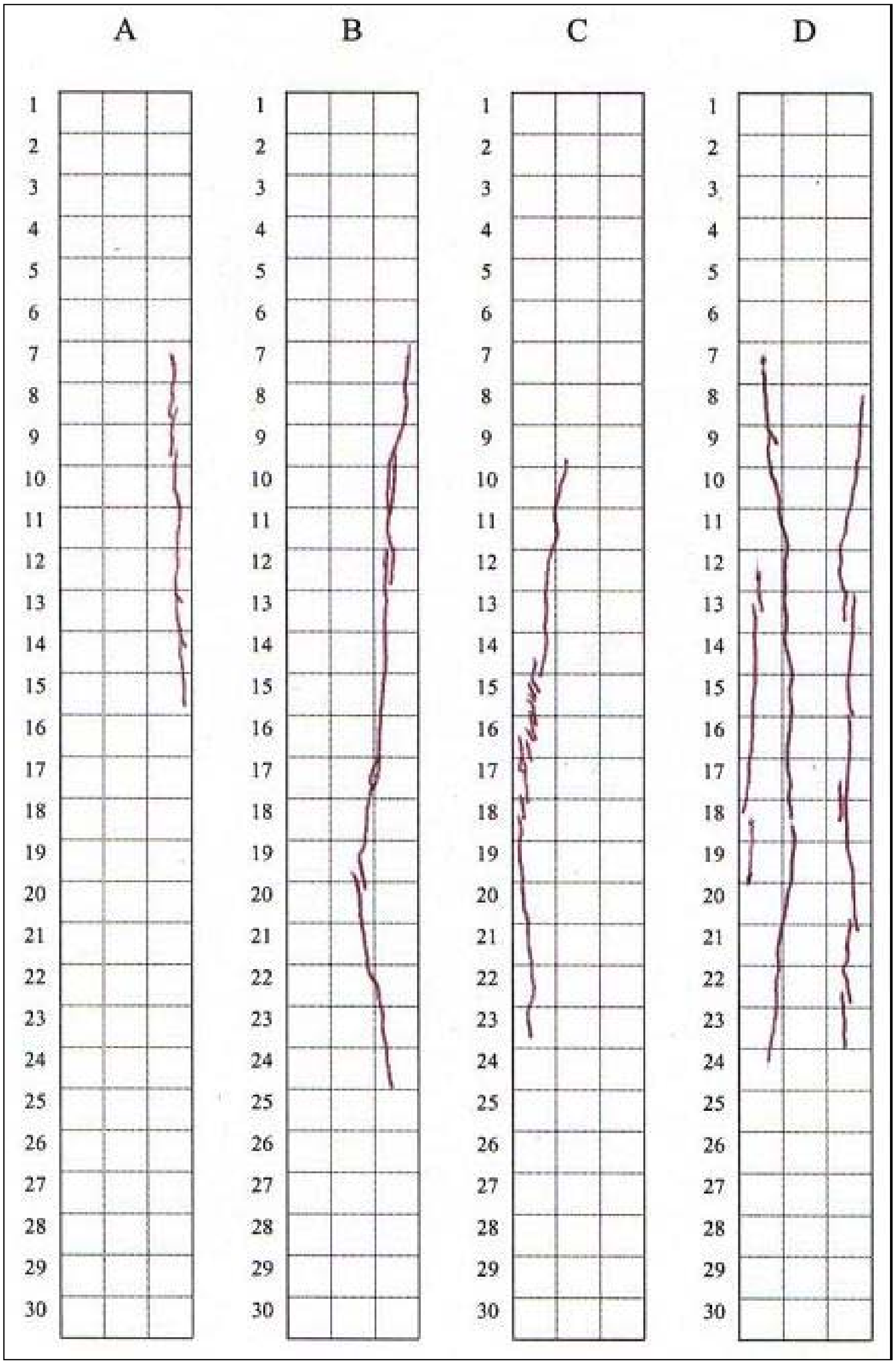


Figure A.13 Crack Mapping of #74 at 25%

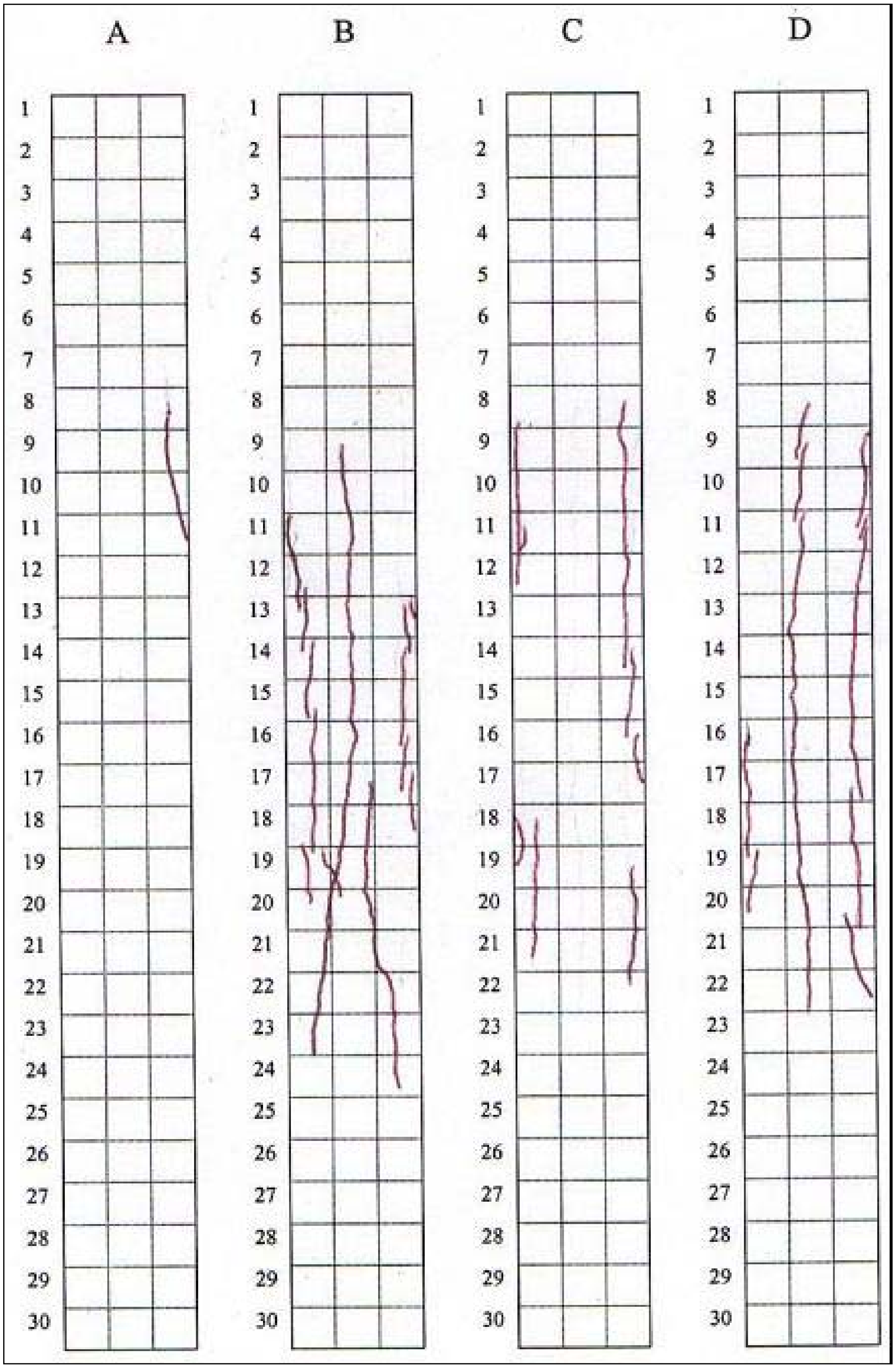


Figure A.14 Crack Mapping of #75 at 25%

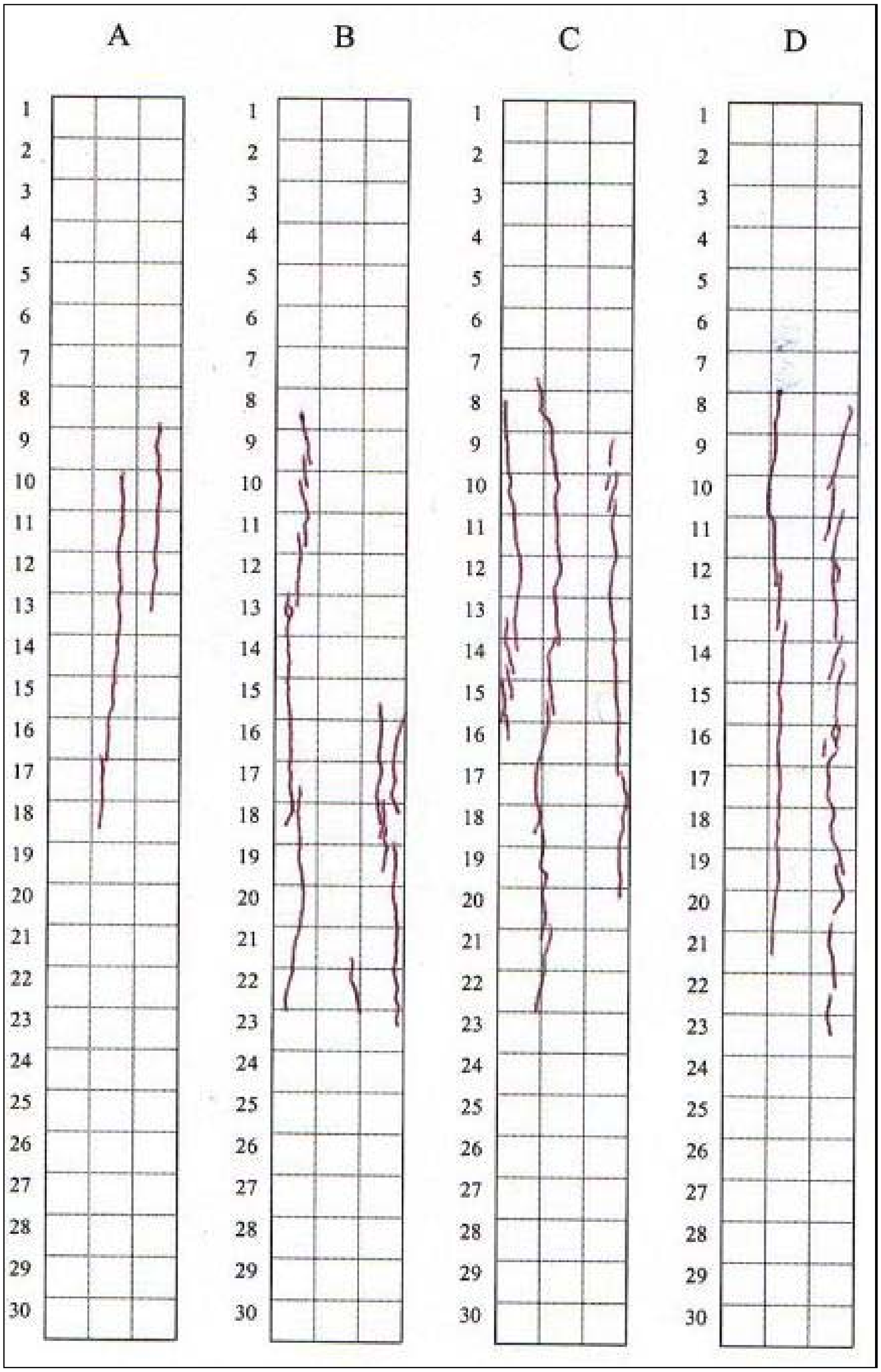


Figure A.15 Crack Mapping of #76 at 25%

A.2 Crack Drawings of 6ft Specimens at 25% of Targeted Steel Loss

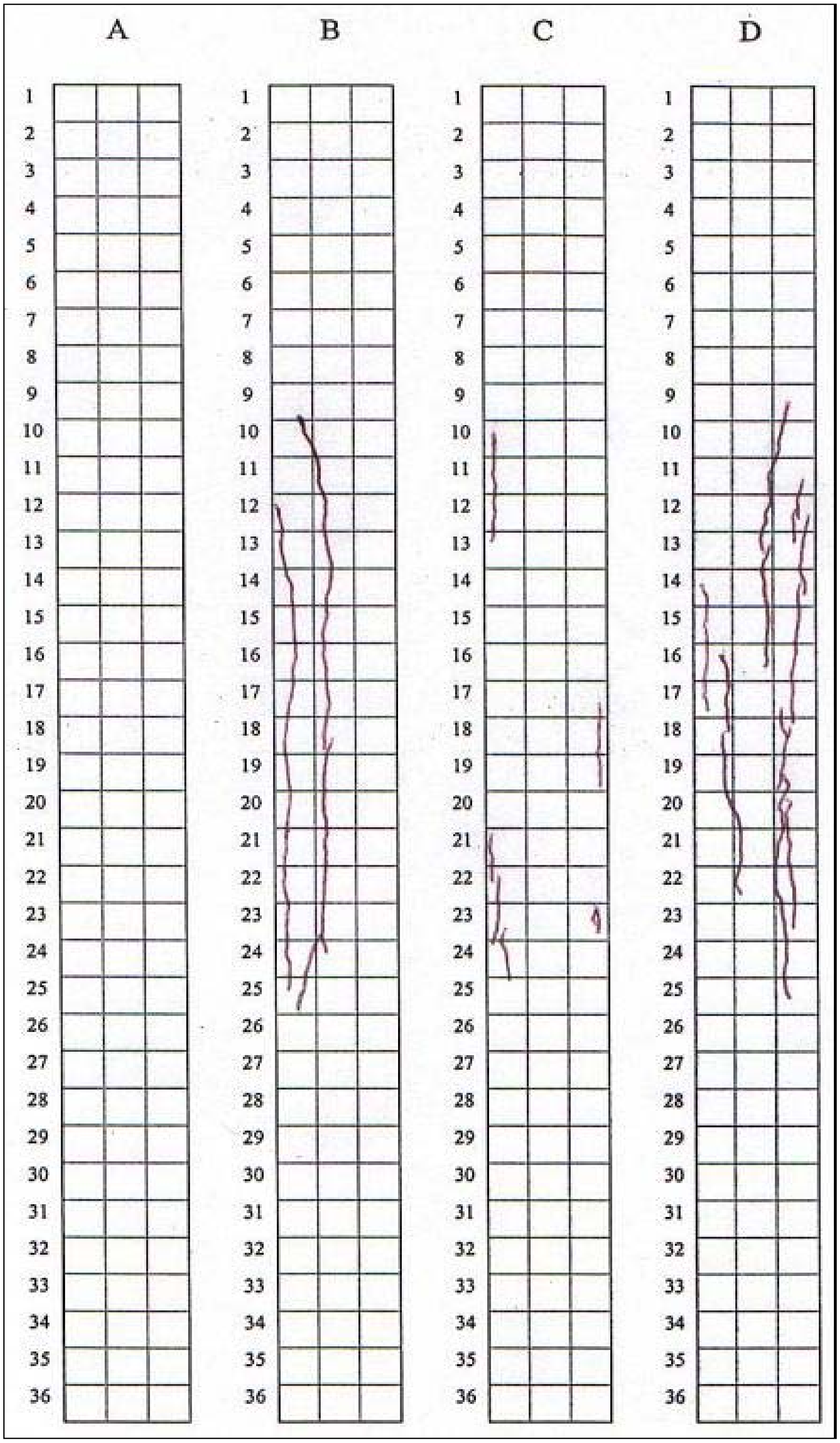


Figure A.16 Crack Mapping of #29 at 25%

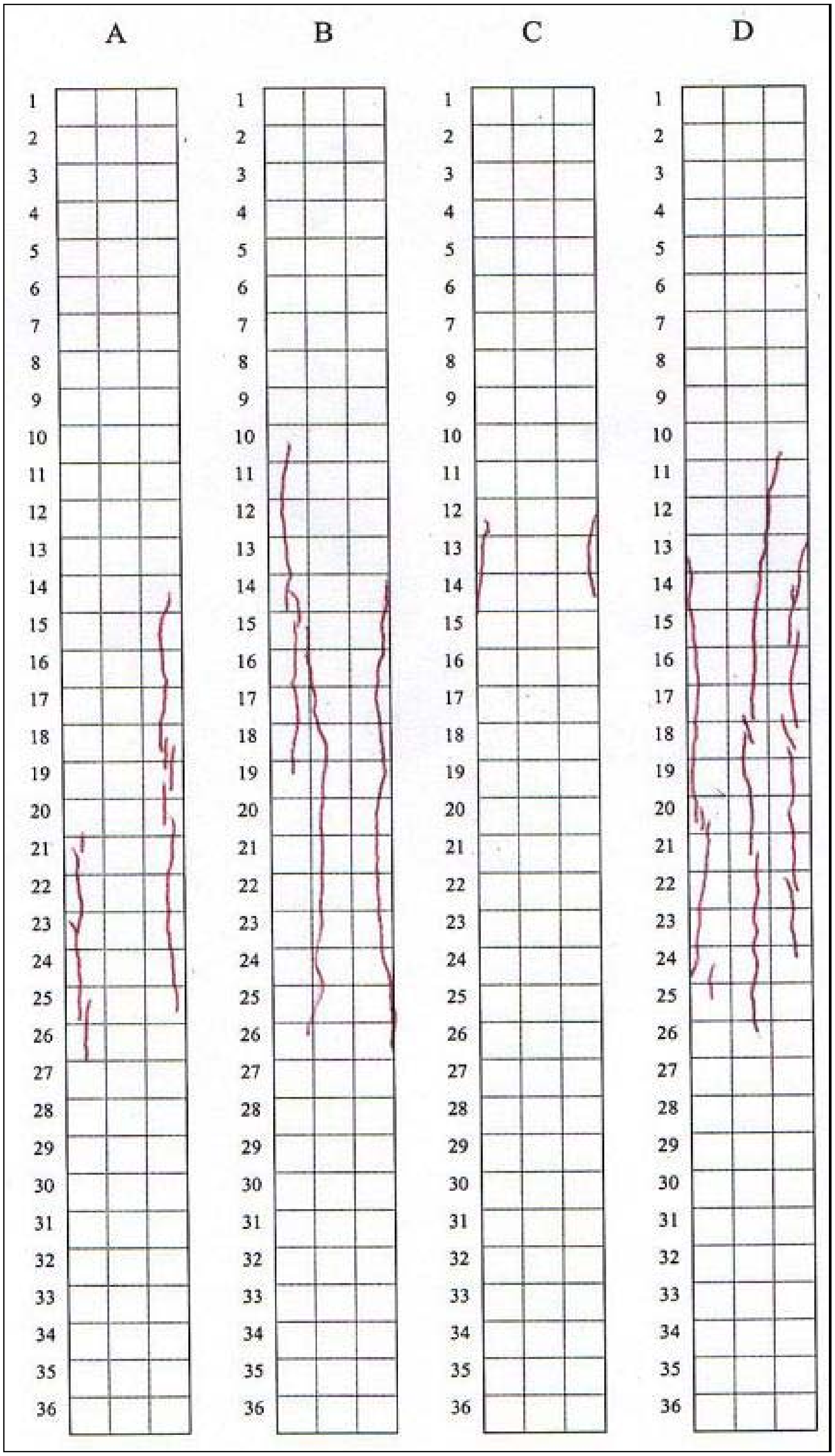


Figure A.17 Crack Mapping of #30 at 25%

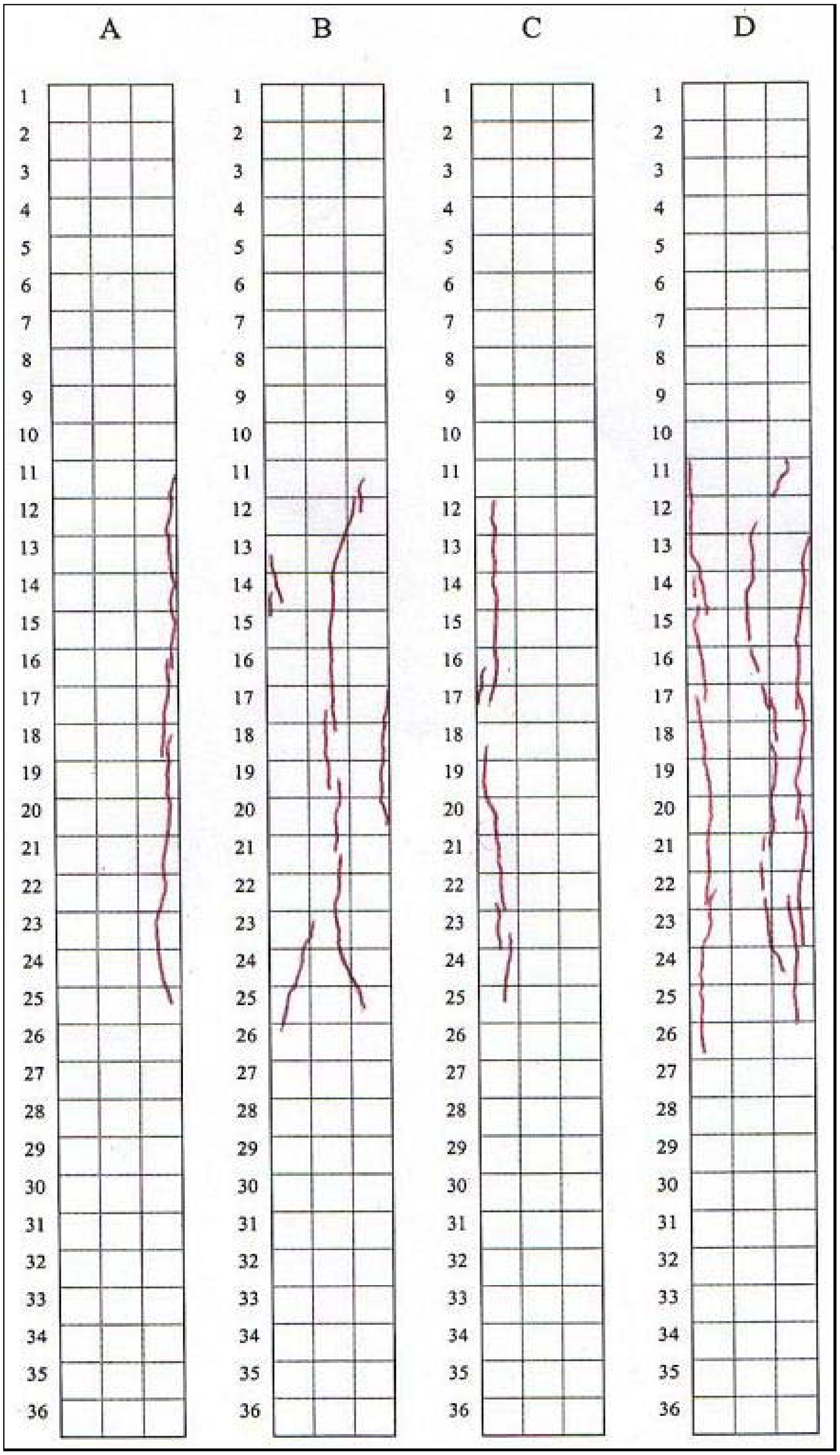


Figure A.18 Crack Mapping of #31 at 25%

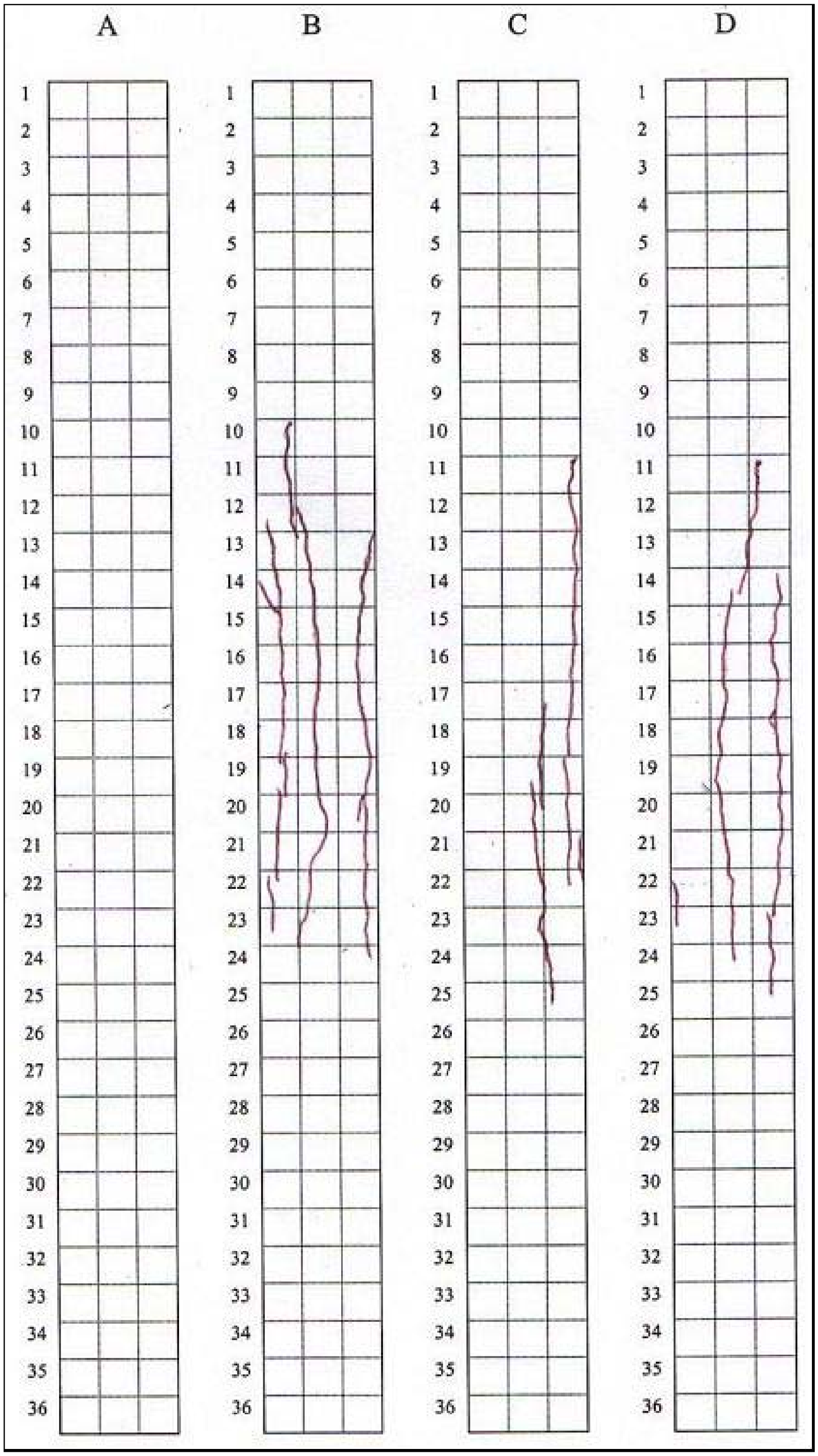


Figure A.19 Crack Mapping of #32 at 25%

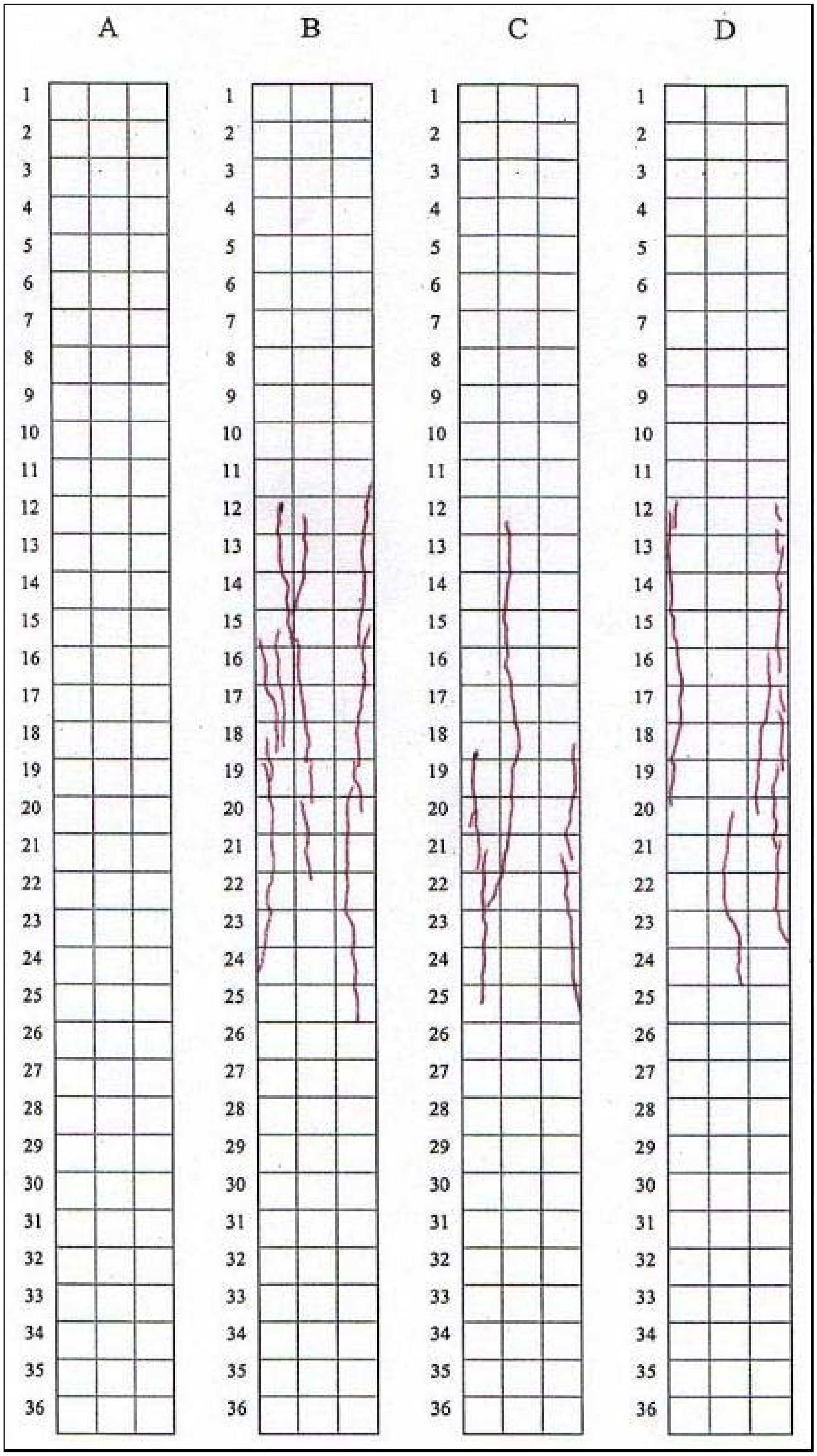


Figure A.20 Crack Mapping of #33 at 25%

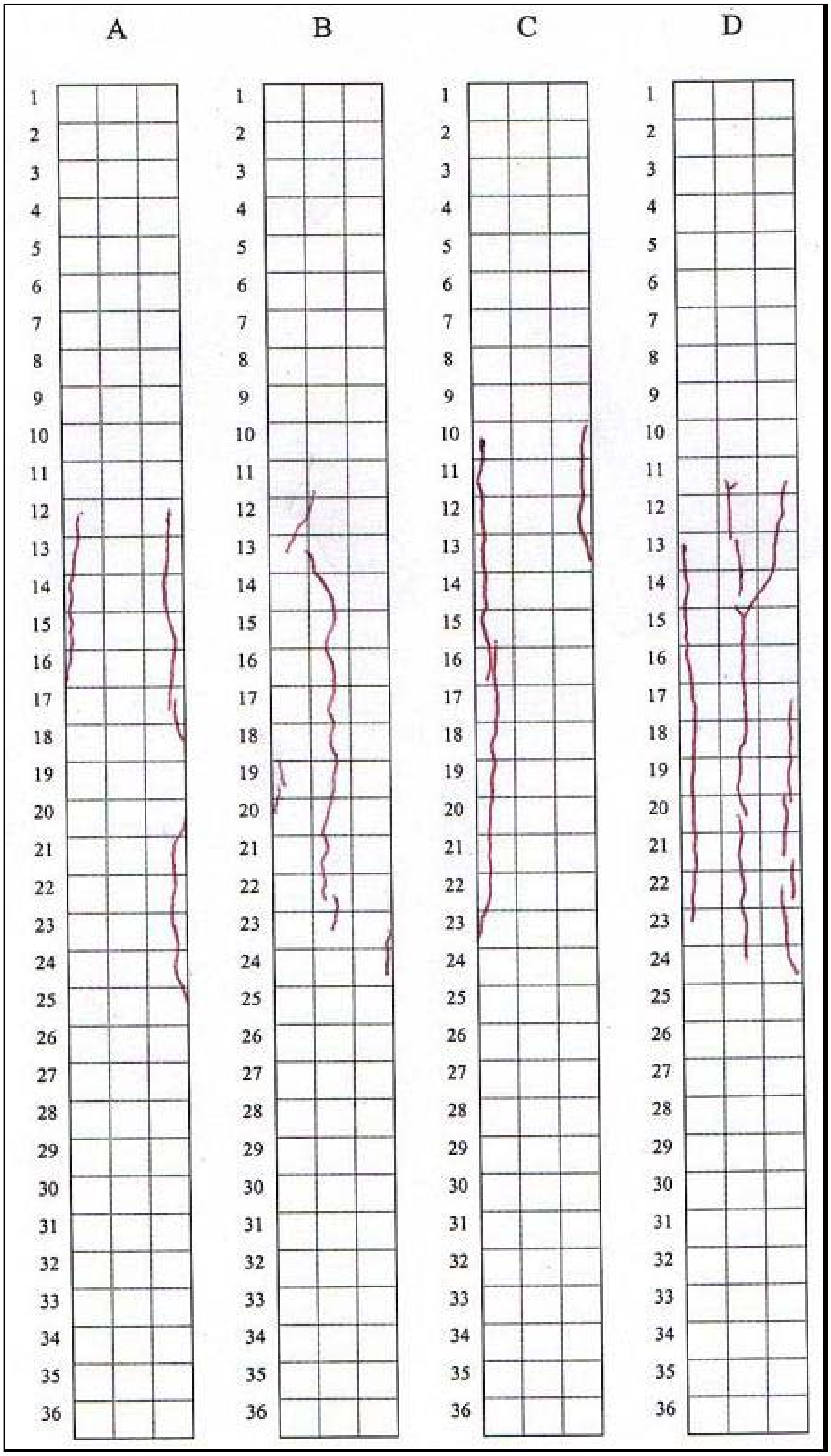


Figure A.21 Crack Mapping of #34 at 25%

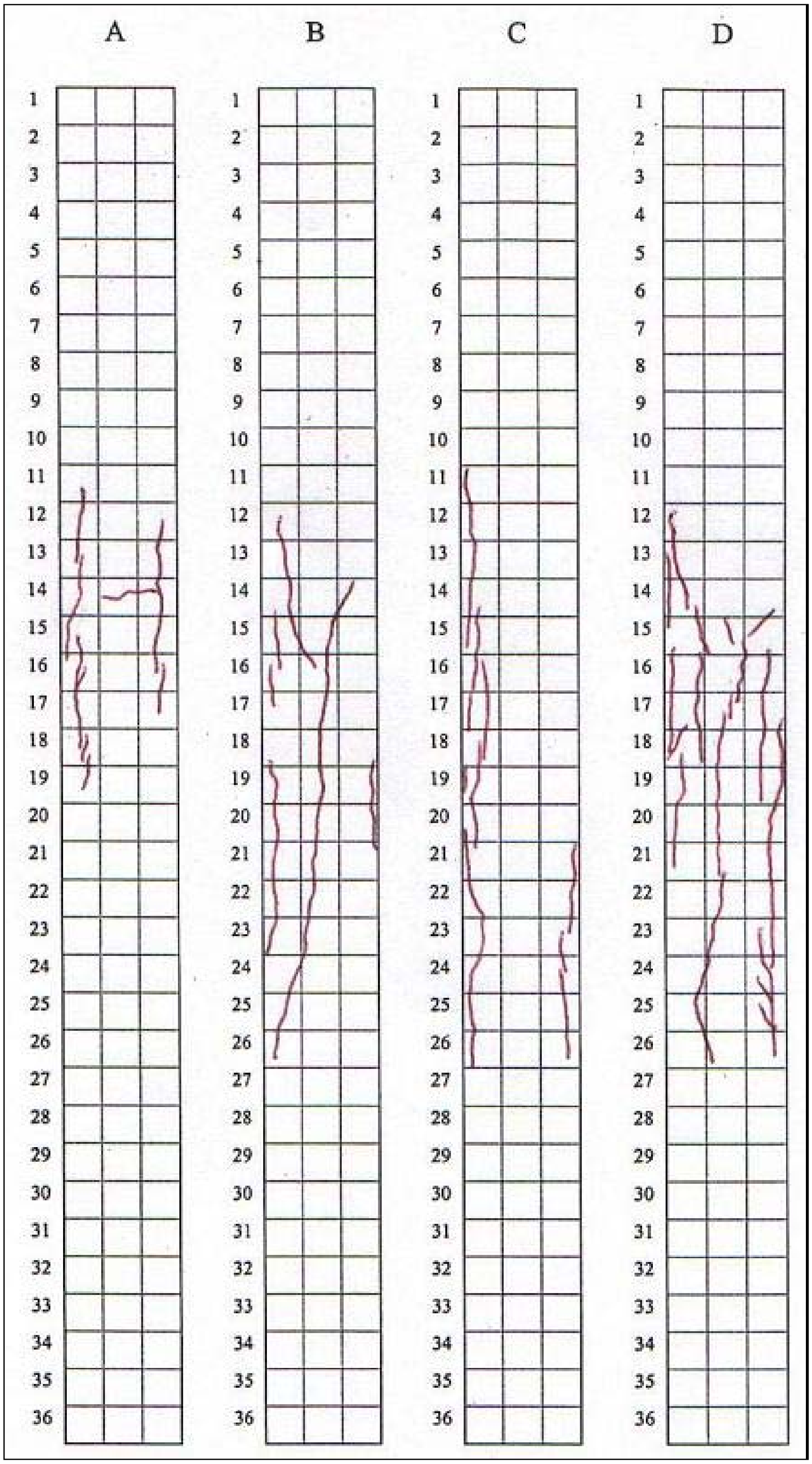


Figure A.22 Crack Mapping of #35 at 25%

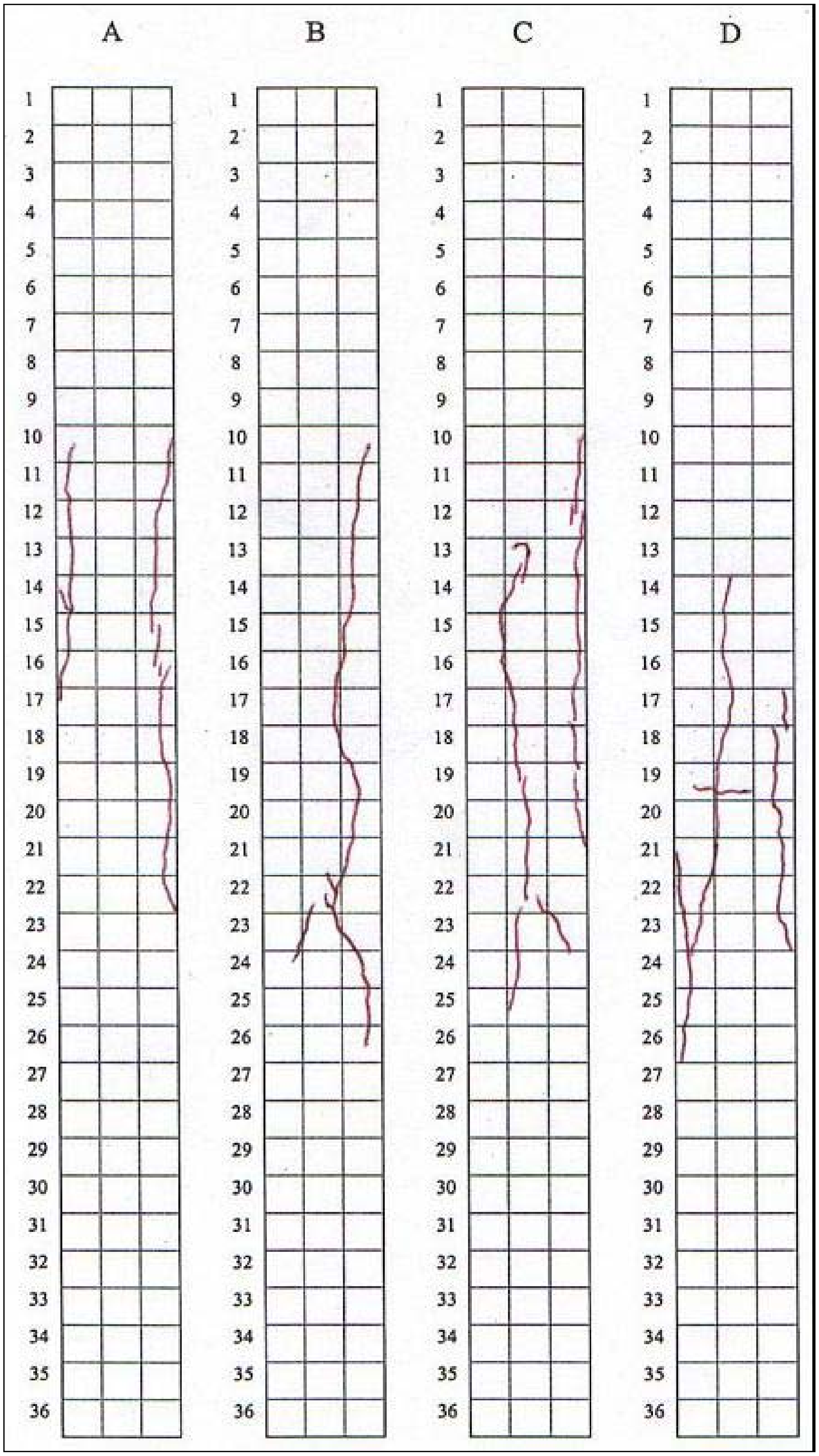


Figure A.23 Crack Mapping of #36 at 25%

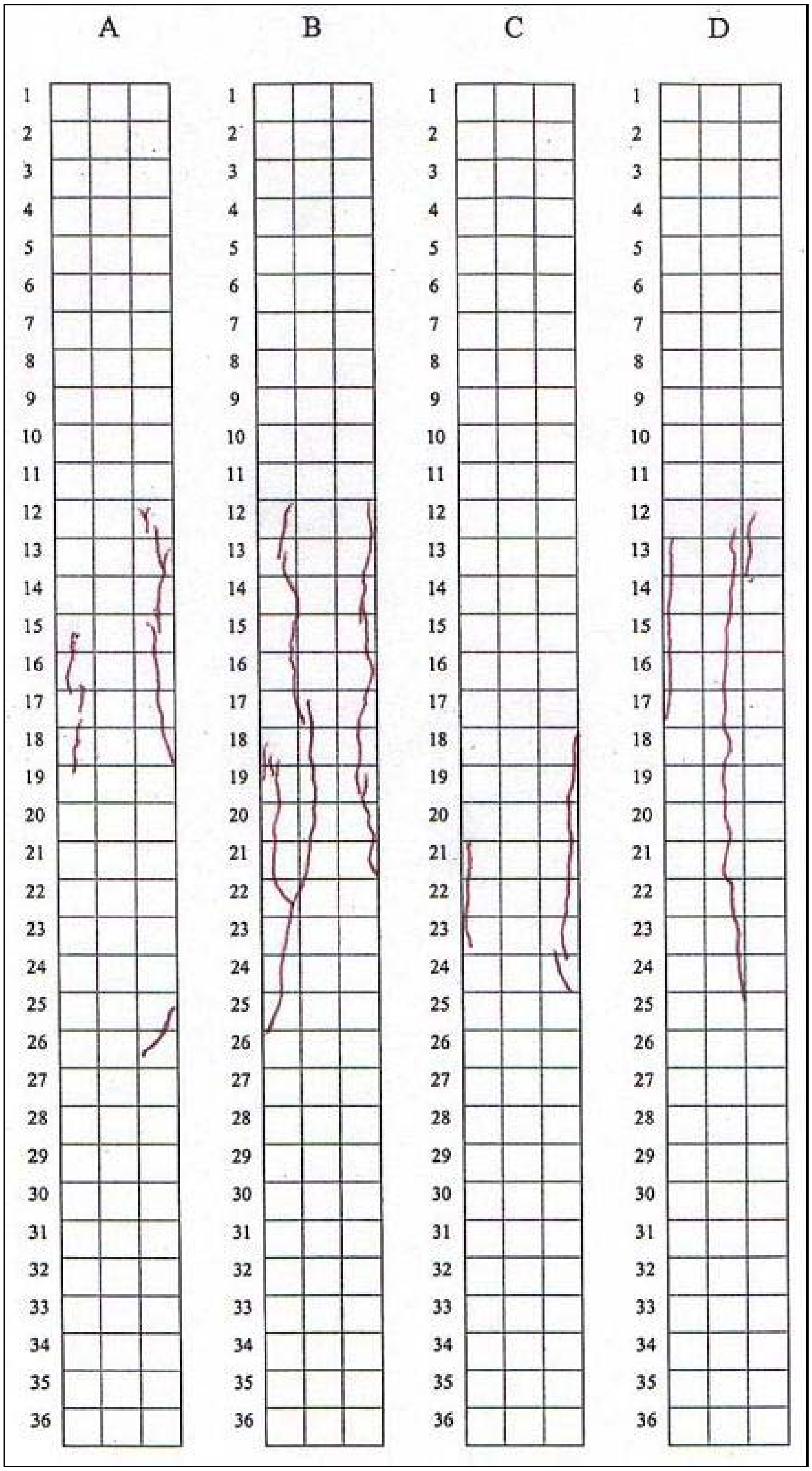


Figure A.24 Crack Mapping of #37 at 25%

Appendix B

Result Drawings

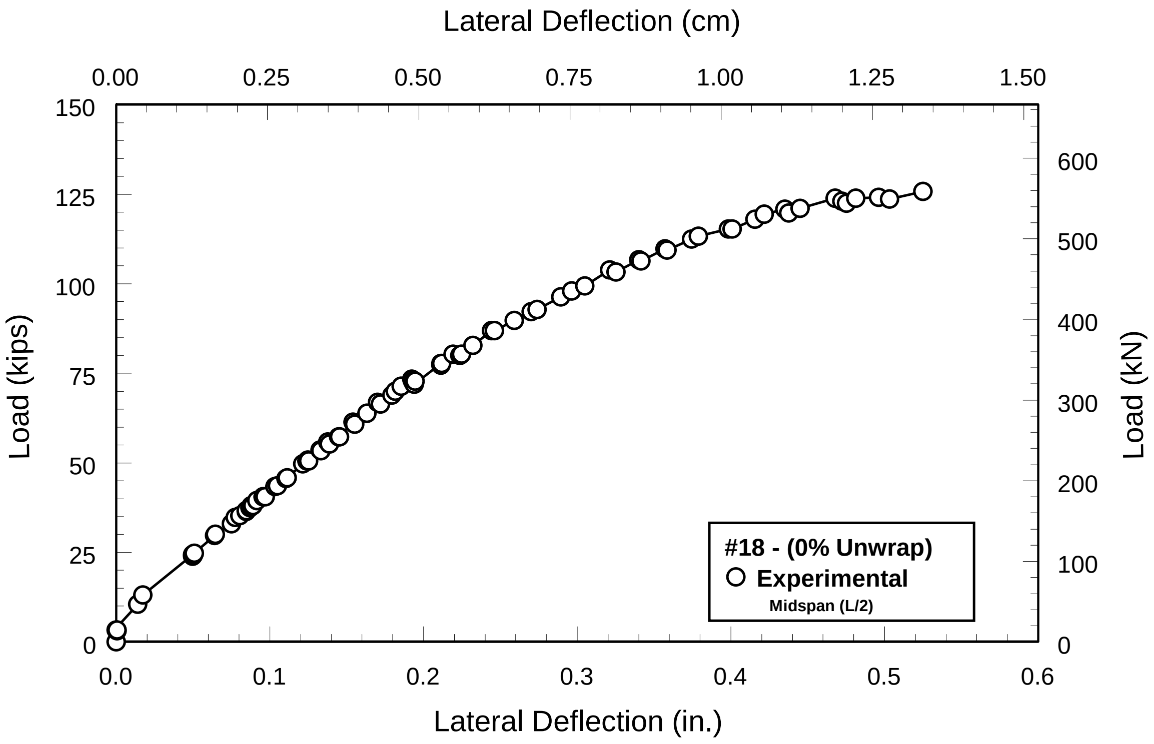


Figure B.1 Load v Deflection Plot for Specimen 18 (Control).

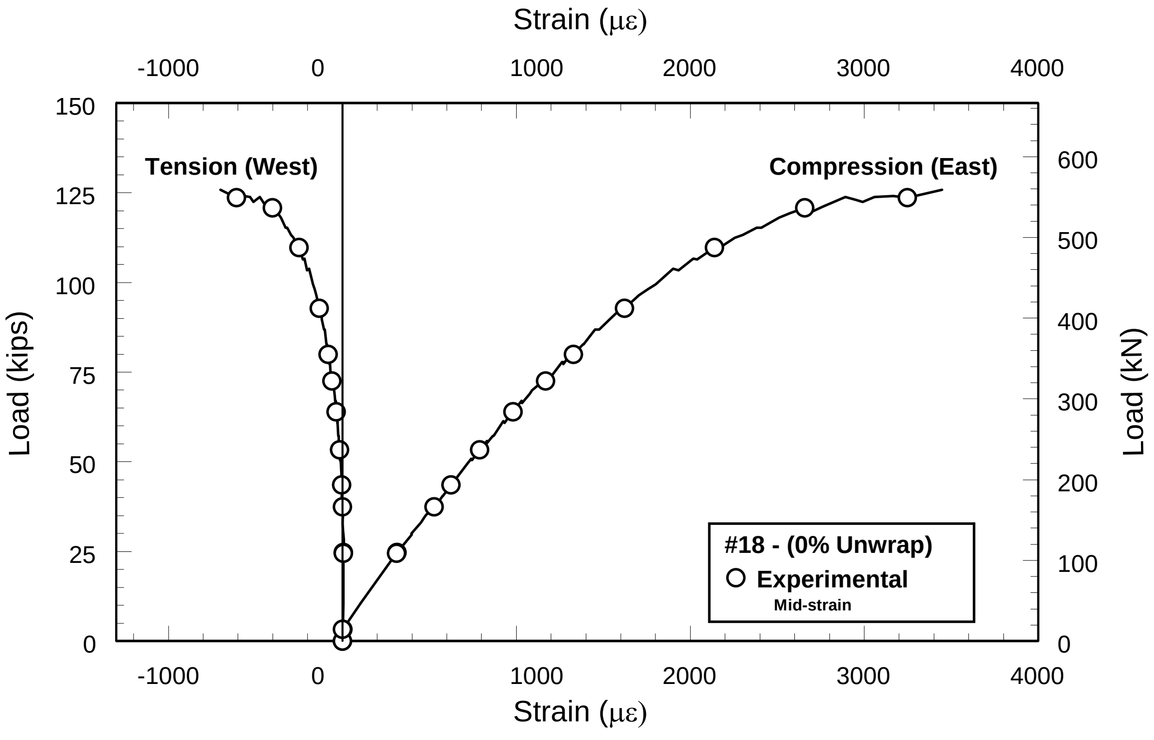


Figure B.2 Load v Strain Variation for Specimen 18 (Control).

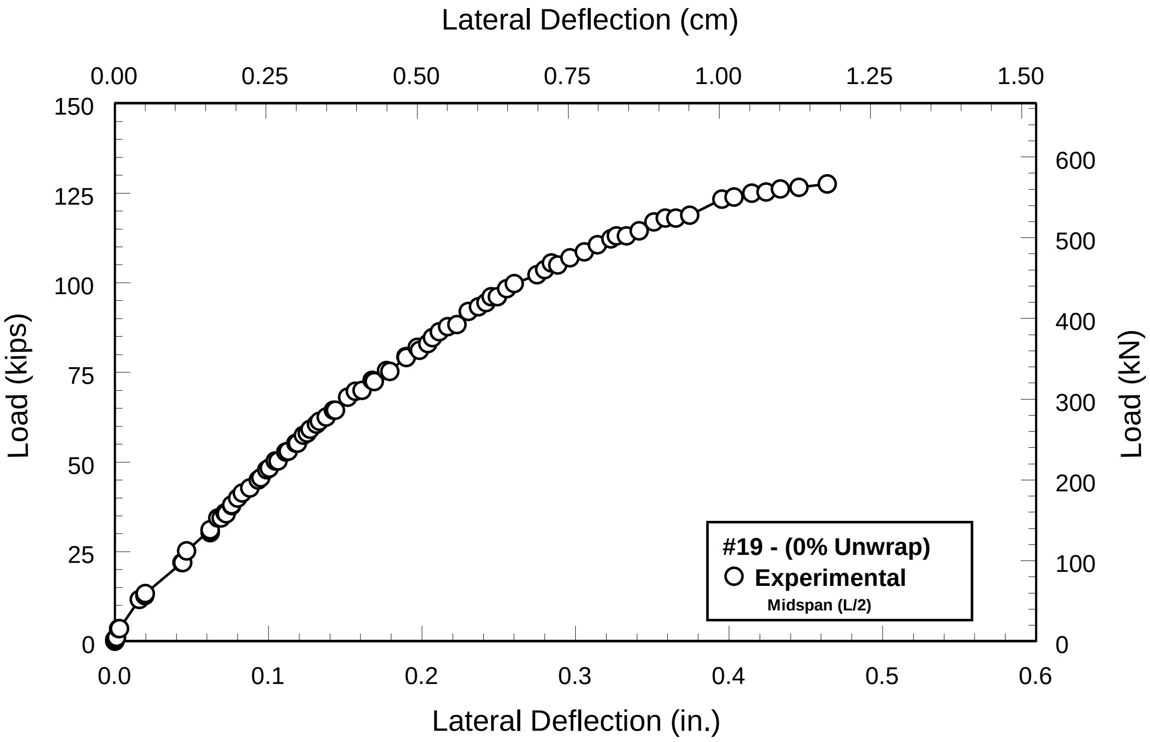


Figure B.3 Load v Deflection Plot for Specimen 19 (Control).

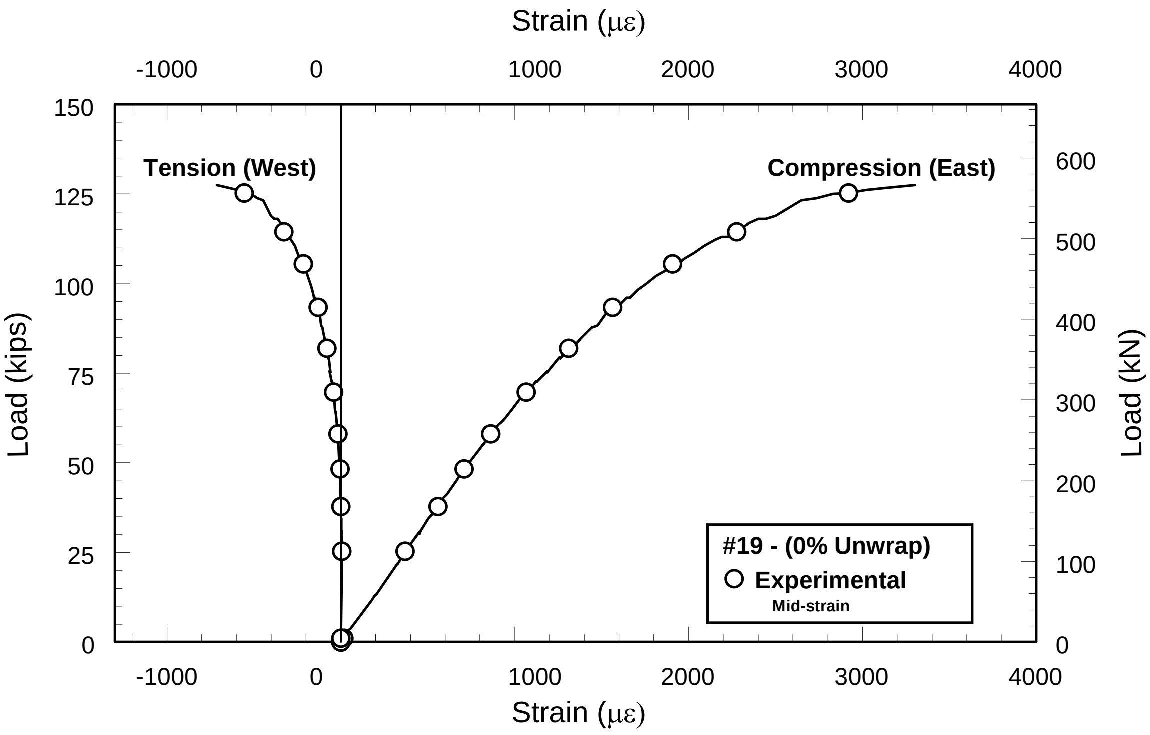


Figure B.4 Load v Strain Variation for Specimen 19 (Control).

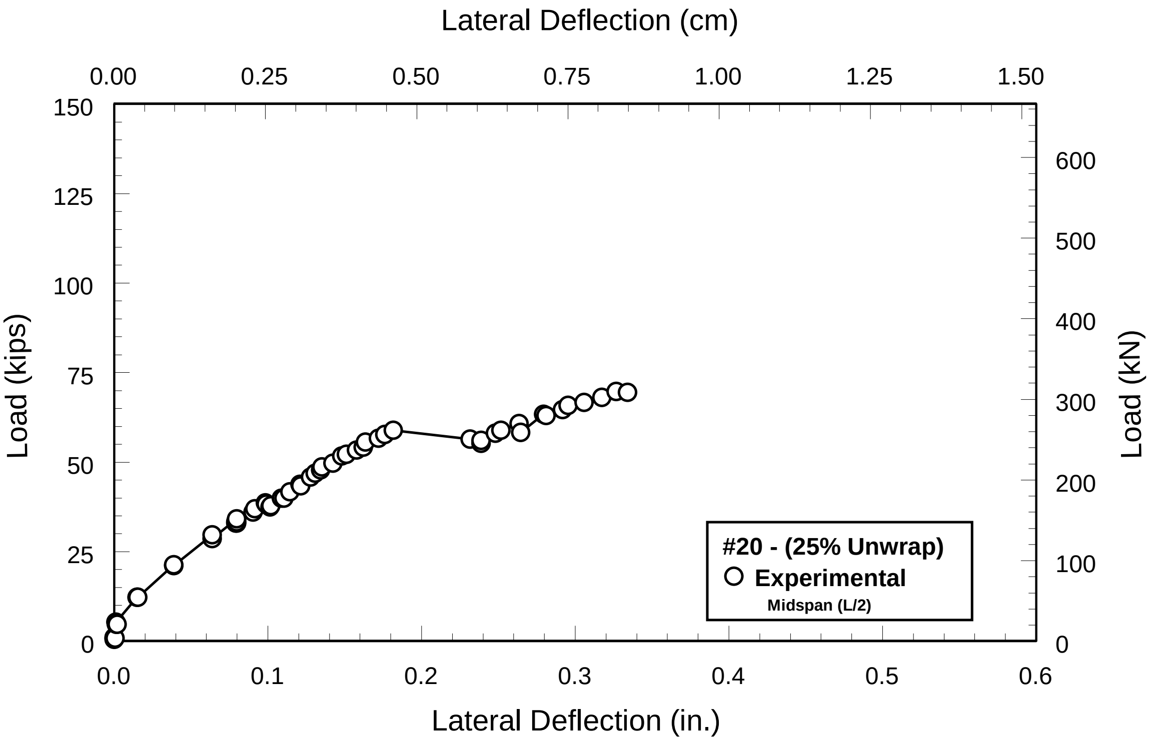


Figure B.5 Load v Deflection Plot for Specimen 20 (25% Control).

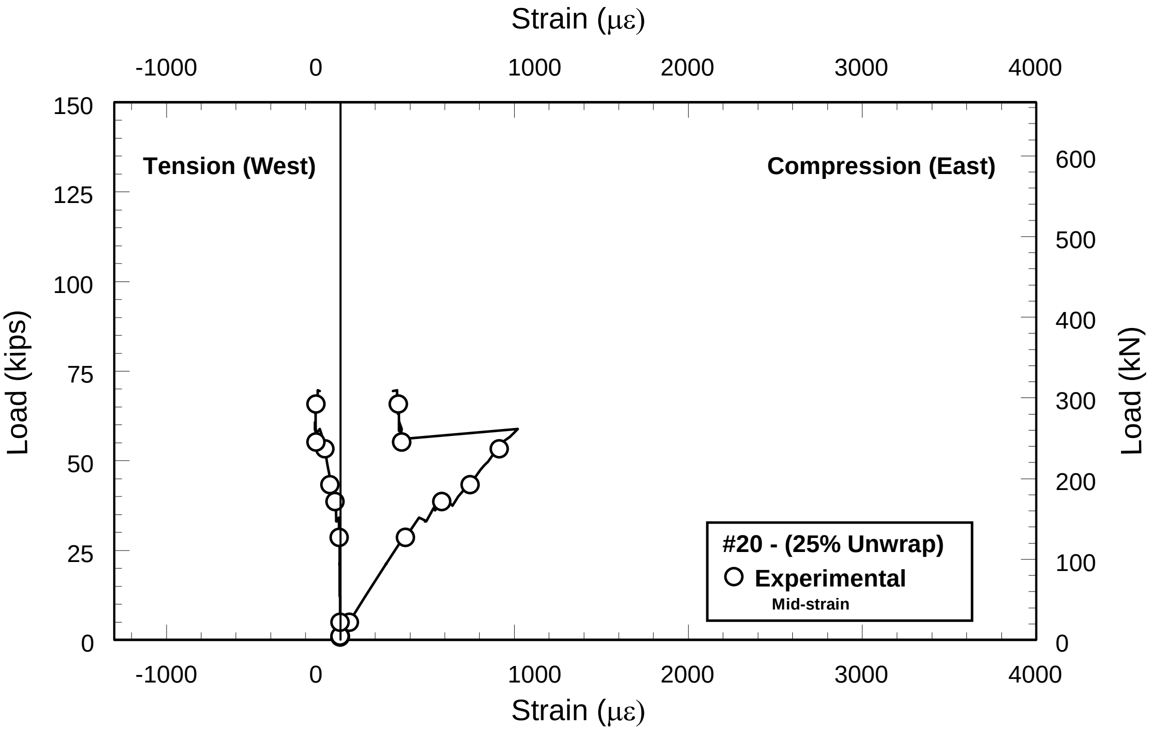


Figure B.6 Load v Strain Variation for Specimen 20 (25% Control).

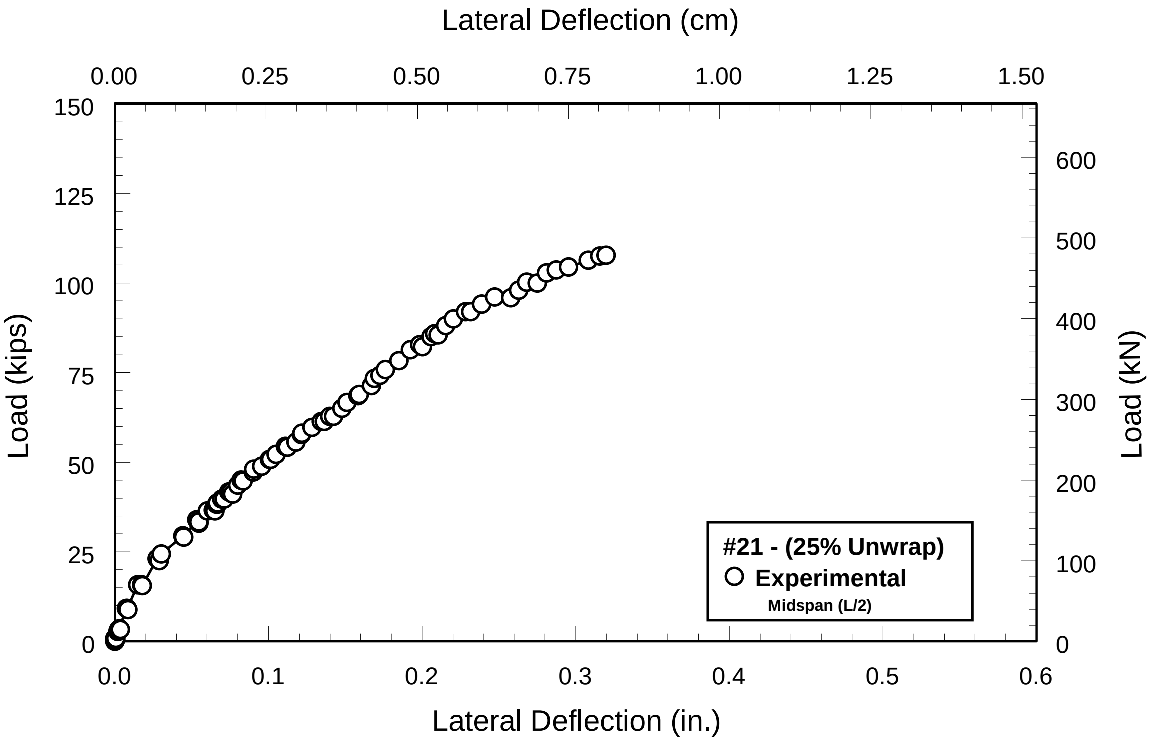


Figure B.7 Load v Deflection Plot for Specimen 21 (25% Control).

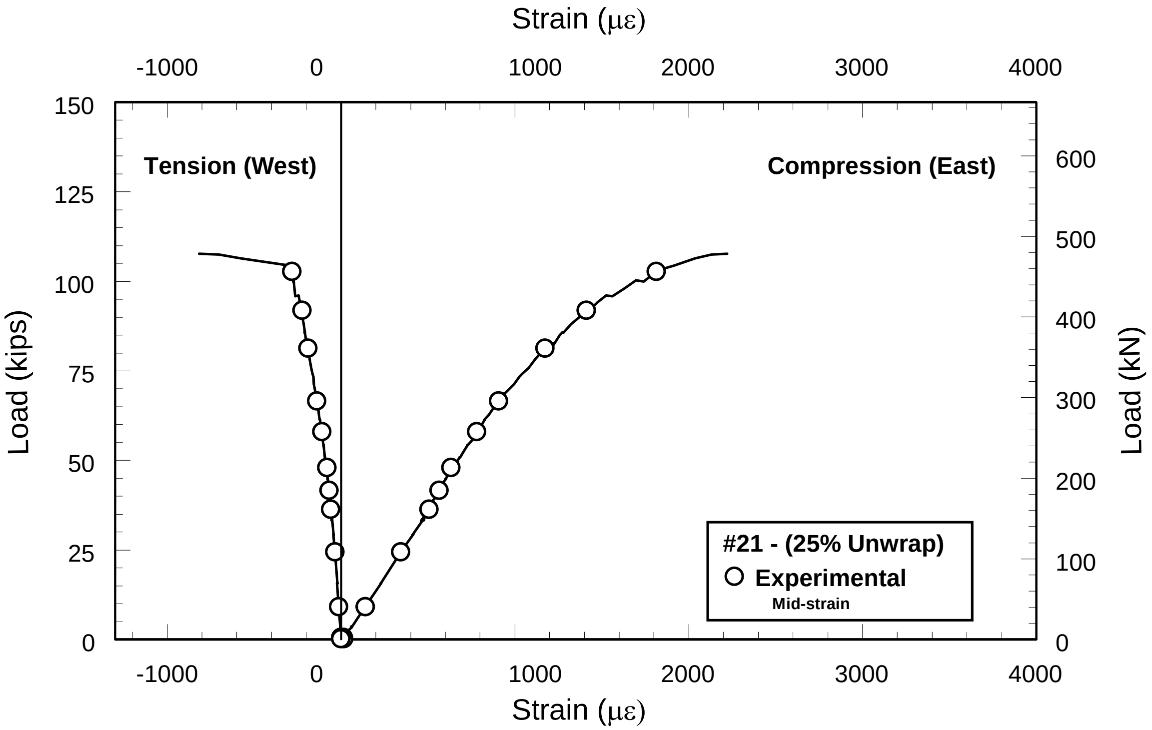


Figure B.8 Load v Strain Variation for Specimen 21 (25% Control).

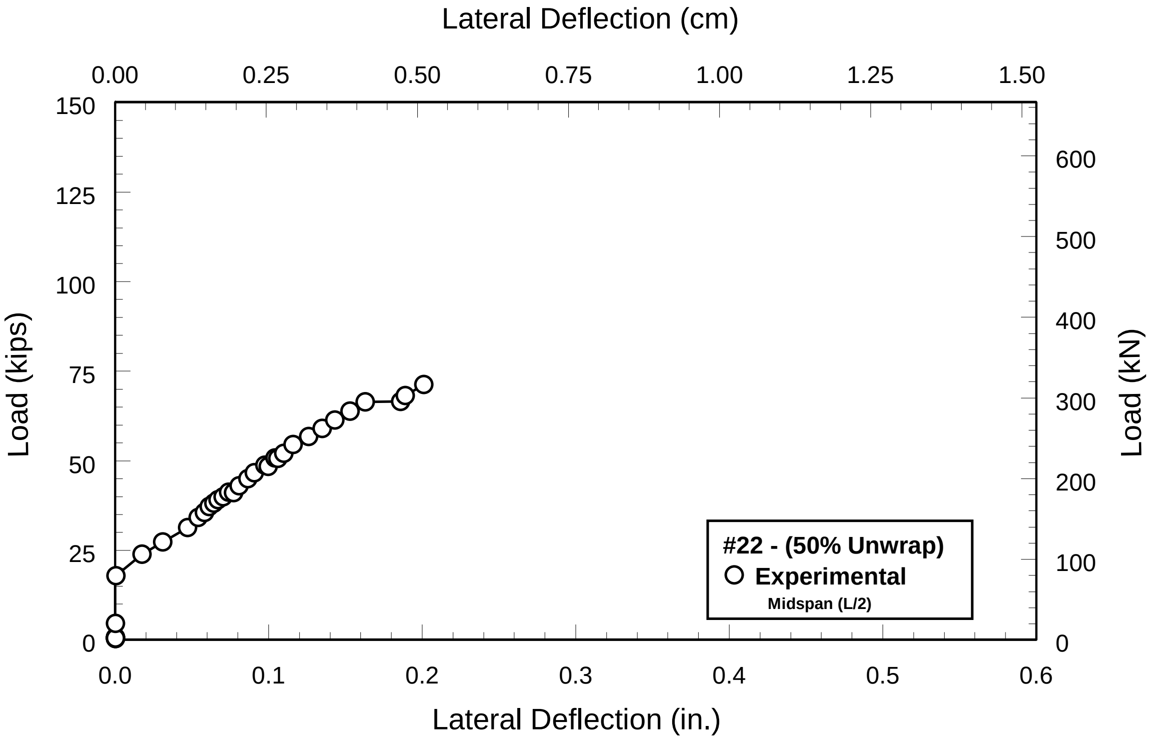


Figure B.9 Load v Deflection Plot for Specimen 22 (50% Control).

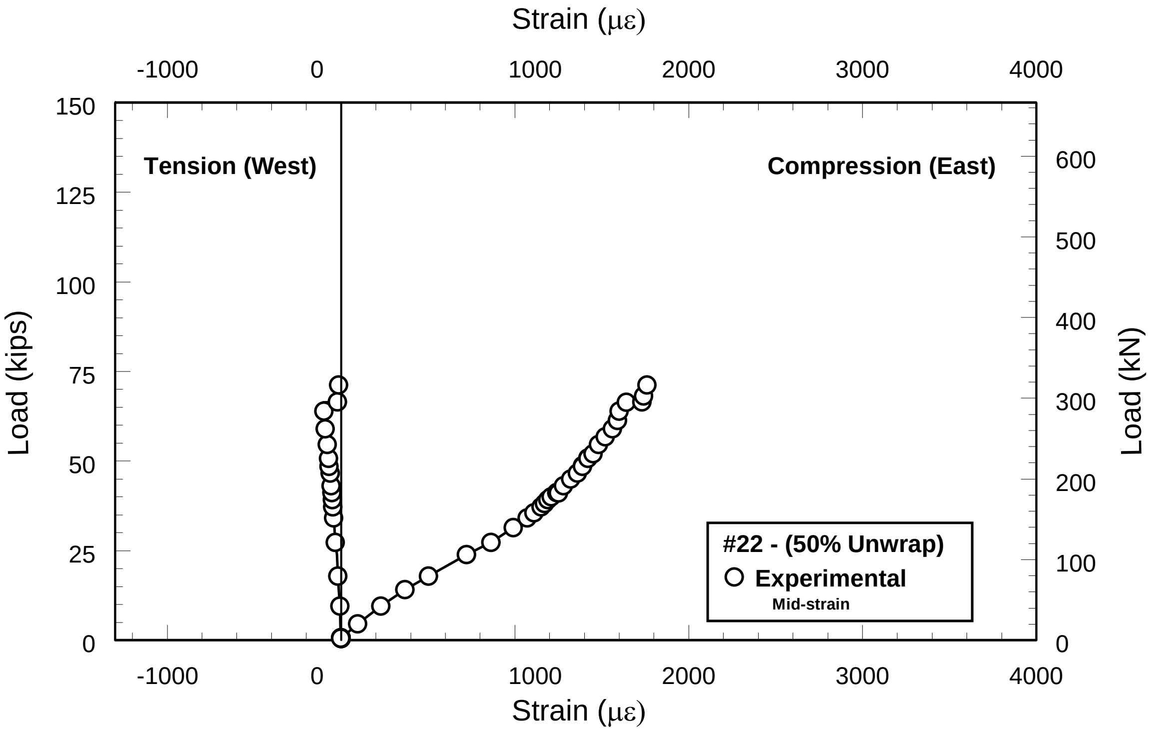


Figure B.10 Load v Strain Variation for Specimen 22 (50% Control).

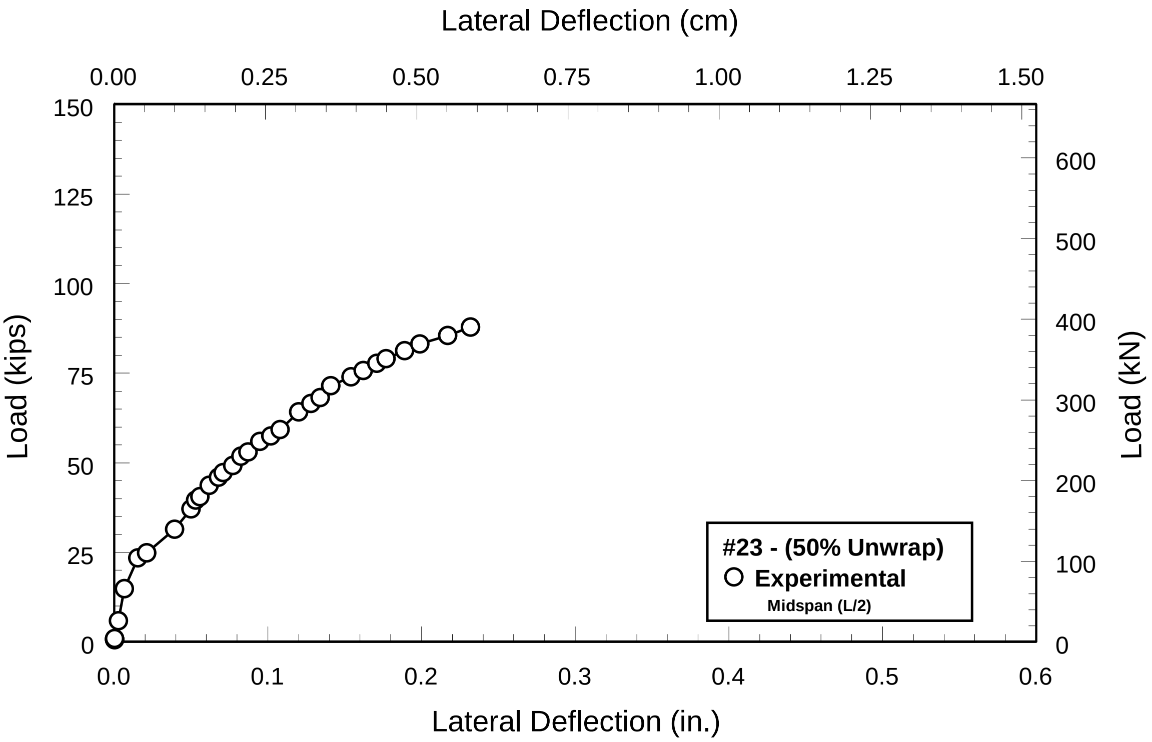


Figure B.11 Load v Deflection Plot for Specimen 23 (50% Control).

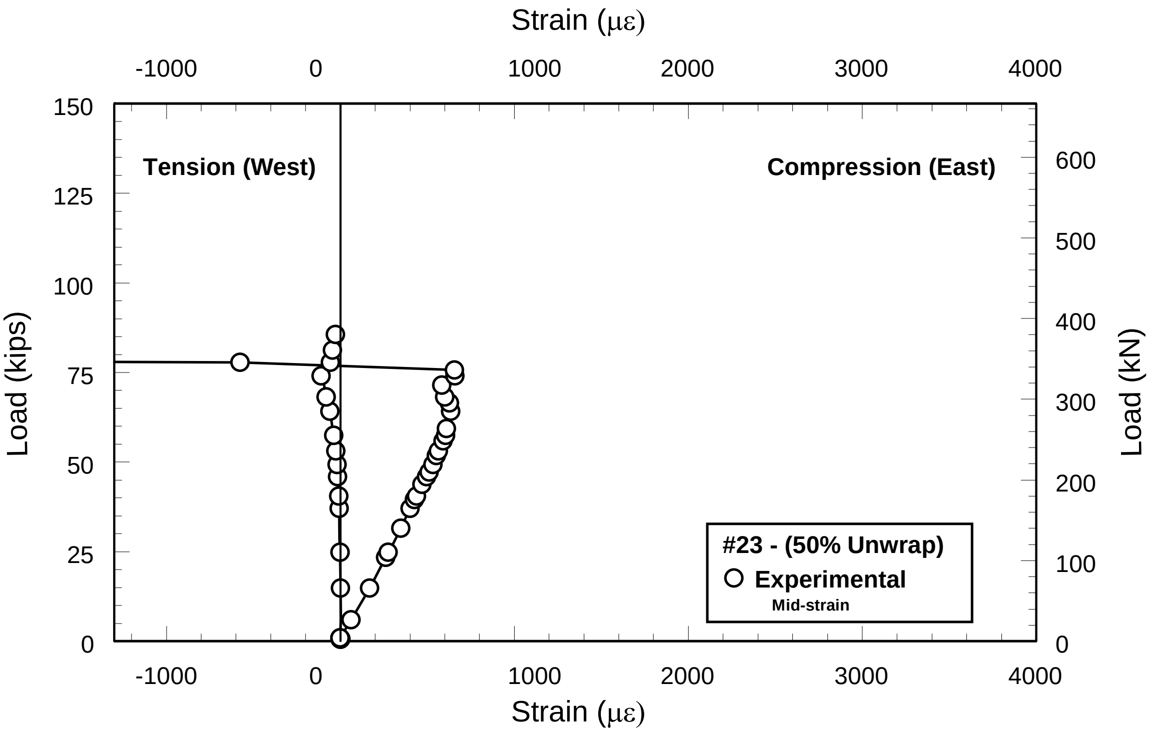


Figure B.12 Load v Strain Variation for Specimen 23 (50% Control).

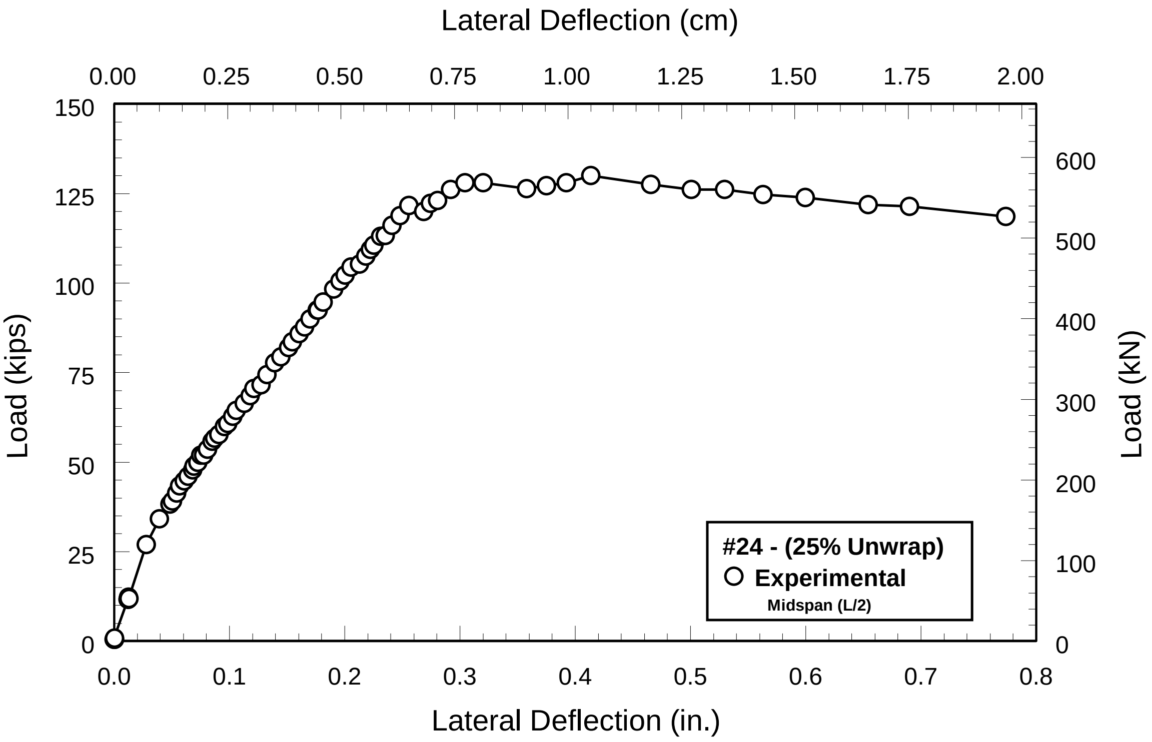


Figure B.13 Load v Deflection Plot for Specimen 24 (25% Control).

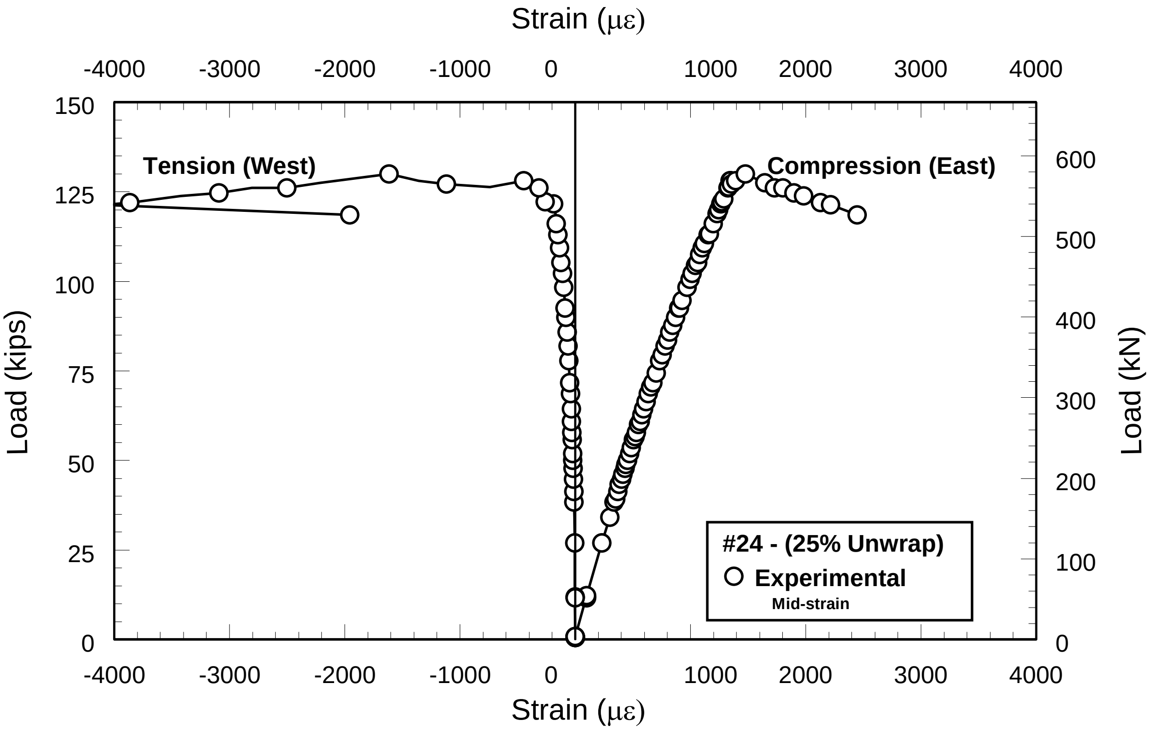


Figure B.14 Load v Strain Variation for Specimen 24 (25% Control).

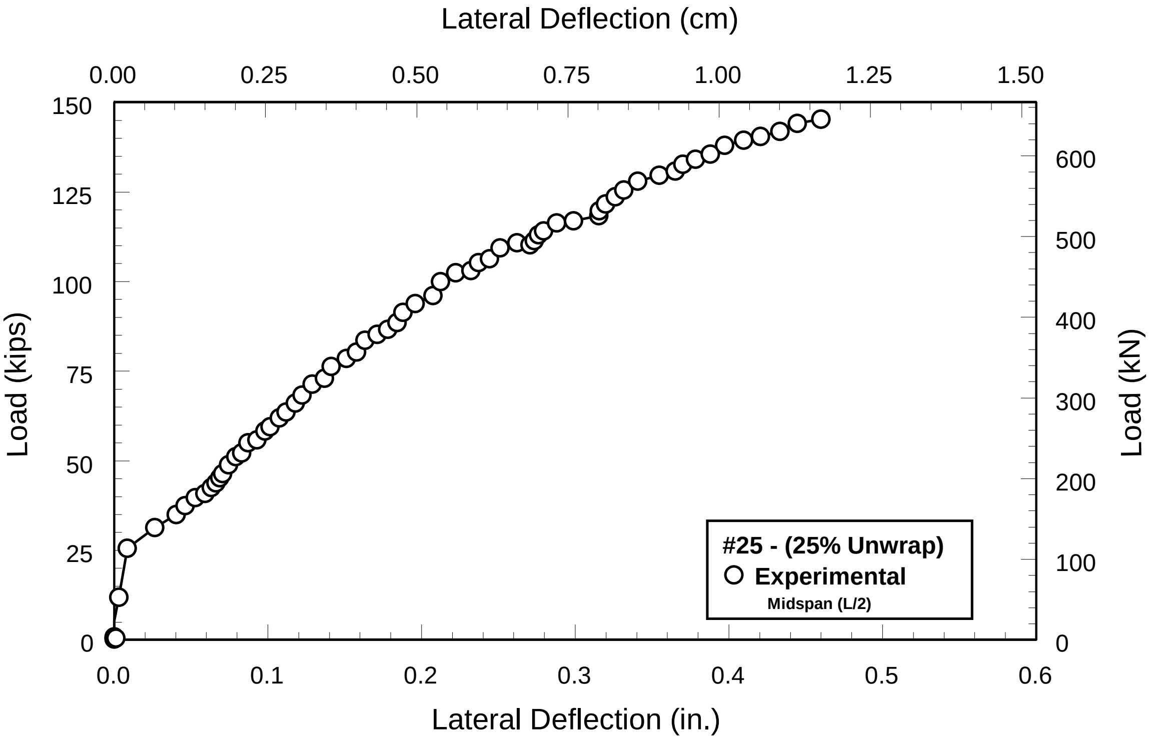


Figure B.15 Load v Deflection Plot for Specimen 25 (25% Control).

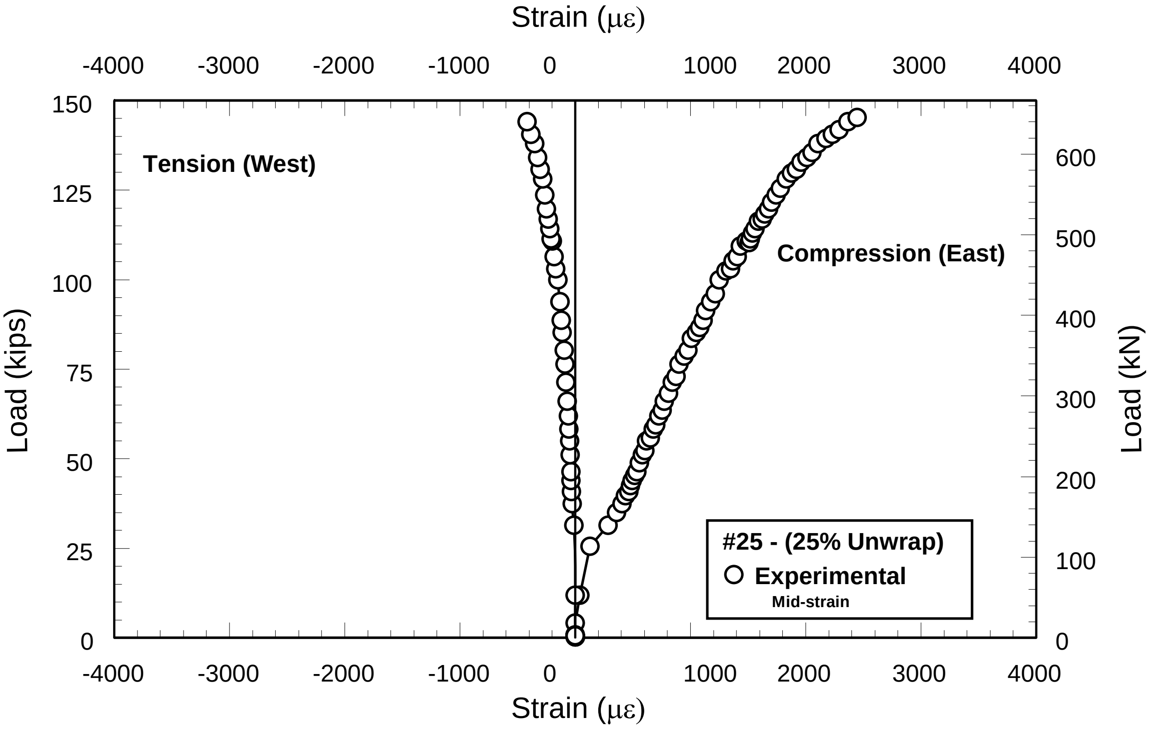


Figure B.16 Load v Strain Variation for Specimen 25 (25% Control).

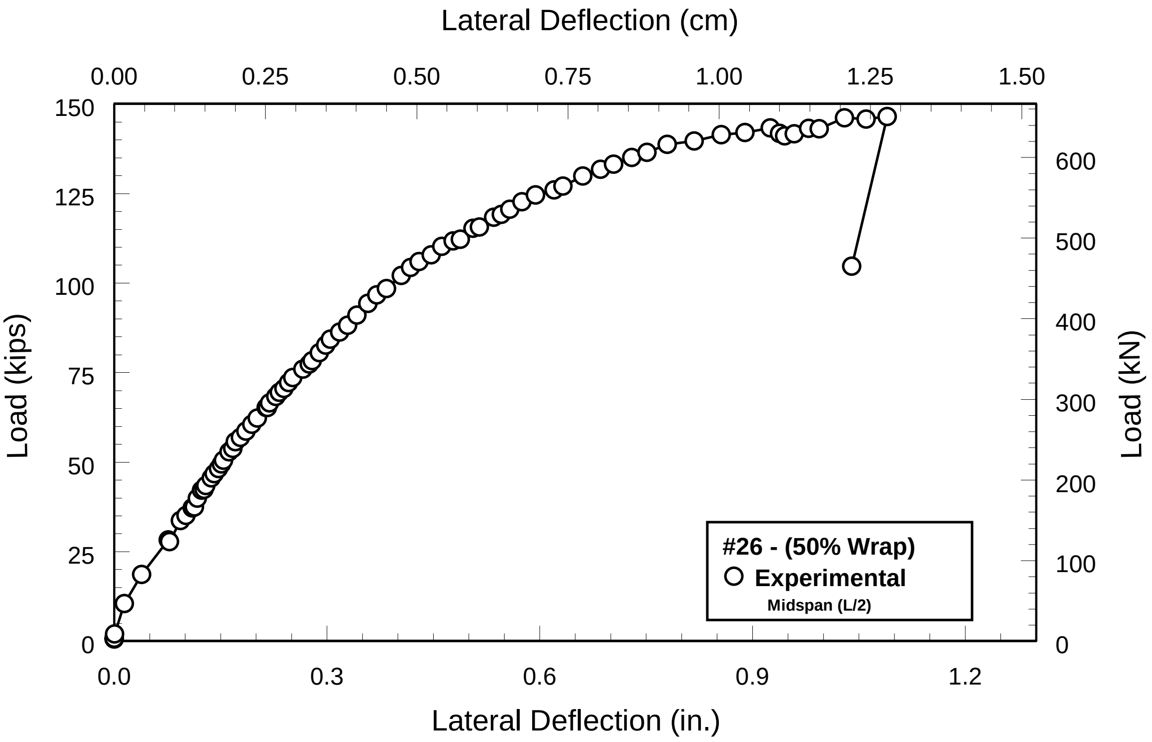


Figure B.17 Load v Deflection Plot for Specimen 26 (50% Wrap).

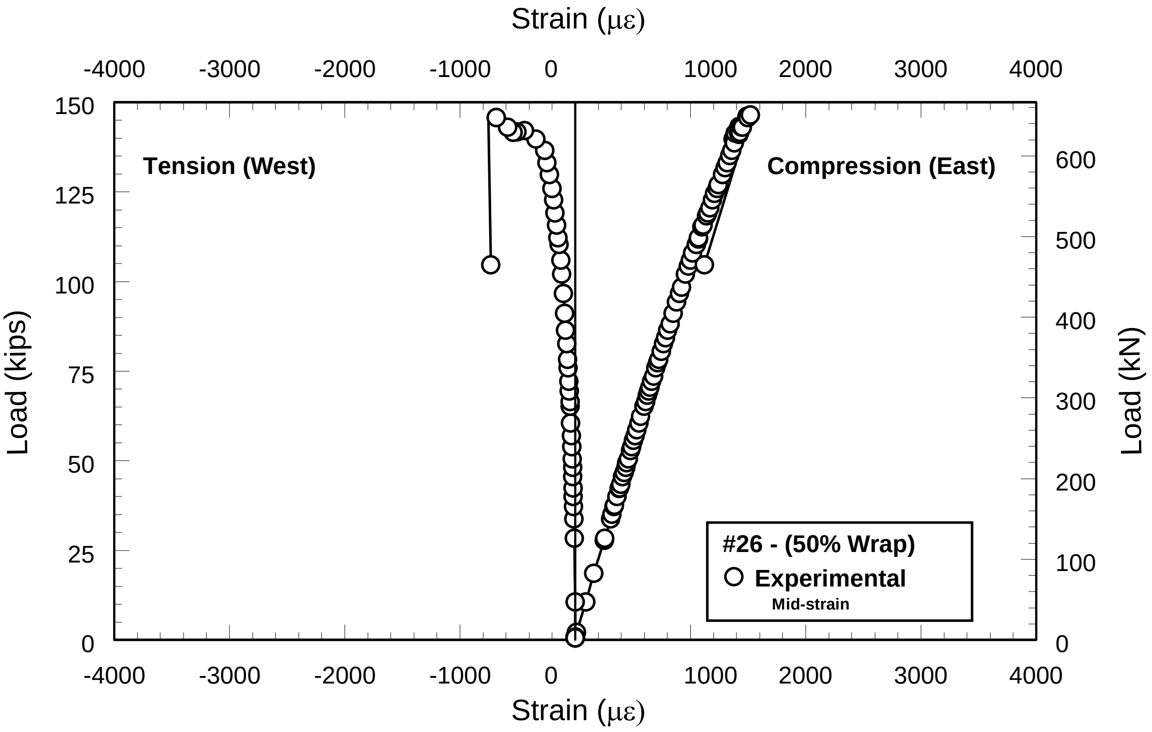


Figure B.18 Load v Strain Variation for Specimen 26 (50% Wrap).

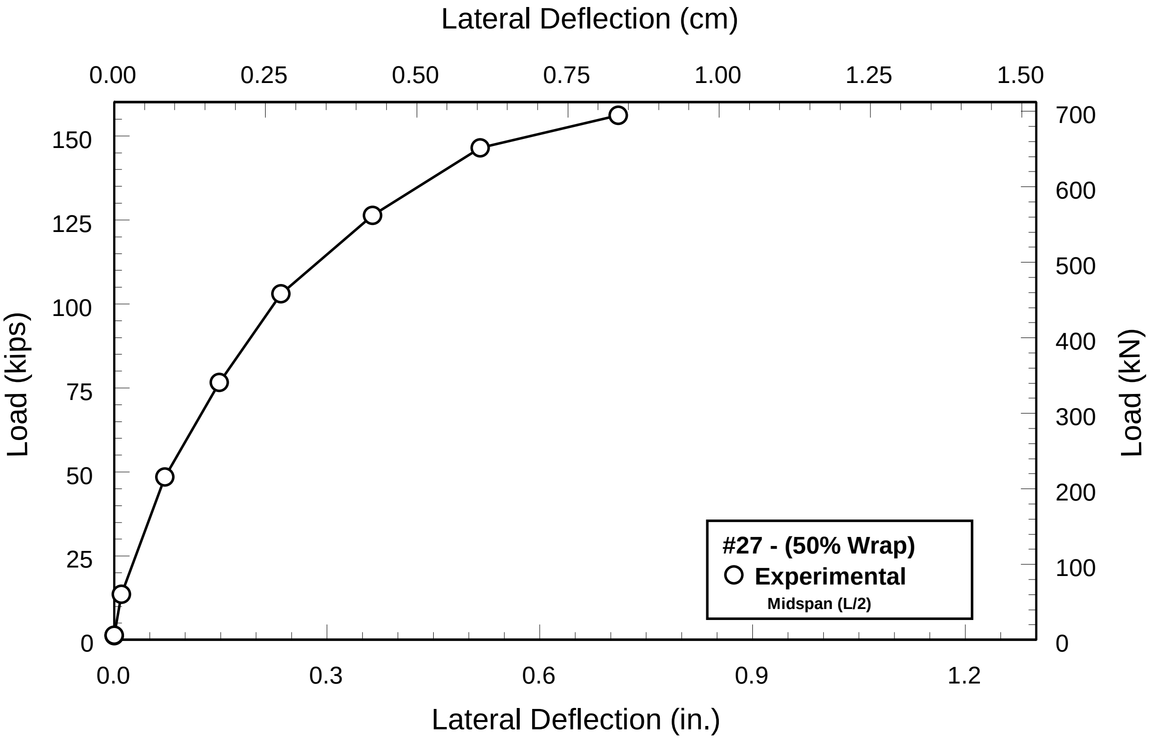


Figure B.19 Load v Deflection Plot for Specimen 27 (50% Wrap).

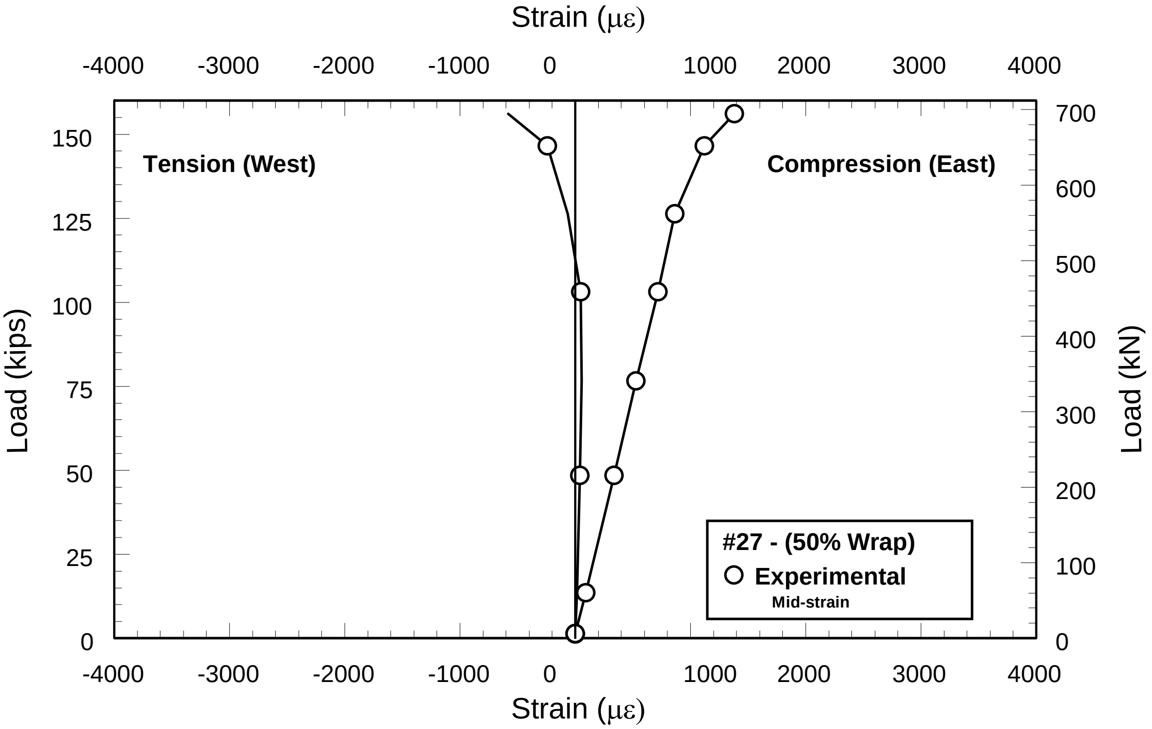


Figure B.20 Load v Strain Variation for Specimen 27 (50% Wrap).

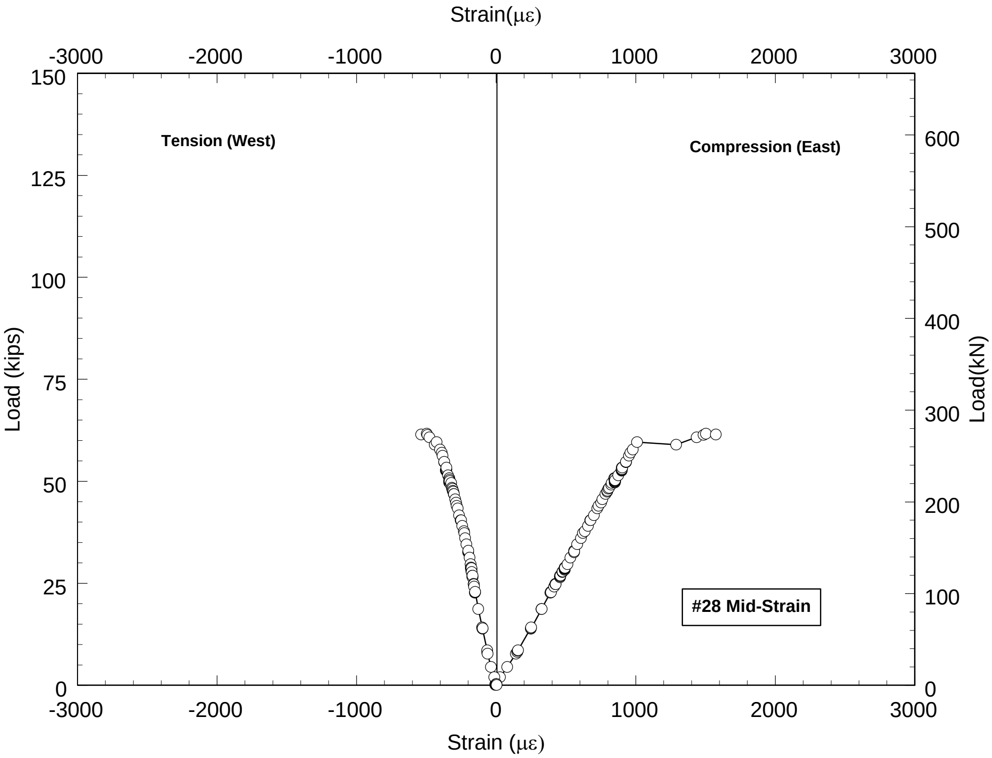


Figure B.21 Load vs Strain Variation for Specimen 28 (Control)

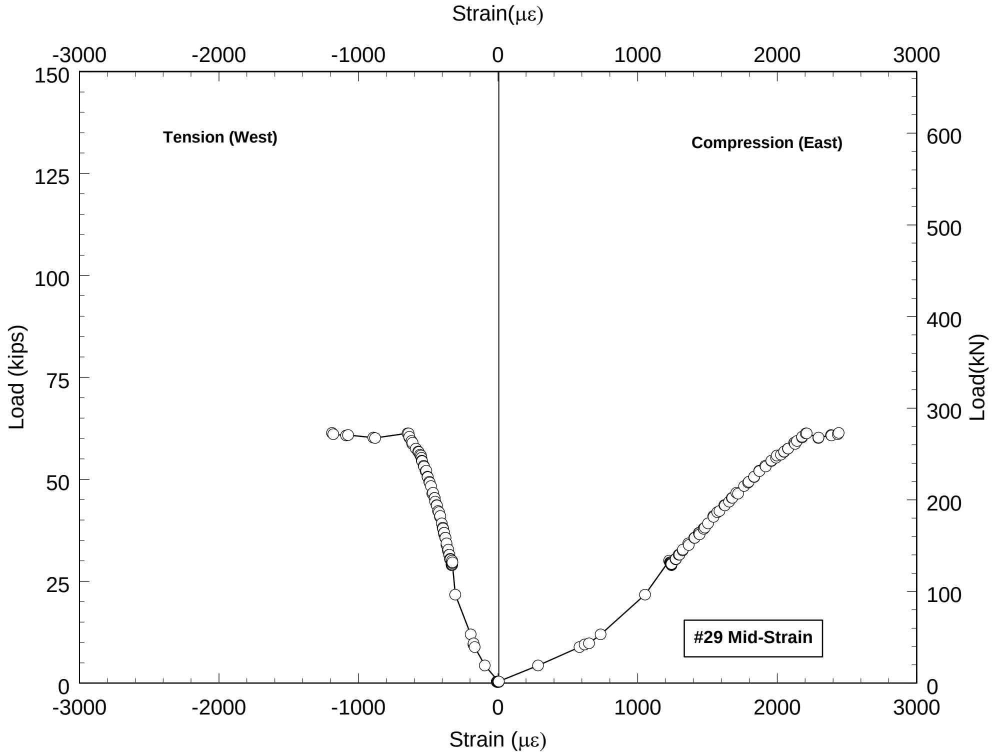


Figure B.22 Load vs Strain Variation for Specimen 29 (Control)

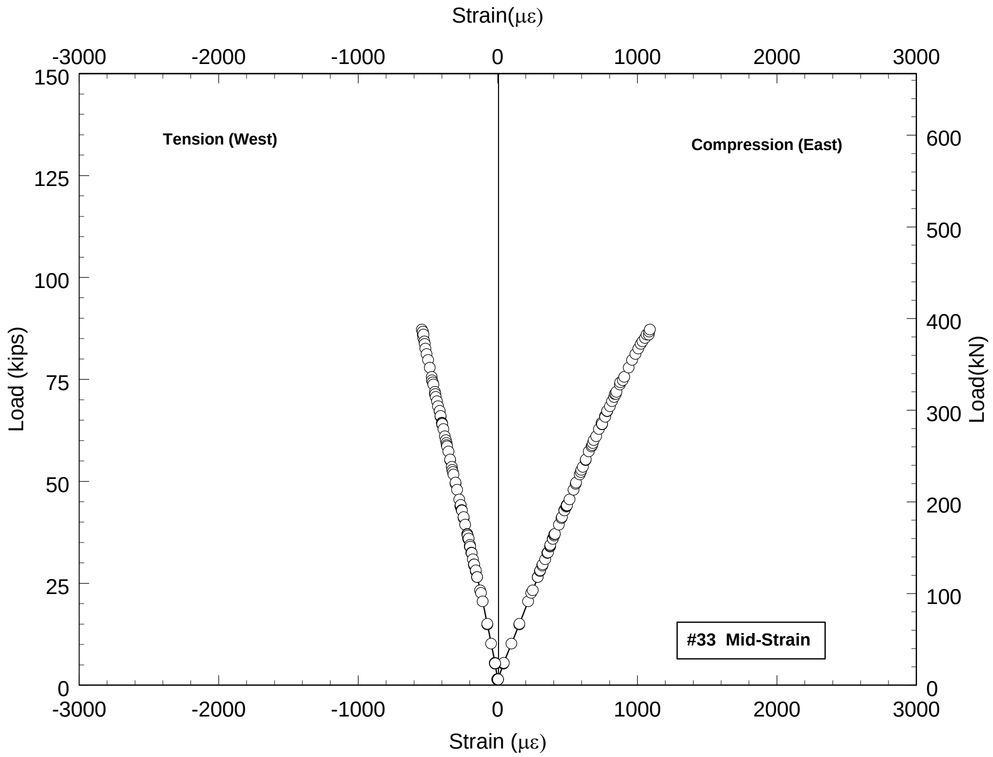


Figure B.23 Load vs Strain Variation for Specimen 30 (Full Repair/36in/2layer/Seal)

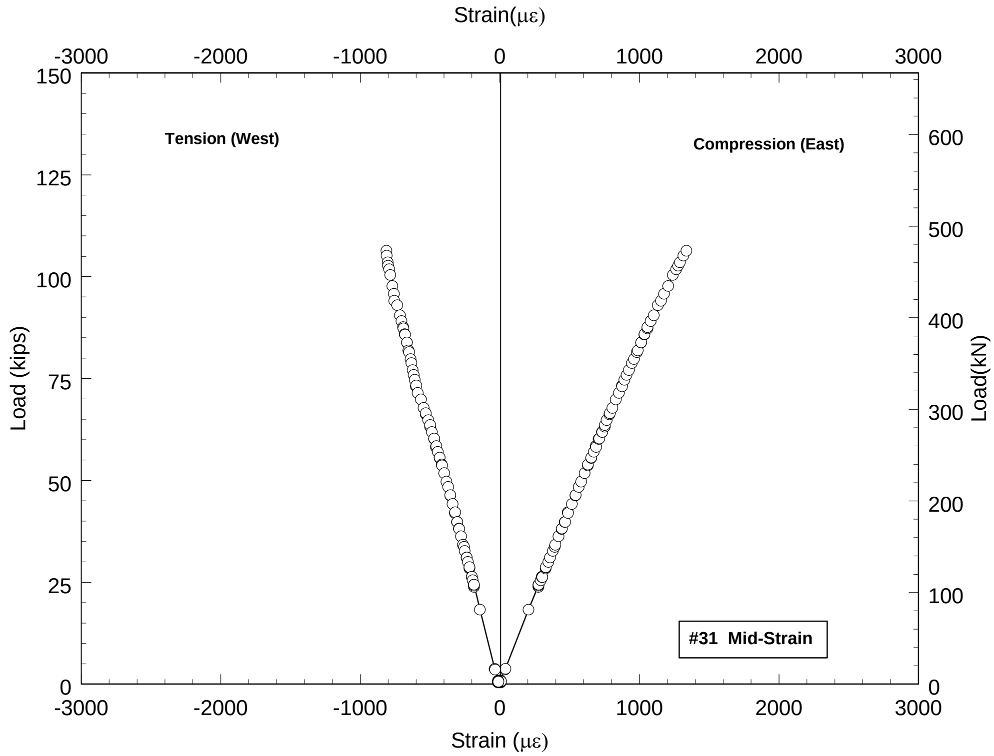


Figure B.24 Load vs Strain Variation for Specimen 31 (Full Repair/36in/2layer/Seal)

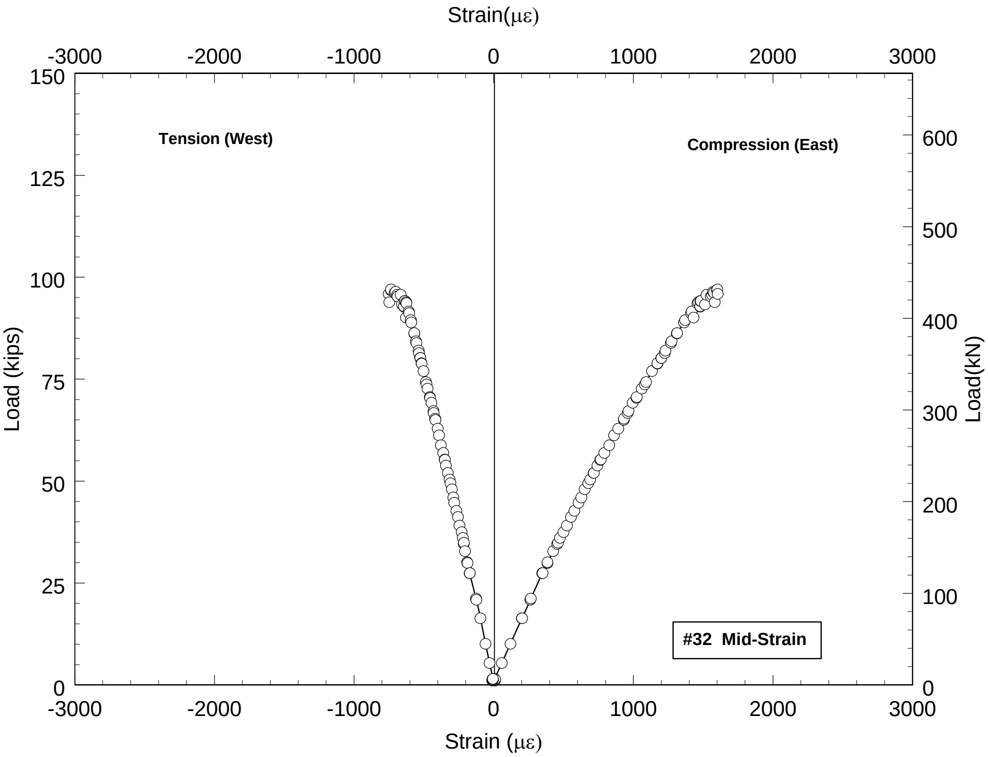


Figure B.25 Load vs Strain Variation for Specimen 32 (Minimal Repair/36in/2layer/Seal)

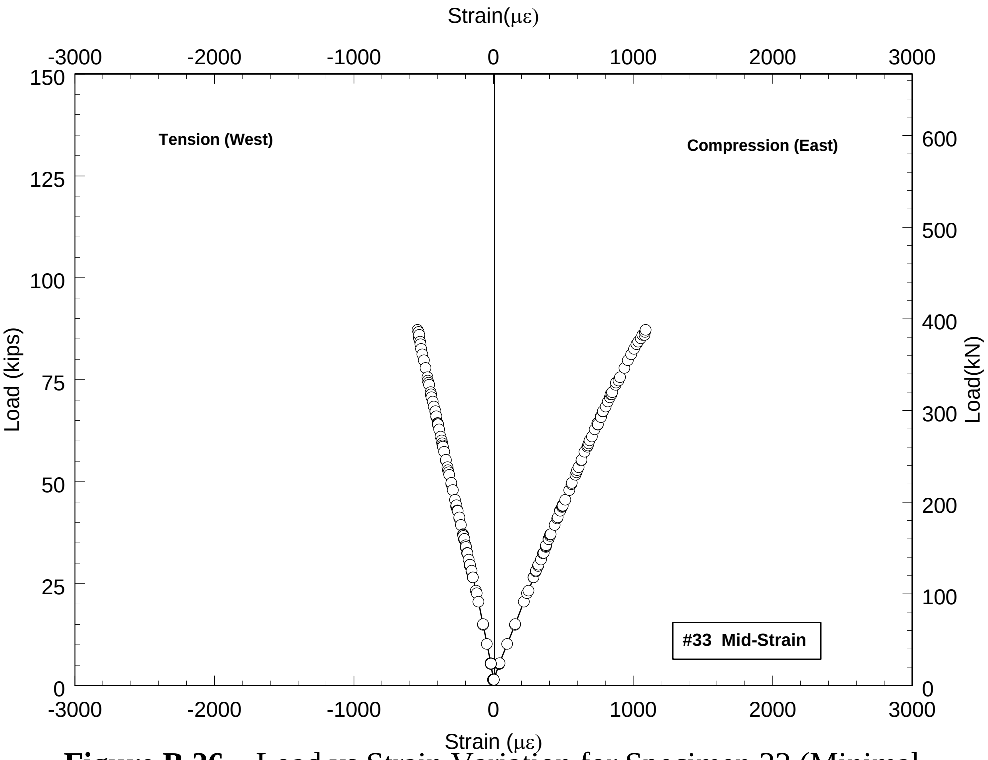


Figure B.26 Load vs Strain Variation for Specimen 33 (Minimal Repair/36in/2layer/Seal)

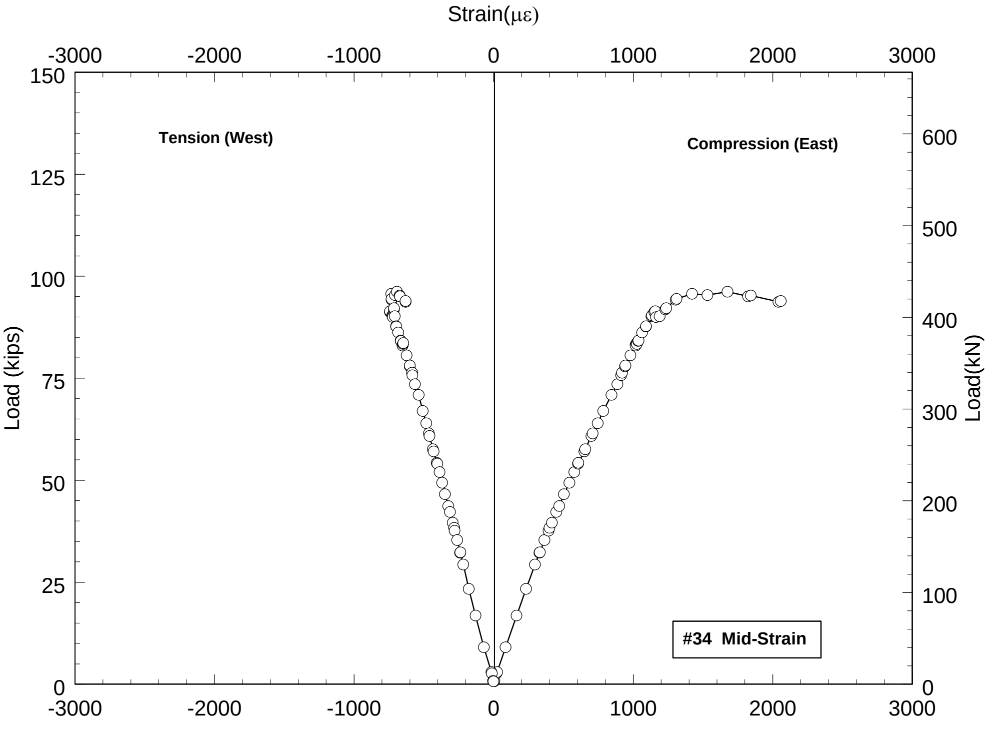


Figure B.27 Load vs Strain Variation for Specimen 34 (Minimal Repair/36in/2layer/Unsealed)

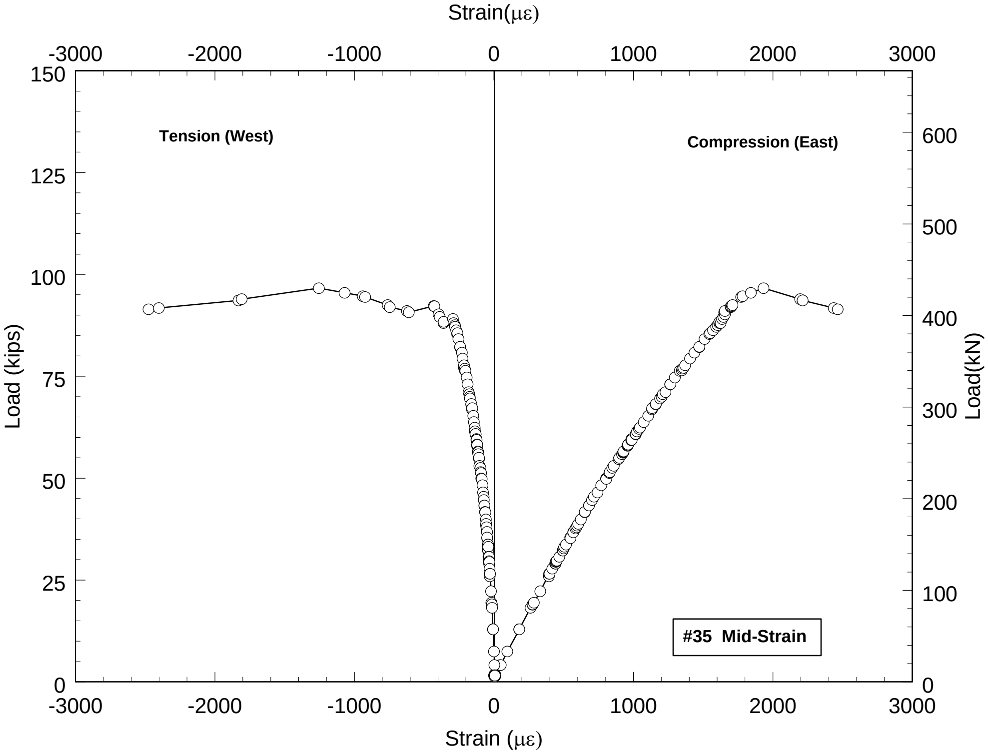


Figure B.28 Load vs Strain Variation for Specimen 35 (Minimal Repair/72in/2layer/Seal)

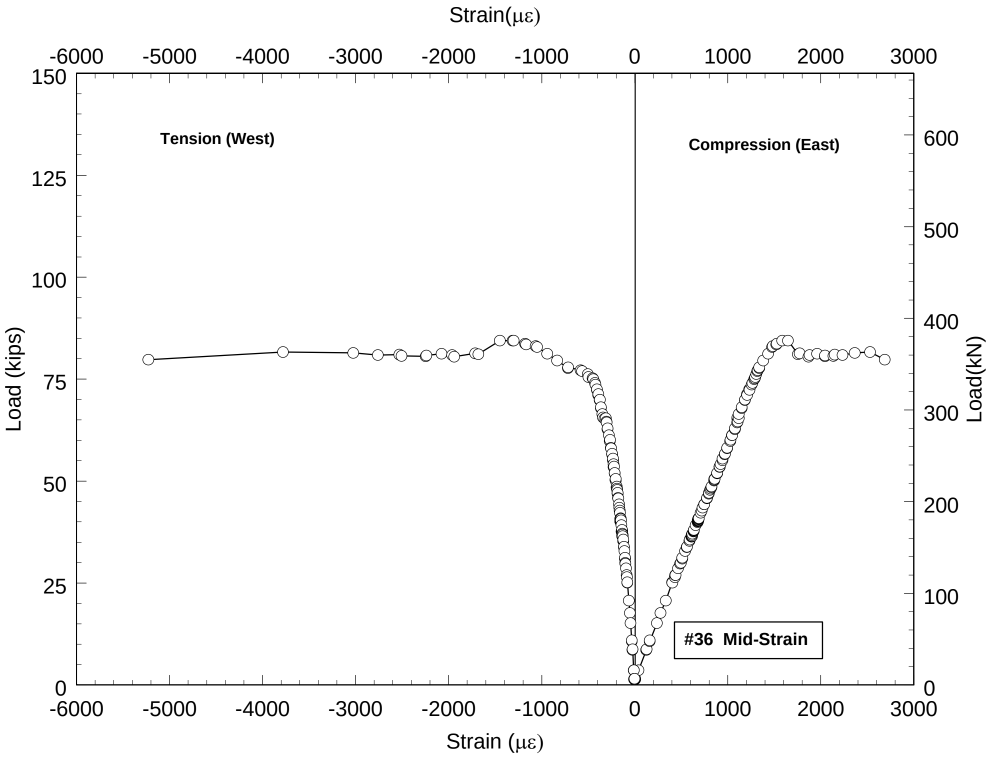


Figure B.29 Load vs Strain Variation for Specimen 36 (Minimal Repair/72in/2layer/Seal)

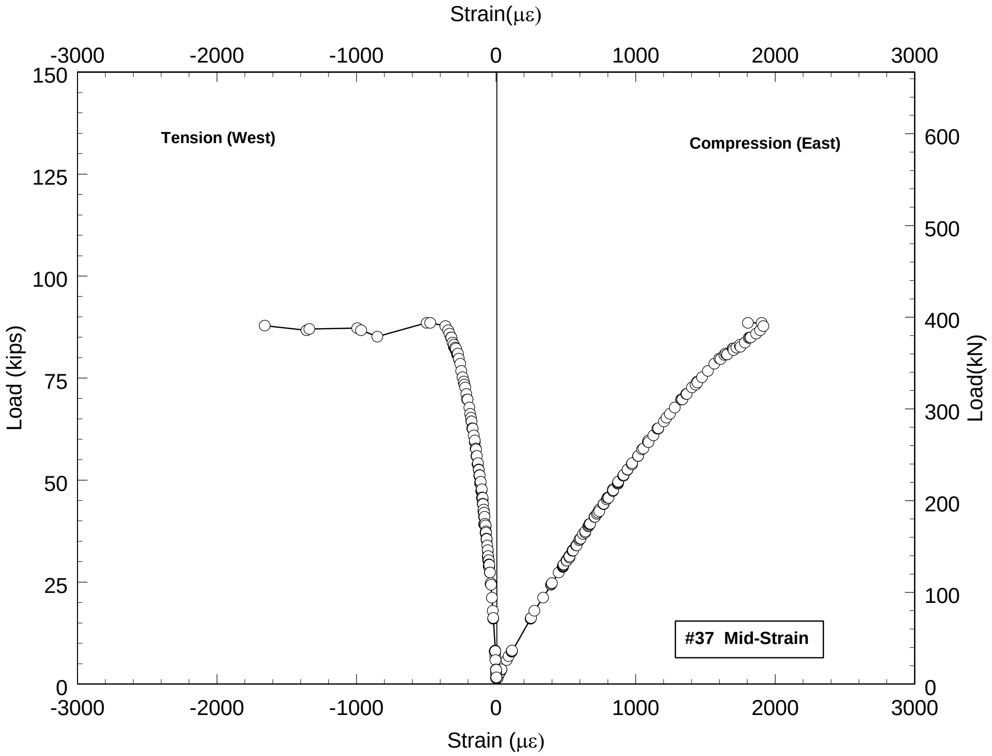


Figure B.30 Load vs Strain Variation for Specimen 37 (Minimal Repair/72in/2layer/Unsealed)

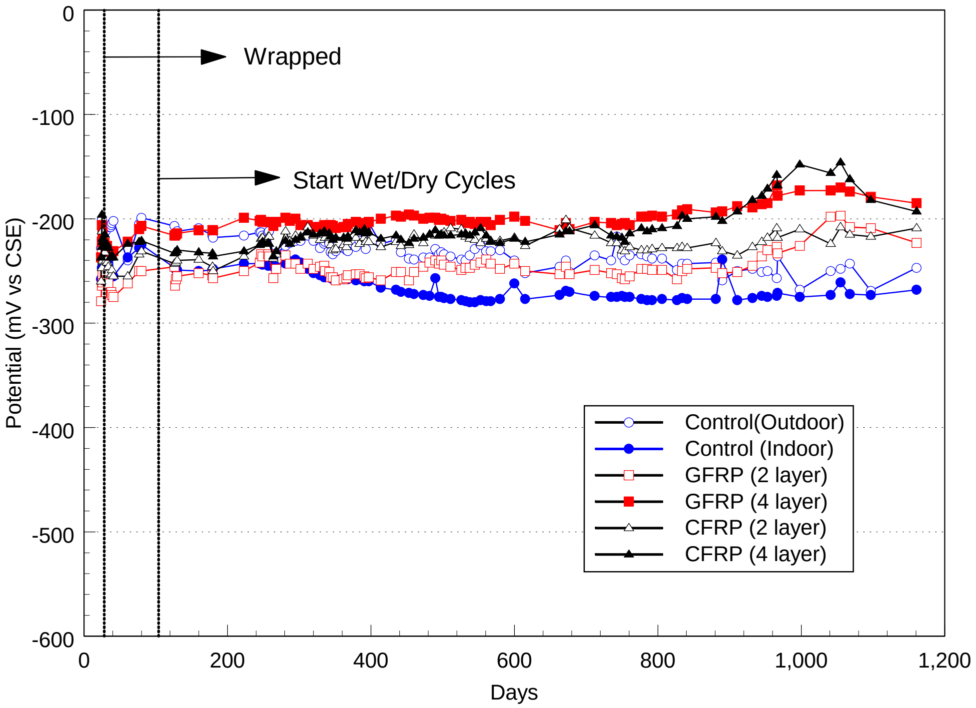


Figure B.31 Potential Variation at Top – A side

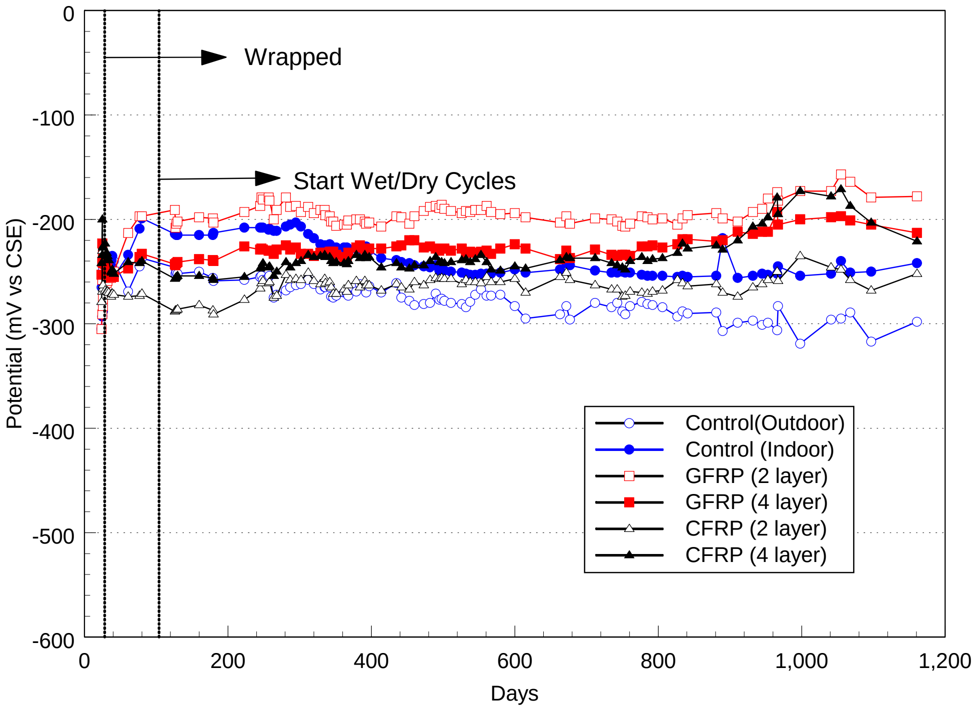


Figure B.32 Potential Variation at Top – C side

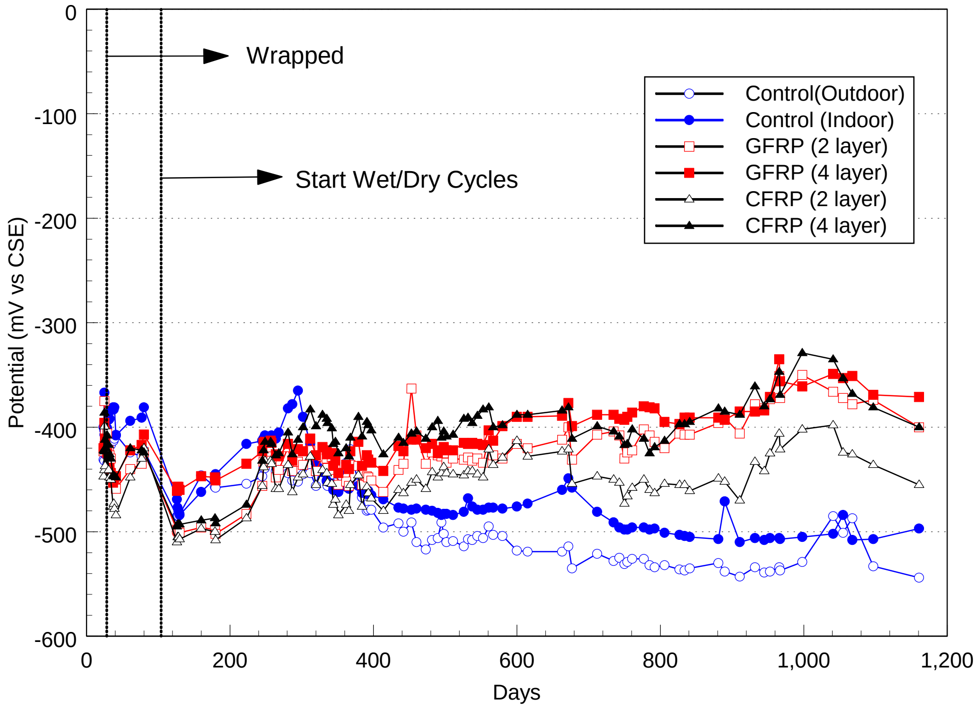


Figure B.33 Potential Variation at Middle – A side

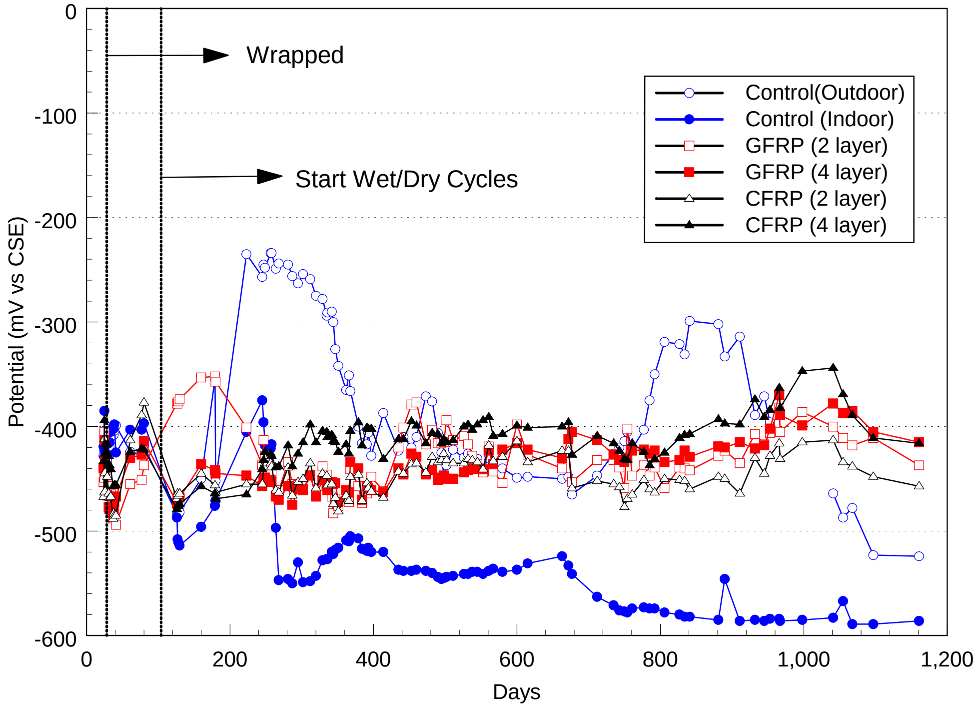


Figure B.34 Potential Variation at Middle – C side

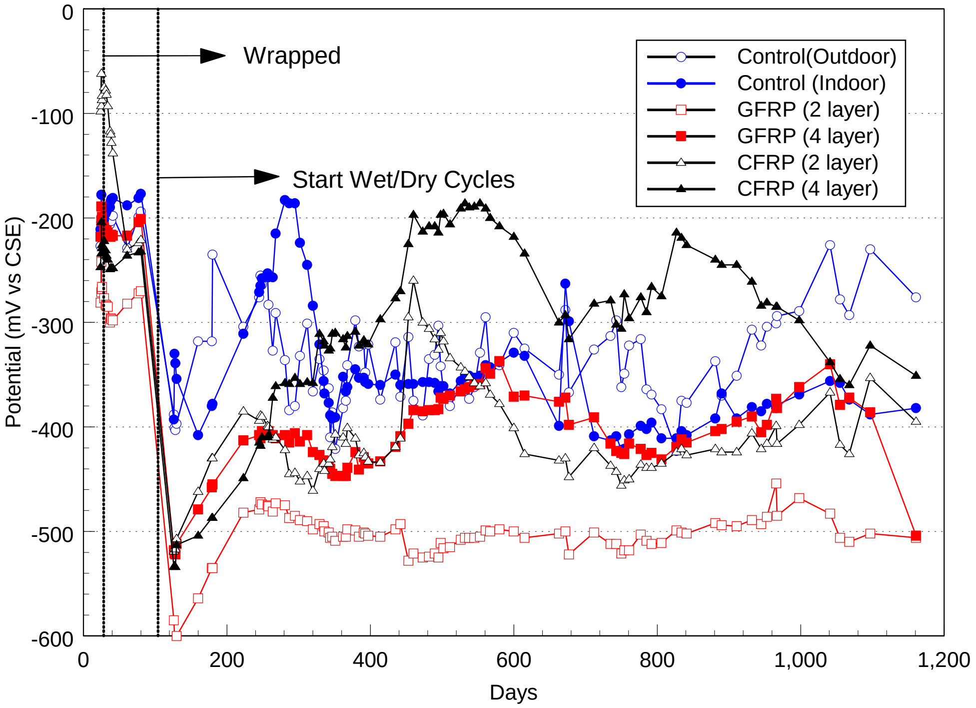


Figure B.35 Potential Variation at Bottom – A side

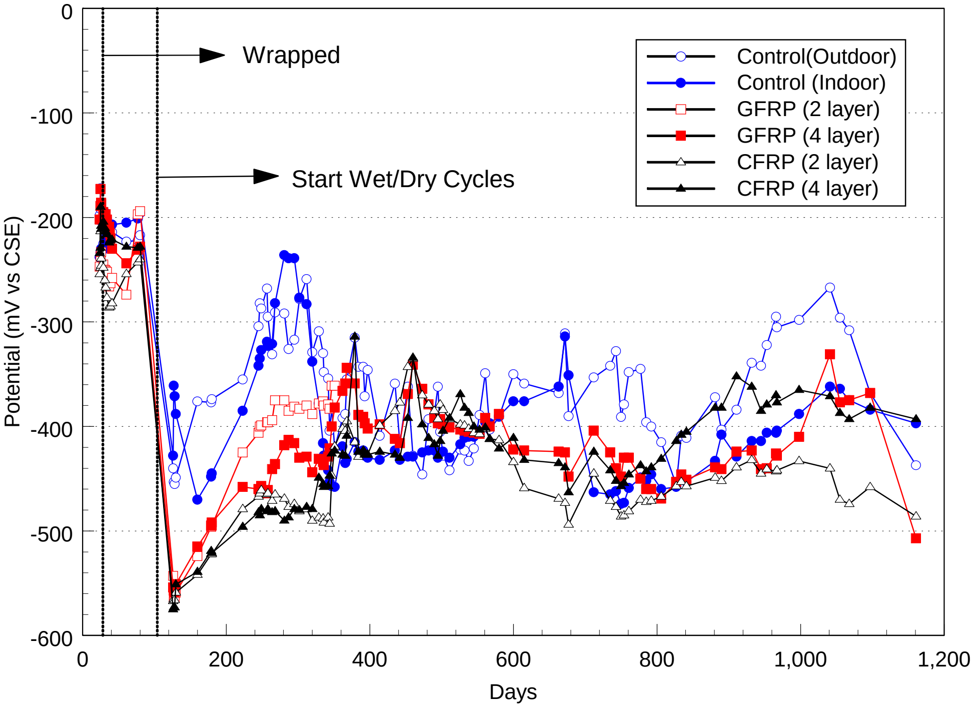


Figure B.36 Potential Variation at Bottom – C side

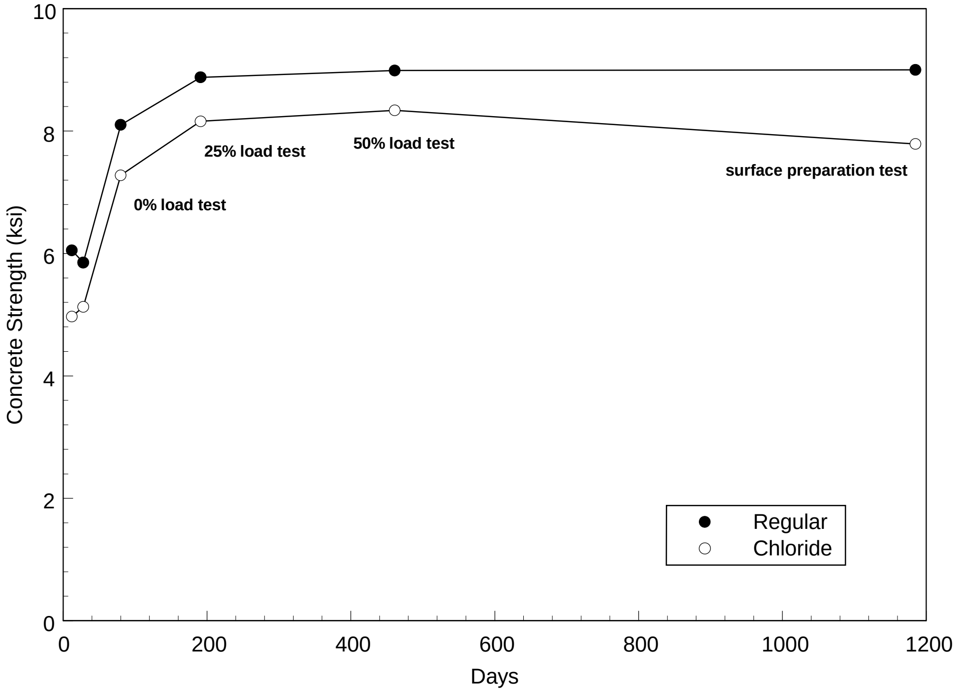


Figure B.37 Cylinder Test Results for Eccentric Load Test

Appendix C

Field Study Report from SDR Engineering

Final Report

CFRP REPAIR AND STRENGTHENING OF STRUCTURALLY DEFICIENT PILES

Prepared

By

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December 2003

CFRP REPAIR AND STRENGTHENING OF STRUCTURALLY DEFICIENT PILES

EXECUTIVE SUMMARY

The design theory along with the repair details are presented in Chapter 1. The damage in the concrete piles has been assessed and analysed. For each pile the damage assessment has been quantified in terms of percentage of loss with respect to initial capacity and categorized in terms of the range of loss as follows: No loss, 0-10% loss, and 10-20% loss.

The analysis of the residual strength of the different piles, given their respective losses, was then performed by developing P-M (axial compression-flexural moment) interaction diagrams. These diagrams are then compared to those corresponding to an original section with 0% loss. Strategies for repair are then elaborated and new P-M interaction diagrams are developed for strengthened piles and compared to target diagrams**.** The repair technique utilizing a CFRP wrap guarantees that the original capacities of the damaged piles are fully restored and the pile is capable of resisting any combination of axial and flexural loads that might be experienced during their remaining service life. A parametric study was carried out on the effect of prestress loss in tension face strands and in compression face strands on the M_n-P_n interaction diagram. Three levels of prestress loss namely 0%, 10% and 20% were considered. It was observed that the interaction diagrams were similar to those corresponding to uniform loss (i.e., distributed over all strands of the pile) and the differences were marginal and did not affect the design. Therefore, a uniform loss on all the strands is adopted for all the calculations presented in this report. In addition, the effect of expansion due to corrosion of steel during the remaining service life of the piles is also addressed using confinement provided by the CFRP wrap in the transverse direction.

The Allen Creek Bridge located in the city of Clearwater was identified as a demonstration site. Four piles were repaired utilizing a carbon wrap system as discussed in the analytical investigation.

A sample construction specifications for an FRP repair project and quality control guidelines are presented in Chapters 2 and 3, respectively.

Table of Contents

1 CFRP REPAIR AND STRENGTHENING

1.1 PROBLEM STATEMENT

This part of the study is aimed at assessing the feasibility of using CFRP as a valid method for strengthening damaged prestressed concrete piles. Also contained in this section are strategies for rehabilitation depending on the extent and the location of the structural damage. Shown below are sketches describing the bridge modeling details.

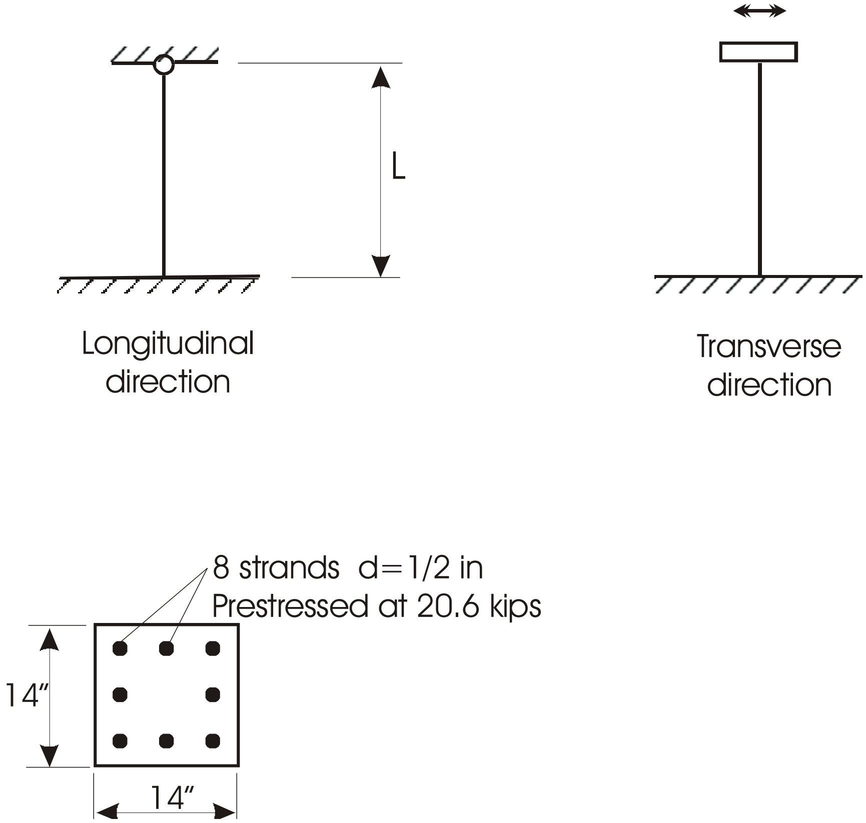


Figure 1.1 Modelling

1.2 OBJECTIVES

a) Evaluate the residual capacity of the prestressed piles for different loss levels due to corrosion of prestressing strands.

b) Evaluate the strengthening effect of repair techniques utilizing advanced CFRP materials.

c) Provide strategies for the structural rehabilitation of the piles.

1.3 BACKGROUND: CONFINEMENT MODELS FOR SHORT COLUMNS WRAPPED WITH FRP COMPOSITES

Confinement of concrete has been studied since the turn of the past century. Various constitutive models have been developed for constant active confining pressure as well as passive confinement with steel hoops, spirals or tubes. Researchers in the early 90's attempted to apply the same steel based models to fiber composites. The investigations, however, have shown that the behavior of concrete encased in fiber composites is not fully captured by such models[1-2]. This led to the development of new models for the confinement of circular columns by FRP jackets[3-7]. Only very recently have investigations on rectangular short columns wrapped with the FRP confinement been reported.

Picher et al.[8] examined the effect of the orientation of the confining fibers on the behavior of concrete cylinders, as well as of square and rectangular prisms, wrapped with carbon fiber reinforced polymer (CFRP) material. It was found that the wrapping could efficiently be applied to prismatic sections, provided the corners are rounded off. It was also found that an increase of the angle of wrapping orientation did not affect the ductility, but resulted in a decrease of the axial strength.

Hosotani et al.[9] studied confinement of concrete cylinders and square prisms by carbon fiber sheets (CFS) for seismic strengthening. The parameters considered in the tests were the shape of the specimen, and the content and the type (normal and high elastic modulus) of CFS. One significant outcome of the study was that as the CFS ratio was increased in the range of 0.05 to 0.15 % by volume, no increase was achieved in the peak axial stress of the concrete, f_{cc}, and in the corresponding axial strain, ε_{cc}, irrespective of the shape of the specimens. It was also found that at a CFS ratio greater than approximately 1%, the axial stress of concrete increased continuously until failure of the CFS.

Harries et al.[10] presented results of an experimental investigation on eight 1830 mm long circular and square reinforced concrete columns confined with external FRP jackets, under axial compression. It was found that externally applied FRP jackets can provide a confinement equivalent to that provided by closely spaced, well detailed conventional transverse steel reinforcement.

Wang and Restrepol[11] proposed analytical expressions based on Mander's model[16] to calculate the capacity of axially loaded reinforced concrete rectangular columns confined with internal steel hoops in addition to an externally applied composite jacket. The equations take into account the confinement effect due to both the steel and the FRP jacket. The predicted values of the ultimate strength of the confined concrete compared favorably to the results from experimental tests, carried out by the same authors, on reinforced concrete columns confined by a FRP jacket.

With regards to circular specimens, Samaan, Mirmiran and Shahawy[12] developed a bilinear model to predict the stress-strain response of FRP confined circular columns in both the axial and the lateral direction. The model is based on the correlation between the dilation rate of concrete and the hoop stiffness of the restraining jacket. Figure 1.2 shows the enhanced confinement due to varying CFRP wrap based on the model. Generally, no initial stresses are introduced in the jacket, therefore, CFRP jackets provide passive confinement. In this case, the confining pressure is engaged as a result of the lateral dilation of the axially loaded column[13-20]. The linear-elastic behavior up to rupture of CFRP materials results in an increasing level of confinement through the entire load history. Tests have shown that confinement by FRP wrap can significantly enhance strength and ductility of the concrete core of columns[20-22].

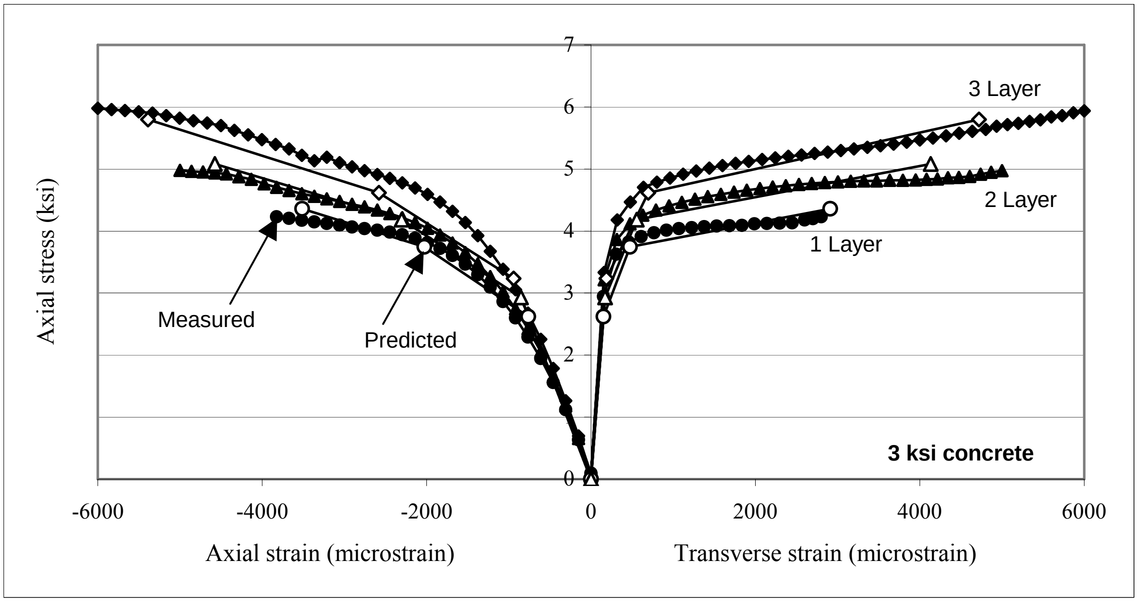


Figure 1.2 Measured and Predicted Stress-Strain Response for 3 Ksi Concrete

Uniaxial compression tests showed that:

1. Ultimate axial strength of concrete could be increased by 2-3 times that of unconfined concrete[21];
2. Ultimate strains could be increased by 10-15 times that of unconfined concrete[21]; and
3. FRP-confined concrete has a unique dilation characteristic that is markedly different from steel-confined concrete[22].

Research also showed that both axial and lateral responses are bi-linear with a transition zone at or near the peak strength of unconfined concrete core. The volumetric response experiences a similar transition toward volume expansion. However, as soon as the jacket takes over, volumetric response undergoes another transition, which reverses the dilation trend and results in volume compaction. Note that this behaviour is markedly different from plain concrete and steel-confined concrete, both of which fail by excessive unstable volume expansion[22].

Figure 1.3 illustrates the differences between steel and FRP confined concrete. Figure 1.3 shows the axial stress-strain curves for the two confinement mechanisms. The axial stresses are normalized with respect to the unconfined strength of their respective concrete cores. The figure also shows a typical response of plain (unconfined) concrete for comparison. It can be seen from the figure that the steel-confined concrete experiences only a mild softening before it reaches maximum strength of f'_{cc} after which it follows a gradual post peak descending branch, and the ultimate (failure) strength (f'_{cu}) is lower than the peak strength (f'_{cc}).

The peak strength of confined concrete (f'_{cc}) occurs as (or shortly after) the steel jacket yields. On the other hand, the FRP-confined concrete displays a distinct bilinear response with a sharp softening and a transition zone at the level of its unconfined strength (f'_c), after which the tangent stiffness stabilizes at a constant value until reaching the ultimate strength (f'_{cu}). As for the ultimate strain and the corresponding ductility ratios, it is obvious from examining the areas under the stress-strain curves that the steel-confined concrete provides a larger energy absorption capacity than its FRP-confined counterpart.

It is also noteworthy that whereas the two confinement mechanisms provide the same level of confinement pressure as a ratio of their respective concrete strengths, the degree of lateral restraint is not the same. The FRP tube applies a continuously increasing pressure on the concrete core until the tube reaches its first-ply failure, whereas the confining pressure of steel tube remains constant after the tube yields under hoop tension.

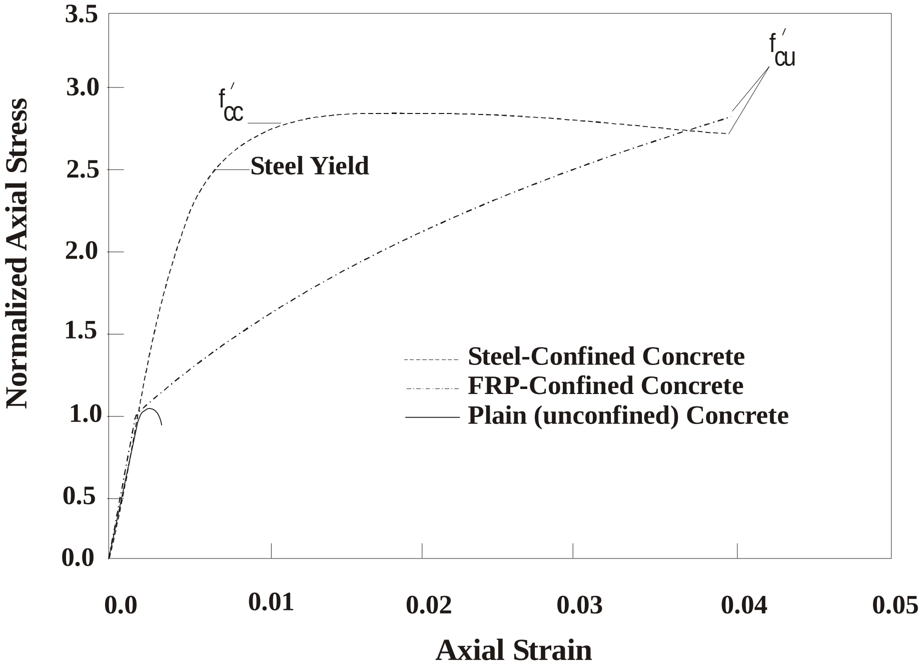


Figure 1.3 Stress-Strain Response of FRP-Confined Concrete Versus Steel-Confined Concrete

Recognizing the significant differences between circular and rectangular cross sections, Chaallal, O., Shahawy, M. and Hassan[23,24], introduced a confinement model for axially load short rectangular columns strengthened with FRP wrapping. The details of this model are presented later in this report and used to evaluate the strength of the piles.

1.4 PARAMETERS OF THE INVESTIGATION

1. Loss of strength of prestressing strands (0%, 10% and 10 to 20 % loss) - Uniform loss for all strands.
2. Loss of strength of prestressing strands (0%, 10% and 10 to 20 % loss) - Loss on one face (tension or compression) only, the remaining strands assumed intact.
3. Effect of concrete strength, f'_c (4 ksi and 6 ksi).

1.5 ANALYTICAL ASSUMPTIONS

1. The distance from the extreme compression or tension fiber to the centroid of the prestressing strands is equal to 3 inches (that is $d'_p = 3$ in.).

2. The prestressing strands are likely to be stress relieved. The effective prestress is 135 ksi, i.e., the force is 20.6 kips.

3. Only the CFRP material on the tension face of the column is considered.

4. The CFRP material used is a bi-directional fabric wrap from AMACO. Its properties are presented in Table 1.1 (A). The epoxy is a specially formulated resin marketed as the MAS2000 Repair System. Its basic properties are presented in Table 1.1 (B).

Table 1.1(A) Material Properties for Carbon Wraps

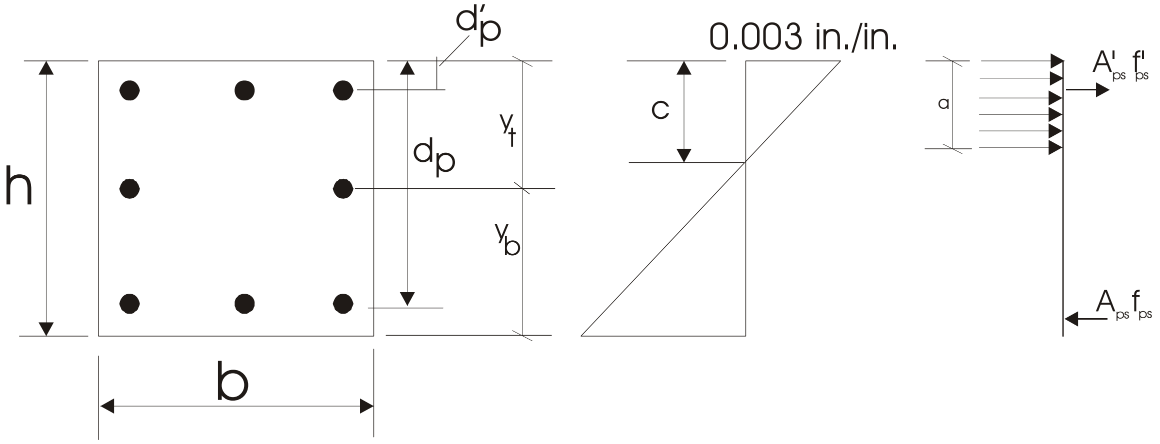
DESCRIPTION	MANUFACTURER'S DATA[(1)]
Tensile Strength	530 ksi (3.65 GPa)
Tensile Modulus of Elasticity	33500 ksi (231 GPa)
Ultimate Tensile Elongation	1.4%
Filament Diameter	7 µm
Number of Filaments per Yarn	12000
Number of yarns (X/Y) per Inch	6.7 / 6.7
Density	12 oz./yd^2
Equivalent Thickness per Layer	0.0048 in.
Section Area per Yarn	70x10^{-5} in.^{2}/yarn
Note: (1) Reported for the carbon fabric only	

Table 1.1(B) Material Properties for Epoxy

DESCRIPTION		VALUES	TEST METHOD
Mix Ratio	By Weight By Volume	100 : 27 3 to 1	Manufacturer
Color		Light Amber	Visual
Mixed Viscosity, centipoise, @ 77°F		900-950 cps	ASTM D2393
Pot Life, 4 fluid ounces mass		1 hour	ASTM D2471
Cured Hardness, shore D		88D	ASTM D2240
Specific Gravity, gms./cc		1.11	ASTM D1475
Density	lb./cu.in. lb./gallon	0.0401 9.26	ASTM D792
Specific Volume, cu. in./lb.		25	ASTM D792
Tensile Strength, psi		45,170 psi	ASTM D638
Elongation(1)		1.96%	ASTM D638
Tensile Modulus, psi		2.62×10^6 psi	ASTM D638
Flexural Strength, psi		62,285 psi	ASTM D790
Flexural Modulus, psi		2.56×10^6 psi	ASTM D790
Glass Transition Temperature (Tg)		196°F	TMA
Coefficient of Thermal Expansion Range: 100°F - 150°F		4.3×10^{-5} in./in./°F	ASTM D696

1.6 EFFECT OF PRESTRESS LOSS (DUE TO CORROSION) ON CAPACITY

1.6.1 Equations for $P_n - M_n$ Interaction Diagram



A- General Case: Prestressing Strands plus Steel Reinforcement plus CFRP

Applying strain compatibility and equilibrium equations:

$$\beta_1 = 0.85 - 0.05\,\frac{f'_c - 4000}{1000} \quad (1)$$

$$\varepsilon'_s = (0.003/c)\,(c - d') \quad (2)$$

$$\varepsilon_s = (0.003/c)\,(d - c) \quad (3)$$

$$\varepsilon_f = (0.003/c)\,(h - c)\ [\varepsilon_f = 0 \text{ if } (0.003/c)(h - c) > 0.012 \text{ or } (0.003/c)(h - c) < 0] \quad (4)$$

ε_f is limited to 0.85 ε_{fu} = 0.85 x 0.014 = 0.012.

$$\varepsilon'_{ps} = f_{se}/E_{ps} - (0.003/c)\,(c - d'_p) \le 0.035 \quad (5)$$
(PCI Design Handbook, 5th Edition, p. 4-73)

$$\varepsilon_{ps} = f_{se}/E_{ps} + (0.003/c)\,(d_p - c) \le 0.035 \quad (6)$$
(PCI Design Handbook, 5th Edition, p. 4-73)

$$f'_s = \varepsilon'_s\,E_s \le f_y\ ; \qquad f_s = \varepsilon_s\,E_s \le f_y \quad (7)$$

$$f'_{ps} = \varepsilon'_{ps}\,E_{ps} \le f_{py}\ ; \quad f_{ps} = \varepsilon_{ps}\,E_{ps} \le f_{py} \quad (8)$$

$$f_f = \varepsilon_f\,E_f \quad (9)$$

$$\begin{aligned} P_n &= (A_{comp} - A'_s - A'_{ps})\,(0.85\,f'_c) \\ &\quad + A'_s\,f'_s - A_s\,f_s - A'_{ps}\,f'_{ps} - A_{ps}\,f_{ps} - A_f\,f_f \end{aligned} \quad (10A)$$

$$\begin{aligned} M_n = \quad & P_n e \;=\; (A_{comp} - A'_s - A'_{ps})\,(y_t - y')\,(0.85\,f'_c) \\ & + A'_s\, f'_s\,(y_t - d') + A_s\, f_s\,(d - y_t) \\ & - A'_{ps}\, f'_{ps}(y_t - d'_p) + A_{ps}\, f_{ps}\,(d_P - y_t) + A_f\, f_f\,(h - y_t) \end{aligned} \tag{11A}$$

B- Case of Prestressing Strands plus CFRP

Applying strain compatibility and equilibrium equations:

$$\beta_1 = 0.85 - 0.05\,\frac{f'_c - 4000}{1000} \tag{1}$$

$$\varepsilon_f = (0.003/c)\,(h - c)\ [\varepsilon_f = 0 \text{ if } (0.003/c)(h - c) > 0.012 \text{ or } (0.003/c)(h - c) < 0] \tag{4}$$

ε_f is limited to $C_E\ \varepsilon_{fu} = 0.85 \times 0.014 = 0.012$. Note that C_E is an environmental factor[25] equal to 0.85 for exterior exposure and 0.95 for interior exposure.

$$\varepsilon'_{ps} = f_{se}/E_{ps} - (0.003/c)\,(c - d'_p) \le 0.035 \text{ (PCI Design Handbook, 5th Edition, p. 4-73)} \tag{5}$$

$$\varepsilon_{ps} = f_{se}/E_{ps} + (0.003/c)\,(d_p - c) \le 0.035 \text{ (PCI Design Handbook, 5th Edition, p. 4-73)} \tag{6}$$

$$f'_{ps} = \varepsilon'_{ps}\, E_{ps} \le f_{py}\ ; \quad f_{ps} = \varepsilon_{ps}\, E_{ps} \le f_{py} \tag{8}$$

$$f_f = \varepsilon_f\, E_f \tag{9}$$

$$P_n = (A_{comp} - A'_{ps})\,(0.85\,f'_c) - A'_{ps}\, f'_{ps} - A_{ps}\, f_{ps} - A_f\, f_f \tag{10B}$$

$$\begin{aligned} M_n = \quad & P_n e \;=\; (A_{comp} - A'_{ps})\,(y_t - y')\,(0.85\,f'_c) \\ & - A'_{ps}\, f'_{ps}(y_t - d'_p) + A_{ps}\, f_{ps}\,(d_P - y_t) + A_f\, f_f\,(h - y_t) \end{aligned} \tag{11B}$$

Where:

a = depth of concrete in compression

A'_s = area of compression steel reinforcement

A_s = area of tension steel reinforcement

A'_{ps} = cross section area of prestressing strands at compression face

A_{ps} = cross section area of prestressing strands at tension face

A_f = cross section area of FRP at tension face = width x yarn thickness x number of layers

A_{comp} = cross section area of concrete zone in compression = a x b

b = width of the concrete cross section

c = depth of the neutral axis

d = effective depth, i.e., distance from extreme compression fiber to centroid of longitudinal steel in tension

d'= depth from extreme compression fiber to centroid of longitudinal steel in compression

d_p = distance from extreme compression fiber to centroid of prestressing strands in tension zone

d'_p = distance from extreme compression fiber to centroid of prestressing strands in compression zone

E_{ps} = modulus of elasticity of prestressing strands

E_s = modulus of elasticity of steel

E_f = modulus of elasticity of FRP

f_y = yield stress of steel

f'_{py} = yield stress of prestressing strands

f'_s =stress in non-prestressed steel in compression

f_s = stress of non-prestressed steel in tension

f'_{ps} = stress of prestressing strands in compression

f_{ps} = stress of prestressing strands in tension

f_f = stress of FRP at tension face

f'_c = specified compression strength of concrete

h = height of the concrete cross section

M_n = nominal moment resistance of column

P_n = nominal compression resistance of column

y' = distance from extreme compression fiber to center of gravity of concrete zone in compression, A_{com}

y_t = distance from center of gravity of gross section to extreme fiber in tension, or

y_t = distance from center of gravity of gross section to extreme fiber in compression

β_1 = ratio of depth of rectangular compression block to depth of the neutral axis

ε'_s = strain of non-prestressed steel in compression

ε_s = strain of non-prestressed steel in tension

ε'_{ps} = strain of prestressing strands in compression

ε_{ps} = strain of prestressing strands in tension

ε_f = strain of FRP in tension face

1.6.2 Procedure for P-M Interaction Diagram Generation

1. Select compression zone depth, a, and compute from Eq. (1)
2. Compute neutral axis depth, $c = a/\beta_1$
3. Compute strains: ε'_s, ε_s, ε'_{ps}, ε_{ps}, and ε_f from Eqs. (2) to (6)
4. Compute stresses: f'_s, f_s, f'_{ps}, f_{ps}, and f_f from Eqs. (7) to (9)
5. Compute P_n and M_n from Eqs. (10) and (11)
6. This gives a point in the P-M Interaction diagram
7. Go to 1 with a new value of 'a' up to a = h.

1.6.3 Sample Example

Following is an example of a calculation of one point of the P-M diagram:

- Data:

f'_c= 4000 psi, b=14in., h=14 in., A_g = 196 in.^{2}, A_s = 0, A'_s=0, A_{ps}= 0.459 in.^{2}, A'_{ps}= 0.459 in.^{2}, d_p= 11 in., d'_p= 3 in., f_{pu}=270 000 psi, f_{py} = 229130 psi, E_{ps}=2.75 E+7 psi, Number of CFRP layers = 2, t (1 layer)=0.0048 in., E_f= 3.35 E+7 psi, Effective prestress in strands = 1.35 E+5 psi.

- Solution:

The following steps can be implemented for the two cases: (a) Prestressing strands alone (no CFRP wrap) with no loss, (b) Prestressing strands with 20% loss plus 2 layers of CFRP wrap.

A. Case of Steel Alone without Loss

1. Select a compression concrete zone depth, say a=6 in. and from Eq. (1) compute β_1=0.85,
2. Compute $c = a/\beta_1$ = 7.06 in.,

It follows: y'= a / 2 = 3 in., y_t = h/2 = 14/2 = 7 in., A_{comp}= b x a =14 x 6 = 84 in.^{2}

3. Compute strains using Eqs. (5) and (6)

   ε'_{ps}= 0.003171

   ε_{ps} = 0.00657

4. Compute stresses using Eqs. (8)

   f'_{ps}= 8.72 E+4 psi

   f_{ps}= 1.81 E+5 psi

5. Compute P_n and M_n using Eqs. (10) and (11)

   P_n = 161 kips

   M_n = 1310 kip-in.

6. This defines one point in the P-M interaction diagram for strands only with no loss
7. Go to step 1 and select another depth a, yielding a second point of the P-M interaction diagram, and so on.

B. Case of Prestressing Strands with 20 % Loss Plus 2 Layers of CFRP Wrap

1. Select a compression concrete zone depth, say a=6 in. and from Eq. (1) compute

   β_1=0.85,

2. Compute c = a/β_1 = 7.06 in.,

   It follows: y'= a / 2 = 3 in., y_t = h/2 = 14/2 = 7 in., A_{comp}= b x a =14 x 6 = 84 in.^{2}

3. Compute strains using Eqs. (2) to (6)

   ε'_{ps}= 0.003171

   ε_{ps} = 0.00657

   ε_f = 0.00295

4. Compute stresses using Eqs. (7) to (9)

   f'_{ps}= 8.72 E+4 psi

   f_{ps}= 1.81 E+5 psi

   f_f = 9.88 E+4 psi

5. Compute P_n et M_n using Eqs. (10) and (11)

   P_n = 173 kips

M_n = 1370 kip-in.

Note that these values are just above those corresponding to no loss, demonstrating that 2 layers should be sufficient at least for this particular point.

6. This defines one point in the P-M interaction diagram for a pile with 20% prestressing strands loss and 2 layers of CFRP wrap.
7. Go to step 1 and select another depth ‘a’, yielding a second point of the P-M interaction diagram, and so on.

Note: the two strands on the middle of the lateral faces are not detailed in this sample example, however they are taken into account in the Excel sheet used to generate the P-M diagram.

1.6.4 Uniform Corrosion versus Corrosion on Tension or Compression Faces

A parametric study was carried out on the effect of prestress loss in tension face strands and of compression face strands on the M_n-P_n interaction diagram. Three levels of prestress loss namely 0%, 10% and 20% were considered. Fig. 1.4 (A) presents the effect of prestress loss in tension face strands on M_n-P_n interaction diagram. It can be noticed that the diagrams are similar to those corresponding to uniform loss (see Fig. 1.5) and follow the same trend. From Fig. 1.4 (A), it is observed that the compression controlled part of the diagram (upper part of the curves) is not influenced by the prestress loss due to corrosion, whereas the moment capacity for typical axial service loads is slightly decreased. This decrease in the moment capacity in the lower part of the curves is clearly captured by the curves of uniform loss (see Fig. 1.5). Fig. 1.4 (B) presents the effect of prestress loss in compression face strands on M_n-P_n interaction diagram. In this case both the flexure and compression controlled parts are slightly affected by the loss but favourably since the axial load capacity is increased.

In summary this parametric study shows that the differences between P-M diagrams related to uniform loss (i.e., distributed in all faces of the column) and loss on strands located on tension face only on one hand and on compression face only on the other hand, are negligible for 0%

to 20% loss and do not affect the design. Therefore, a uniform loss is adopted for all the calculations presented hereafter in this report.

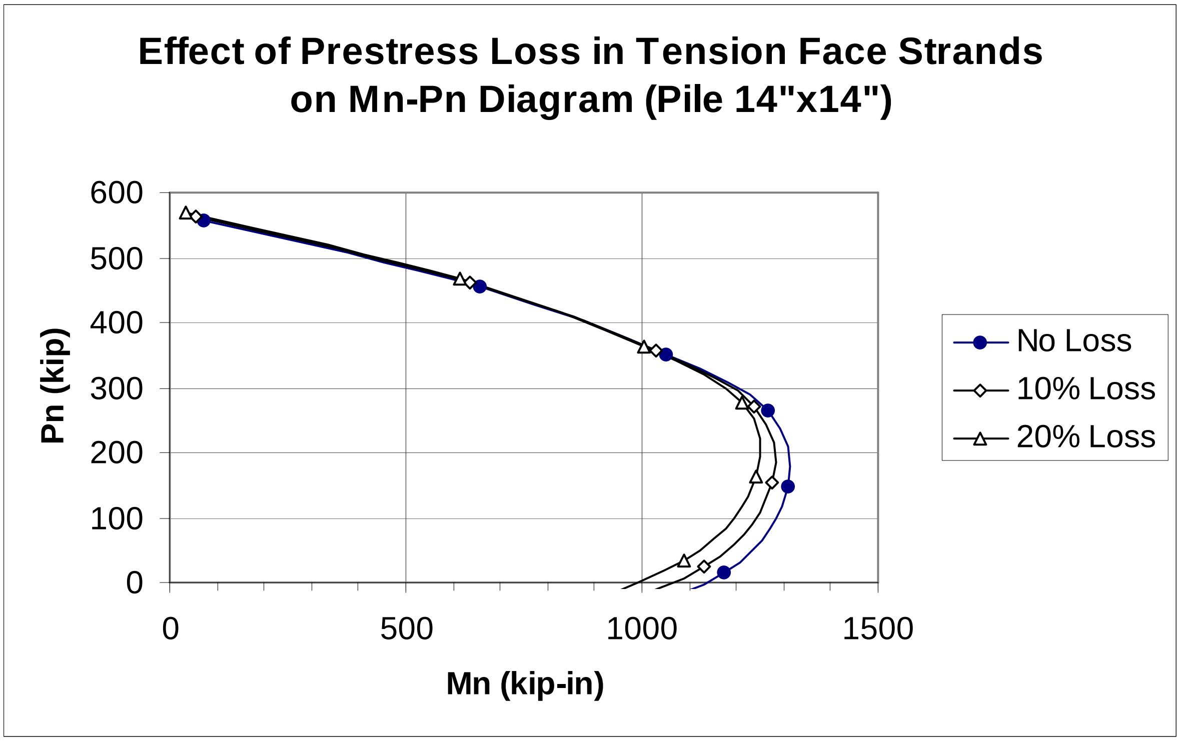


Figure 1.4 (A) Effect of Prestress Loss in Tension Face Strands

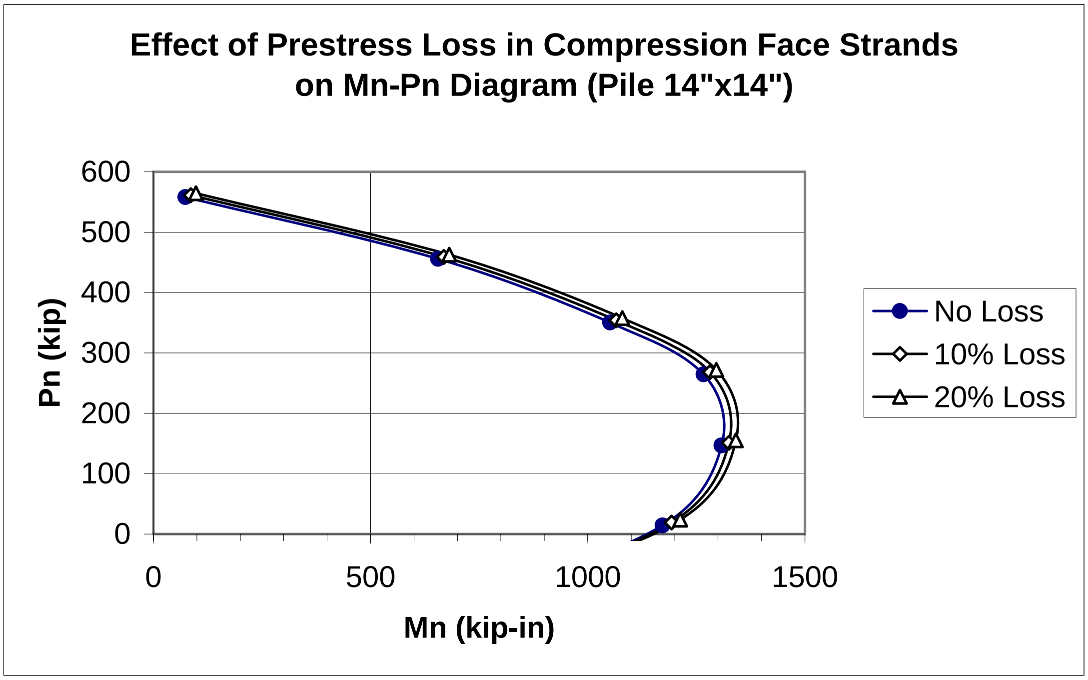


Figure 1.4 (B) Effect of Prestress Loss in Compression Face Strands

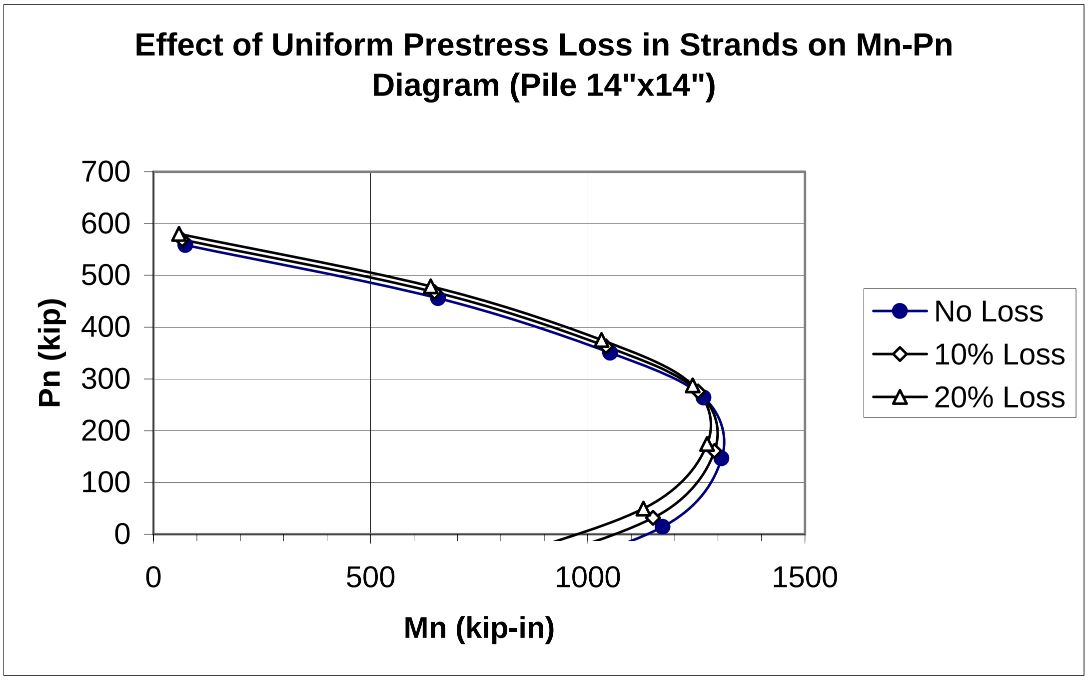


Figure 1.5 Effect of Uniform Prestress Loss (10% and 20%) on P-M Diagram

1.6.5 Presentations of P-M Diagrams

Fig. 1.6 shows the M_n-P_n interaction diagrams for the 14” x 14” piles corresponding to a uniform prestress loss of 0%, and 20%. It can be seen that the compression-controlled part of the diagram is practically not influenced by the prestress loss due to corrosion. However, the flexure-controlled portion is substantially affected by the loss. A 20% loss resulted in an approximate 15% decrease in the moment capacity for typical axial service loads. Also shown in Figure 1.6 are the results from strengthening with 2 and 3 layers of carbon wraps. It can be concluded that for an assumed concrete compressive strength of 4.0 ksi, two layers of CFRP are sufficient to restore the initial capacity of the pile.

Fig. 1.7 shows the effect of the compressive concrete strength on the overall performance. The variation of concrete strength generally influences the axial load capacity and to a lesser magnitude, the moment capacity. It can be seen from the figure that utilizing two (2) layers of CFRP is sufficient to establish the original load carrying capacity regardless of the assumption for the compressive concrete strength.

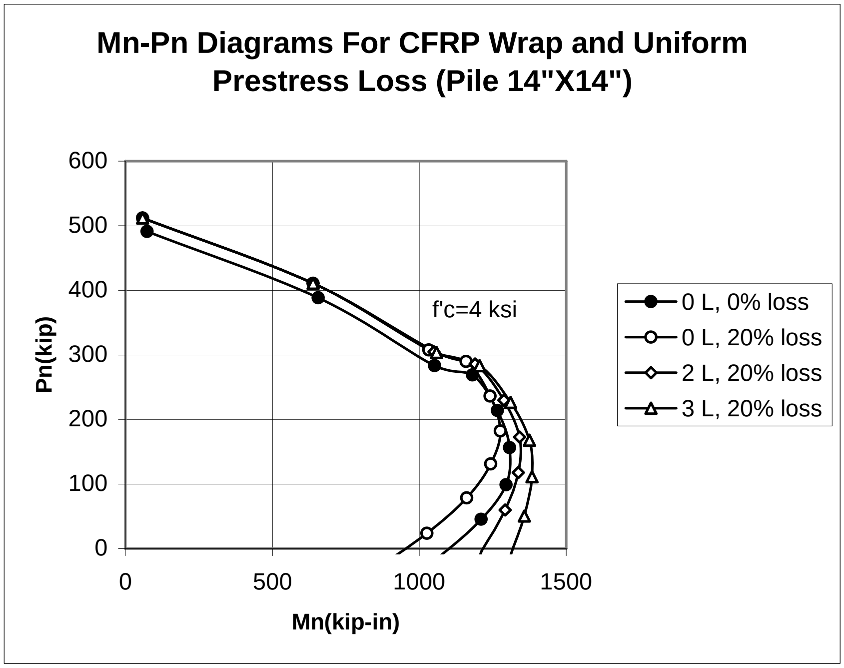


Figure 1.6 Effect of Uniform 20% Loss of Prestressing Strands

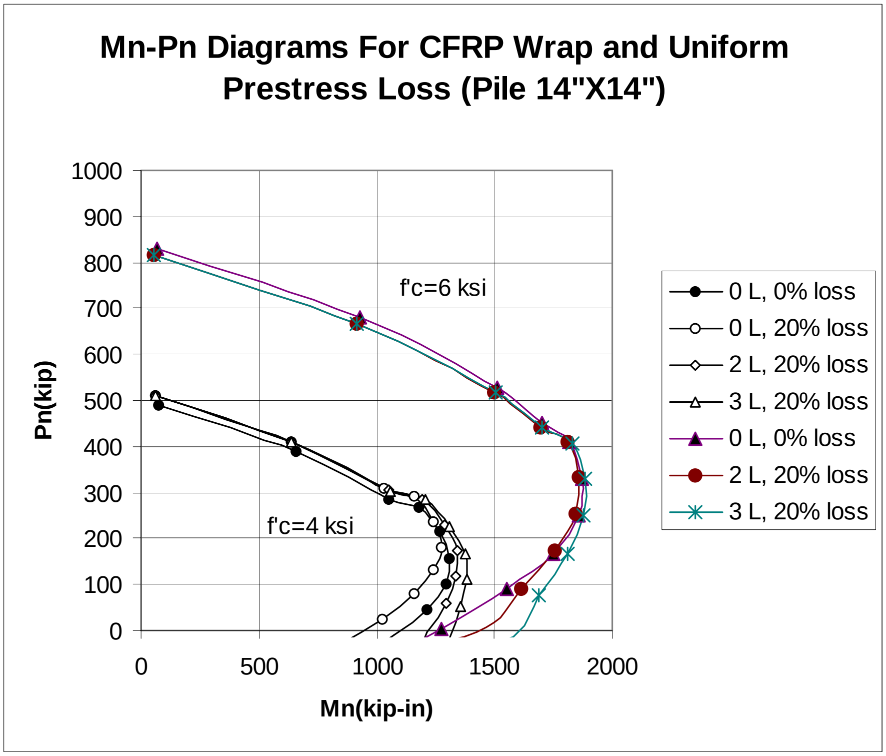


Figure 1.7 Effect of Concrete Strength

1.7 CORROSION EXPANSION DURING SERVICE LIFE

The strengthening solution outlined above uses only the longitudinal fibers of the FRP wrap. In other words the confinement effect of the lateral CFRP fibers in the case of a bi-directional wrap on the compressive concrete strength are not considered in the proposed solution, which is conservative since such a confinement would lead to yet a greater resistance of the column. Table 1.2 provides some insight on the conservative gain that can be achieved for a 14 in. x 14 in. pile wrapped with different numbers of CFRP layers, using the authors' model[4]. For convenience, the notations used are those of the source paper. This confinement effect provided by the FRP wrap in the transverse direction can also be used to mitigate corrosion by containing the expansion of concrete that develops as steel corrodes during the service life of the pile.

Table 1.2 Gain in Compression Strength of 14”x 14” 4000 psi Concrete Pile As a Function of the Number of CFRP Layers

Number of CFRP Layers[(b)]	**Strength of Confined Concrete f_{cc}**[(a)] (psi)	**Gain Due to CFRP** (%)
0	4000	0.00
1	4093	2.33
2	4188	4.70
3	4282	7.05
4	4376	9.40

Notes: (a) $f_{cc} = f_{co} + 4.12 \times 10^5$ k , where $k = E_{frp} A_{frp} / E_{co} A_{co}$ and f_{co} = 4000 psi = unconfined concrete strength; E_{frp} = modulus of elasticity of FRP jacket; A_{frp} = thickness x 1 in. = area of FRP per inch column length in the lateral direction; E_{co} = modulus of elasticity of unconfined concrete; A_{co} = cross section area of unconfined column.

(b) Gain for other numbers of layers can be obtained using the equation given in (a)

The number of layers of FRP wrap needed to contain corrosion expansion is an important element that needs to be addressed.

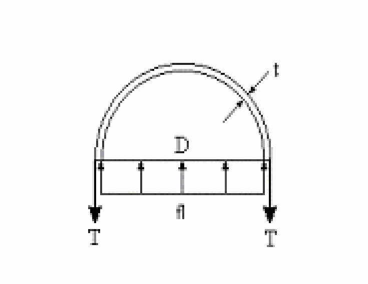
In the following calculations, three methods are used to address the problem. These methods are based on: (a) CALTRANs’ recommendations[26], (b) ISIS recommendations[27], and (c) the model proposed by Chaallal, O., Shahawy, M. and Hassan, M.[25]. However, it must be noted that the CALTRANs’ recommendations, particularly those for the column hinge zones, are very severe since they are applicable for bridges located in seismic regions, such as California. Therefore they must be adapted for reality in Florida (see numerical application A3 below). Note that all the notations used are those of the source publications; however they are redefined here for convenience.

1.7.1 CALTRANS’ Model[26]

According to CALTRANS, the analysis to be performed is for a circular column, with the resulting number of layers found multiplied by a factor of 1.5 to give the correct layer setup for a rectangular column.

Based on the cross sectional area of concrete, the square pile 14in. x 14in. is equivalent to a circular pile with a diameter D =15.8 in.

CALTRANS recommends a confining stress of 300 psi at strain of 0.004 in the hinge region and 150 psi at strain of 0.004 in other regions. For such a situation, the thickness of FRP can be calculated as follows:



$T \quad = (A_j)(f_{yj})$

Where:
T = the hoop stress developed in the jacket
t = the jacket thickness based on the composites dry fiber thickness (layers)
D = the column diameter
f_l = the confining stress developed over the surface defined by "D"

For equilibrium, it follows:

$$\frac{2\,(T)}{s} = (f_l)(D)$$

Where "s" is the wrap spacing and "t" may be taken as $\frac{A_j}{s}$

Since advanced composite materials do not typically exhibit a yield stress and the strains will be limited based on the radial dilating strain mentioned above, f_{yj} may be defined as follows:

$$f_{yj} = (E_j)(\varepsilon_j)$$

Where: Ej = the Young's Modulus for the jacket material multiplied by an appropriate reduction factor of 0.90.

ε_j = the dilating strain as defined above.

Upon substitution, it follows:

$2(t)(E_j)(\varepsilon_j) = (f_l)(D)$, from which the required FRP thickness t follows.

Numerical applications:

- *A1) Hinge region*

2 (t)(E_j)(0.9)(0.004) = (300 psi)(D) where 0.9 a reduction factor for modulus of elasticity of FRP.

2 (t)(33,500,000)(0.9)(0.004) = (300 psi)(15.8)

t=0.02 in.

t (1 layer) = 0.0048 in.

Number of layers = 0.02/0.0048=4.16

For rectangular we need 1.5 x 4.16 = 6.24 (say 7 layers)

- *A2) Other region*

2 (t)(E_j)(0.9)(0.004) = (150 psi)(D) where 0.9 a reduction factor for modulus of elasticity of FRP.

2 (t)(33,500,000)(0.9)(0.004)=(150)(15.8)

t = 0.0098 in.

t (1 layer) = 0.0048 in.

Number of layers = 0.0098/0.0048=2.04

A rectangular column would require 1.5 x 2.04= 3.06 (say 4 layers)

- *A3) Application to Florida*

The data shows that the unrestrained strain corresponding to 20% metal loss is around 1.11%. This value is high because the cracks were free to expand. However, the expansion just prior to cracking (the ultimate tensile strain) is roughly 10% of the ultimate compressive strain (0.1)(0.003) =0.0003 and provides a lower bound on the expansion strain. It is believed that a strain value of 0.001 (that is a safety factor of 3) is appropriate for this case.

In this instance the confining stress for 3 layers of CFRP lateral confinement (t = 3 x 0.0048 = 0.0144 in.) can be calculated as follows using CALTRANS' Equation:

2 (t)(E_j)(0.9)(0.001) = (f_l)(D)

f_l = 2 (0.0144)(33,500,000)(0.9)(0.001)/(15.8) =

f_l = 55 psi for a circular section

That is 1.5 x 55 = 83 psi for a rectangular column.

Using the CALTRANS' equation and for a confining stress of 150 psi and a strain of 0.001, it would require 8.2 layers of CFRP wrap for circular column and 12.3 layers for rectangular columns.

- Summary for CALTRANS

Table 1.3 gives the number of CFRP layers for different confinement strain and stress after CALTRANS recommendations.

Table 1.3 Number of CFRP Layers for Different Confinement Strain and Stress Based on CALTRANS Model for Circular Column

Confinement[(a)]		Number of CFRP Layers[(b)]		Comments
Strain	Stress (psi)	Circular	Rectangular[(c)]	
0.004	300	4.1	6.2	For hinge region in seismic areas – Very severe
0.004	150	2.1	3.1	For other region in seismic areas
0.002	150	4.1	6.2	
0.001	150	8.2	12.3	
0.001	55	3	4.5	Values for 3 layers

Notes: (a) For circular column; (b) Based on thickness/layer = 0.0048 in.; (c) Value of circular times 1.5

1.7.2 ISIS Model[27]

ISIS Canada limits the lateral strains in FRP wrap for rectangular columns to 0.002, which is twice the representative value of 0.001. The lateral stress $f_{l,frp}$ is given by the following equation:

$$f_{l,frp}=\frac{2N_b\phi_{frp}E_{frp}\varepsilon_{frp}t_{frp}(b+h)}{bh} \qquad (12)$$

Where:

b = width of the concrete cross section

E_{frp} = modulus of elasticity of FRP

$F_{l,frp}$ = lateral stress of FRP

h = height of the concrete cross section

N_b = number of FRP layers

t_{frp} = thickness of one FRP layer

ε_{frp} = strain of FRP

Given that the number of layers recommended for a 20% loss is 2, we calculate the lateral stress corresponding to the same number of layers, i.e., $N_b = 2$. Using the above equation, it follows:

$f_{l,frp} = 2 \text{ x } 2 \text{ x } 0.75 \text{ x } 33{,}500{,}000 \text{ x } 0.002 \text{ x } 0.0048 (14 + 14) / (14 \text{ x } 14) =$ 138 psi.

For a confining stress of 138 psi and a strain of 0.002, two (2) layers of FRP are required. It follows that for 300psi confinement, as recommended by CALTRANS for plastic hinge zones, and a strain of 0.002, 4.3 layers will be required, same as for 138 psi and a strain of 0.001.

- Summary for ISIS

Table 1.4 gives the number of CFRP layers for different confinement strain and stress after ISIS recommendations.

Table 1.4. Number of CFRP Layers for Different Confinement Strain and Stress Based on ISIS Model for Rectangular Columns

Confinement[(a)]		Number of CFRP Layers[(b),(d)]	Comments
Strain	Stress (psi)		
0.002	138	2.0	
0.002	300	4.3	
0.001	138	4.3	
0.001	83[(c)]	2.6	To be compared to last line of Table 1.3

Notes:
(a) For rectangular column
(b) Based on a thickness per layer of 0.0048 in.
(c) Equivalent to 55 psi in circular column. To be compared to last line of Table 1.3 (CALTRANs)
(d) $Nb = \dfrac{f_L\,(b \times h)}{2\,\phi_{frp}\,E_{frp}\;\varepsilon_{frp}\,t_{frp}\,(b+h)}$

1.7.3 Chaallal, O., Shahawy, M. and Hassan, M.' Model[13]

Based on an expansion strain of 0.001 (That is, the strain at crack multiplied by a safety factor of 3), the ultimate lateral strain, $\varepsilon_{cc,t}$ for a given confinement coefficient k (provided by 2 layers of CFRP) and an unconfined concrete strength f_{co}, is calculated according to the model for rectangular columns as follows:

$$\varepsilon_{cc,t} = [4.2 + 4000\ k - 320\ 000\ k^2]/f_{co} \qquad (13)$$

With

f_{co}= 4000 psi, and k is the stiffness factor given by:

$$k = E_{frp}\ A_{frp}\ /\ E_{co}\ A_{co} \qquad (14)$$

That is:

k = 33,500 x 2 x 0.0048 / (3605 x 14 x 14) = 0.0005

It follows:

$$\varepsilon_{cc,t} = 0.0015$$

The two (2) CFRP confinement layers provided will therefore provide a lateral strain of 0.0015. Note that this strain value is of the same order as the representative strain corresponding to crack strain times a SF of 3.

Using this model and assuming that $\varepsilon_{cc,t} = 0.002$, it will require approximately 4.4 layers of CFRP.

- Summary for Chaallal, O., Shahawy, M. and Hassan, M.'s Model

Table 1.5 gives the number of CFRP Layers for different confinement strain based on authors' model. A sample example for the third entry for instance is provided below:

The problem here is to obtain the number of CFRP layers, N, given $\varepsilon_{cc,t} = 0.003$ (third entry in Table 1.5).

Applying Eq. (13) and given f_{co}= 4000 psi, yields a second order equation in terms of k:

$320{,}000\ k^2 - 4000k + 7.8 = 0$

from which the smallest positive value of k is obtained (first branch of a parabolic curve).

It follows: k = 0.00242.

Now applying Eq. (14) and substituting A_{frp} by (N x layer thickness), it follows:

$0.00242 = 33{,}550 \times N \times 0.0048 / (3605 \times 14 \times 14)$

Hence: N = 10.6 Layers.

Table 1.5 Number of CFRP Layers for Different Confinement Strain Based on Authors' Model for Rectangular Columns

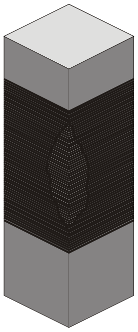
Confinement Strain[(a)]	Number of Layers
0.0015	2.0
0.002	4.4
0.003	10.6
0.004	20.9

Note: (a) for a concrete strength of 4000 psi

1.8 REHABILITATION STRATEGIES

Table 1.6 presents the suggested rehabilitation techniques as a function of prestress loss. It can be seen that for prestress loss up to 20 % two layers of CFRP wrapping can be used to restore the initial capacity of the piles.

Table 1.6 Repair Strategies

DESCRIPTION OF DAMAGE	RECOMMENDED REPAIR TECHNIQUE	COMMENTS
- 0-20% uniform strength loss due to corrosion (uniform) - Light damage with concrete spalls and up to three corroded strands - Damage zone extends to the mean high water line	1. Section restoration by patching the spalled concrete area using polymer concrete or other no shrink patching material 2. Apply 2 layers of bi-directional carbon wrap with a minimum nominal strength of 2.0 and 1.5 kips/in./layer in the transverse and longitudinal direction, respectively. The minimum length per layer is 5 feet with at least 9 inches extending below the mean low water line 	1. Repair technique and recommended patch material are as outlined in the construction specifications.

1.9 COMPOSITE WRAP APPLICATION

The Allen Creek Bridge is located along US 19 in the city of Clearwater. The bridge consists of eleven (11) 15 ft. long simply supported spans and caries carries six lanes of vehicular traffic. Each simply supported span consists of cast in place solid 24 inch reinforced concrete slab supported by reinforced concrete caps and variable size prestressed concrete piles as shown in Figure 1.8. The bridge was completed in 1951. From the visual inspection of the bridge, it appears that the substructure went through several stages of rehabilitation at various time periods. Currently the bridge is scheduled for replacement within the next five years. The bridge was identified by the FDOT to be a suitable candidate for the carbon repair demonstration project.

Four piles were identified for the carbon repair utilizing the MAS2000 Repair System. Visual inspection of these piles showed no signs of cracking. Sounding of the concrete along these piles showed no signs delamination or spalling. However, electric measurements indicated an active corrosion of the prestressing strands.

Figure 1.8 Views of the Bridge

1.9.1 Description of Repair Method

Strengthening of four piles was accomplished through the application of bi-directional carbon fiber fabric bonded to the surface of the piles according to the details provided in Table 1.6. These four piles are designated as C1, C2, D1, and D2.

The repair system, produced by SDR Engineering, Inc. was supplied in the form of fabric with varying width. The carbon composite system relies on a combination of structural grade carbon fibers impregnated with an high grade epoxy adhesive. The heart of the carbon system is the individual yarns, which make up the larger weave. These yarns are produced from a continuous length, "high-strength", "high-modulus" fiber consisting of 12,000 filaments. The combination of 12,000 filaments produces one strand with a total cross sectional area of 70 x 10^{-5} in^2 and a tensile strength of 530 ksi. The minimum material properties for the MAS2000 Repair System are shown in Table 1.7. Table 1.8 shows the material properties of the structural adhesive.

Table 1.7 Minimum Nominal Material Properties for Cured CFRP Wrap (Laminate)

Description	Values
Strength per Inch Width	
-X direction:	70x10^{-5} in.^{2}/yarn x 6.7yarn/in. x 530 ksi = 2.48 kips
-Y direction	70x10^{-5}in.^{2}/yarn x 6.7yarn/in. x 530 ksi = 2.48 kips
Elongation at Break	1.4 %
Layer Thickness of Laminate	0.02 in.
Apparent Tensile Strength$^{(a)}$	2.48 kips /(0.02 in x 1 in.) = 124 ksi
Apparent Modulus of Elasticity$^{(a)}$	8860 ksi

Note: $^{(a)}$ Based on a CFRP laminate thickness of 0.02 in. per layer.

Table 1.8 MAS-2000 Structural Adhesive Characteristics

Tensile Strength	45,170 psi
Modulus of Elasticity	2620 ksi
Viscosity	1400 cps
Working Time	Varies
Glass Transition Temperature	196 ºF

1.9.2 Application of the CFRP Laminates

Due to the site conditions and the rapid fluctuation in the water tide, the use of a cofferdam was deemed necessary to allow for surface drying and protection of the carbon system during the early stages of curing. Figure 1.9 shows the schematic details of the cofferdam. Generally, the two sides forming the cofferdam were floated to the desired pile, positioned in place and bolted together to form the cofferdam. Once the cofferdam is positioned and sealed, the existing water was drained into the creek using a sump pump. The sump pump was fitted with a sensor to detect the rise in the water level inside the cofferdam and automatically turn the pump on to drain the water and maintain the cofferdam dry. Figure 1.10 shows the installation of the cofferdam.

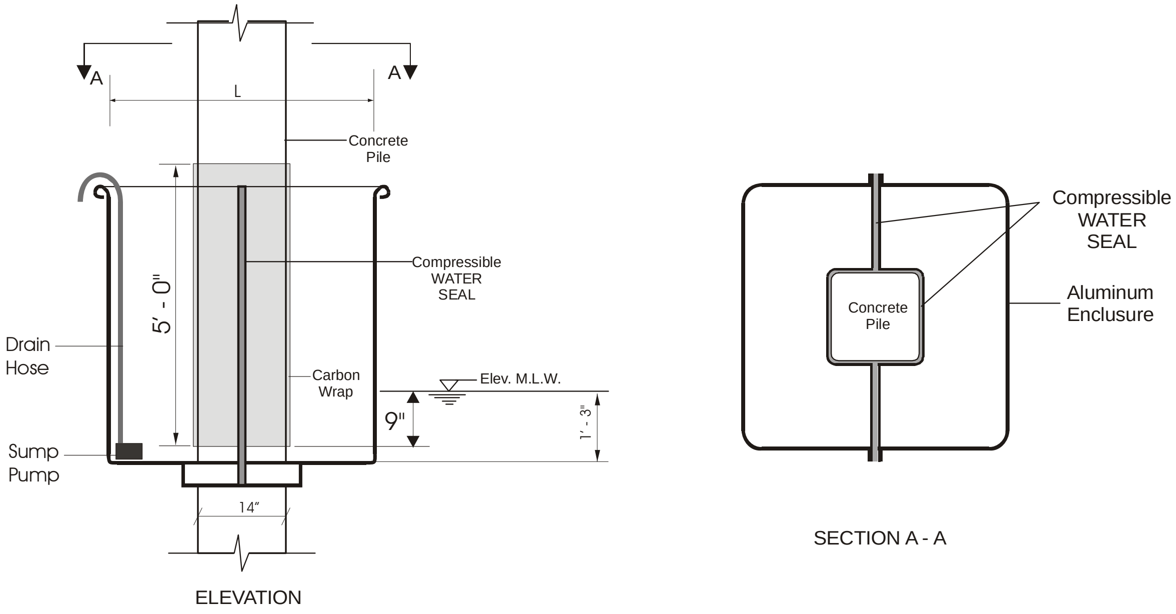


Figure 1.9 Cofferdam Details

Figure 1.10 Cofferdam installation

Once the cofferdam was installed, the concrete surface was cleaned using an electrical grinder to remove any marine organisms followed by pressure washing to remove any existing dust. Any existing holes or concrete spalls were restored to the original state using a fast drying hydraulic cement. Deformities and sharp edges were ground smooth such that areas of stress concentration were removed. The restored and cleaned pile surface was then dried out from existing moisture using a hand held torch. This treatment was necessary to eliminate the possibility of any existing moisture prior to the application of the epoxy coating. Figure 1.11 shows the preparation procedure. The preparation procedure took approximately four hours to be completed.

The carbon wrap length was 5 feet and extended for a length of 9 inches below the mean low water level. The various configurations used in the repair of the piles are shown in Table 1.9. Pile C1, C2 and D2 received an average of two layers of carbon wrap while pile D1 received four layers of carbon wrap. Applying four layers of carbon wrap was not originally planed, however, it was determined that adding this parameter will be useful in determining the effectiveness of increased number of layers. The configurations shown in Table 1.9 are designed to optimize the use of the carbon materials and to eliminate unnecessary waste. Also handling of the material and controlling the quality of the application in a tight space necessitate shorter length of fabric to eliminate the possibility of creating air pockets and insure continuous bond between the concrete and the carbon wrap.

A minimum of two-inch splice length is provided at any splice location. Also, staggering the locations of the splices is desirable and is achieved by this configuration. It should be noted that these configurations don't affect the cured laminates performance providing that the minimum splice length is maintained. Pile C1 has a two-inch horizontal seam while all other piles contained only vertical seams.

Once the pile surface was determined to be clean and dry, it was coated by a generous layer of epoxy prior to the application of the carbon wrap system. The adhesive epoxy was applied to the concrete by paint rollers. The specially designed composite carbon laminates were then applied to the pile surface within fifteen minutes of the epoxy application. Figure 1.12 shows the carbon wrap application. The removal of air pockets was accomplished by smoothing the carbon with a rubber roller. Before applying any additional layers of carbon, adhesive was applied to the pre-existing layer before it cured, then the new layer was placed in the same manner. Thickness of the bonded layer was controlled by squeezing out the excess resin with a rubber roller. The application of the MAS2000 system took approximately six (6) hours under strict quality control in order to ensure adequate bond and fiber orientation.

The original appearance of the structure was restored and will be further enhanced by painting the laminates with a specially designed UV coating.

1.9.3 Painting Composites

After wrapping the columns, exposed surfaces of composite wraps were painted with one finish coat of Sikagard 62 (manufactured by SIKA), a waterproof membrane containing UV resistance component. Coating was applied to the entire surface around the repair. The total dry film thickness of all applications of the first finish coat was approximately 2 mils.

Once the paint coating was dry (approximately 6 hours), the repaired surface was wrapped by a tightly wrapped sheet of plastic before the removal of the cofferdam. This plastic wrap is intended to provide additional protection to the repaired surfaces from the aggressive surrounding environment during the early stages (2 days). The plastic wrap was removed after 3 days. Figure 1.11 shows the several steps of the carbon wrap application.

Figure 1.11 Surface Preparation

Table 1.9 Repair Details

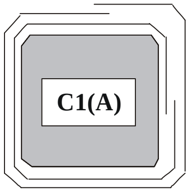
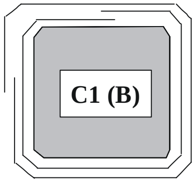
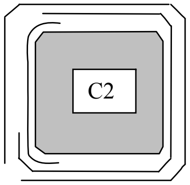
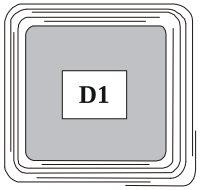
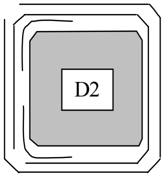
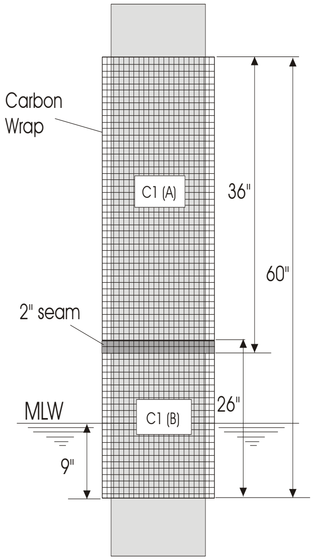
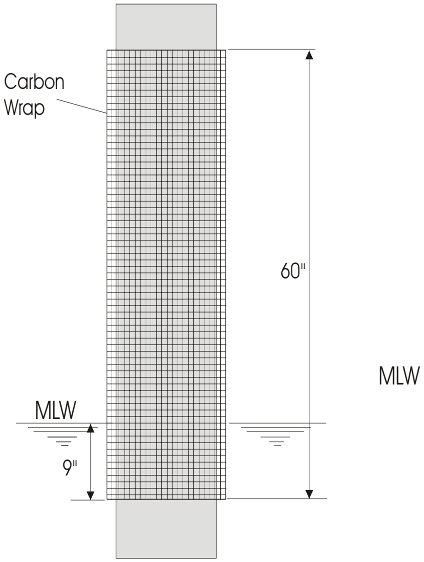
	PILE DESIGNATION			
	C1	**C2**	**D1**	**D2**
Average number Of carbon layers	**2**	**2**	**4**	**2**
Schematic Configuration of Applied Carbon Laminates	 			
				

Figure 1.12 Carbon Wrap Application

Figure 1.12 Carbon Wrap Application (Cont.)

1.10 REFERENCES

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2 SAMPLE TECHNICAL SPECIAL PROVISIONS FOR CARBON FIBER WRAP PILES REPAIR

One of the main problems facing the FDOT is the unfamiliarity with the required Technical Specifications for an FRP repair job. This chapter presents a sample Technical Special Provisions for Carbon Fiber Wrap Piles Repair. Chapter 3 discusses other aspects of the quality control and assurance thought a typical project. The information provided in Chapters 2 and 3 are for guidance only and need to be modified according to the basic requirements of each job.

SAMPLE TECHNICAL SPECIAL PROVISIONS

Table Of Contents

DIVISION 1 - GENERAL SPECIFICATIONS

SECTION 01100: SUMMARY OF WORK

1.0 GENERAL

1.1 DESCRIPTIVE SUMMARY OF THE WORK

The scope of the work for this project is the repair/restoration of the prestressed concrete piles for the **xxxxx** project.

The work shall include the cleaning, preparation, crack repair, treatment of corroded steel reinforcement and repair including application of the Carbon Fiber Reinforced Polymer (CFRP) in accordance with these technical specifications and drawings. Areas to be repaired are identified on the drawings. The Contractor shall provide sufficient materials and labor to perform the repairs. Any additional areas requiring repair, which are found during the course of the work, shall be brought to the attention of the Engineer. The Engineer will make a determination of the necessity for repairs. It is the contractors' responsibility to familiarize themselves with the site and scope of work prior to submitting a bid.

All of the work shall be coordinated with the Engineer, FDOT Representative, and the field Representative.

1.2 QUALITY ASSURANCE

Contractor Qualifications: The Contractor shall be a licensed General Contractor in the State of Florida and has successfully completed a minimum of five (5) projects of similar size and scope. In addition, Contractors shall be experienced in the installation of the specified product, have completed a program of instruction in the use of the specified repair material, and provides a notarized certification letter from the Manufacturer attesting the contractor is currently qualified to install the materials.

1.3 WARRANTY

All CFRP repair materials, including epoxies, CFRP sheet, and workmanship (system warranty) shall be guaranteed for a period of ____ years against defects.

2.0 SCOPE OF WORK - GENERAL

Repairs with CFRP Wrap

1. Remove unsound concrete and/or repair structural cracks using a Pressure Injection Method.
2. Treat corroded steel reinforcement and apply coating (corrosion inhibitor), according to specifications and drawings.
3. Restore section, according to specifications and drawings.

4. Prepare surface for CFRP application, according to specifications and drawings.
5. Apply CFRP sheets at required locations, according to specifications and drawings.
6. Perform the appropriate testing covered in section 01300 of these specifications.
7. Apply protective coatings, according to specifications and drawings.

3.0 EXECUTION

A. Pre-construction Meeting

If a subcontractor is to perform any part of the work he shall attend the Pre-construction meeting with the FDOT, Engineer, and Field Representative(s) to review the Work Plan, Schedule, Maintenance Of Traffic (MOTs), and any submittal issues.

B. Materials

The Contractor, according to specifications and drawings, shall provide concrete repair material, CFRP sheets, and top coatings. The surfaces should be prepared as noted in the drawings and specifications following manufacturer's recommendations.

3.1 QUALITY CONTROL

The Contractor shall be responsible for quality control of all construction materials and methods incorporated into the work of this contract. All work failing to comply with the requirements of the Contract Documents shall be rejected by the Engineer and shall be removed, replaced, or remedied by the Contractor to be in full compliance with the Contract Documents at no additional cost to the FDOT.

3.2 CLEAN-UP

At completion of the work, the Contractor shall remove from the working site all tools, appliances, surplus materials, debris, temporary structures and facilities, scaffolding, and equipment; sweep clean the work area thoroughly; and remove all marks, stains, and the like from all new surfaces and existing surfaces.

END OF SECTION

SECTION 01200: QUALITY CONTROL

1.0 GENERAL

1.1 REFERENCES

A. American Concrete Institute (ACI):

1. 503R-93 Use of Epoxy Compounds with Concrete.
2. 546R-96 Concrete Repair Guide.
3. 318-02 Building Code Requirements for structural concrete.
4. ACI 117 – Specifications for Tolerances for Concrete Construction and Materials.

B. American Society for Testing and Materials (ASTM):

1. A 370 Test Methods and Definitions
2. D 3039-93 Test Method for Tensile Properties of Fiber Resin Composites
3. D 4541-93 Standard Test Method for Pull-off Strength of Coatings Using Portable Adhesion Tester

C. International Concrete Repair Institute (ICRI):

1. #03730 Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion
2. #03732 Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays
3. #03733 Guide for Selecting and Specifying Materials for Repairs of Concrete Surfaces

1.2 RELATED SECTIONS

A. Section 03100 – Testing Requirements.

1.3 QUALITY ASSURANCE – CONTROL OF INSTALLATION

A. Manufacturer/Supplier Qualification:

1. The Manufacturer/Supplier must specialize in the manufacturing of the products specified in these Specifications with documented experience.

2. The Manufacturer/Supplier must have a minimum of 25 documented successful field installations with a minimum of five years in business.

3. The Manufacturer/Supplier must support a training program to instruct applicators in the installation of the products specified in these Specifications.

B. Applicator Qualification:
Applicator must be approved by the Manufacturer/Supplier.

C. Field Representative:

A Field Representative who has completed the course of instruction on proper installation of CFRP (supported by the Manufacturer/Supplier) must be present on site during concrete section restorations, surface preparation, installation of the CFRP system, installation of the Protective Coating, and all required testing. The Field Representative will observe site conditions of surfaces and installation, quality of workmanship, start-up of equipment, testing, adjustment and balancing of equipment, and will initiate instructions when necessary. The Field Representative must be fully qualified to perform all aspects of the work.

Submit qualification of Field Representative to the Engineer and FDOT Representative for approval prior to starting work.

a. The Field Representative will attend the pre-construction meeting and subsequent meetings, as required.

b. The Field Representative will maintain a Daily Construction Log.

c. The Field Representative will report observations and site decisions or instructions given to applicators or installers that are supplemental or contrary to Manufacturers’ written instructions.

d. The Field Representative shall be a third party acceptable to the Manufacturer, Contractor, Engineer, and the FDOT. Other alternatives to a third party Field Representative may be considered. The Contractor can submit alternatives to the Engineer and FDOT for approval.

e. The Contractor shall be completely responsible for all of the expenses for the Field Representative and the contract price shall include full compensation for all costs in connection therewith.

D. Contractor

1. Monitor quality control over repair products, services, site conditions, and workmanship, to produce work of specified quality.
2. Comply with Manufacturers’ instructions, including each step sequence.
3. Should Manufacturers’ instructions conflict with Contract Documents, request clarification from Engineer before proceeding with the work.
4. Comply with specified standards as minimum quality for the work except where more stringent tolerances, codes, or specified requirements indicate higher standards or more precise workmanship.
5. Perform Work by persons qualified to produce required and specified quality.

1.4 TOLERANCES

A. Monitor fabrication and installation tolerance control of Products to produce acceptable Work. Do not permit tolerances to accumulate.

B. Comply with Manufacturers' tolerances. Should Manufacturers' tolerances conflict with Contract Documents, request clarification from the Engineer before proceeding with the work.

C. Adjust Products to appropriate dimensions; position before securing Products in place.

1.5 STANDARDS

A. For Products or workmanship specified by association, trade, or other consensus standards, complies with requirements of the standard, except when more rigid requirements are specified or are required by applicable codes.

B. Obtain copies of standards where required by product specification sections.

2.0 EXECUTION

2.1 EXAMINATIONS

A. Verify that existing site conditions are acceptable for starting work. Submission of a bid for the work means acceptance of existing conditions.

B. Examine and verify specific conditions described in individual specification sections.

2.2 PREPARATION

A. Clean substrate surfaces prior to applying next material or substance. Follow Manufacturer's instructions.

B. Seal or inject cracks or openings of substrate prior to applying next material or substance.

C. Apply Manufacturer required or recommended substrate primer, sealer, or conditioner prior to applying any new material or substance in contact or bond.

END OF SECTION

DIVISION 2 – CONCRETE RESTORATION AND CARBON FIBER REPAIR

SECTION 02100: CRACK AND CONCRETE REPAIR

1.0 GENERAL

1.1 Crack And Concrete Repair

A. Work Included

1. Furnish all materials, labor, tools and equipment for the repair of cracks with varying depths as designated by the Engineer.
2. Furnish all materials, labor, tools and equipment for concrete repair as necessary.

1.2 Quality Assurance

A. Manufacturing Qualification: the manufacturer of the specified product shall have had in existence for a minimum of 5 years, a program of training, certifying, and technically supporting a nationally organized program to provide training for Contractors.

B. Contractor Qualifications: contractors shall be experienced in the installation of the specified product, have completed a program of instruction in the use of the specified repair materials, and provide a certified letter from the Manufacturer attesting that they are currently qualified to install the materials.

C. Warranty: the contractor and manufacturer shall provide the FDOT with a (___) five-year warranty on the application and product covered in this specification.

1.3 Delivery, Storage, And Handling

A. Deliver the specified product in original, unopened containers with the Manufacturer's name, labels, product identification, and batch numbers.

B. Store and condition the specified product as recommended by the Manufacturer.

1.4 Job Conditions

A. Environmental Conditions: do not apply material if it is raining or if the rain appears to be imminent.

B. Protection: precautions should be taken to avoid damage due to mixing and handling of the specified repair material to any surface, property, or vehicle near the work zone.

1.5 Submittals

A. Submit three copies of Manufacturer's literature, to include: Product Data Sheet, System Data Sheet, Application Guide, and appropriate Material Safety Data Sheet (MSDS).

D. Submit, at the completion of the project, the warranty for the application and materials for the contract specified period.

2.0 PRODUCTS

2.1 Acceptable Materials

A. Epoxy resin adhesives:

1. Sikadur 35, Hi-Mod LV (LPL optional), as manufactured by Sika Corporation, Lyndhurst, New Jersey, Concresive 1380, manufactured by Master Builders Inc., Cleveland, Ohio, or approved equal shall be used for the pressure injection of cracks.
2. Sikadur 31, Hi-Mod Gel, as manufactured by Sika Corporate, Lyndhurst, New Jersey, Concresive LPL Paste or MBrace Putty where applicable, manufactured by Master Builders, Inc., Cleveland, Ohio, or approved equal shall be used for concrete repair, the sealing of cracks, and porting devices for the pressure injection of cracks.

2.2 Performance Criteria

A. Properties of the mixed epoxy resin adhesive used for the pressure injection grouting:

1. Pot Life: 20-30 minutes or 60-65 minutes, (LPL, long pot life)
2. Tack-free Time to touch (3-5 mils): 2.5-4 hours @ 73 degrees F
3. Initial Viscosity (Brookfield Viscometer, Spindle #2; speed 100): 300-450 cps
4. Color: clear, amber

B. Properties of the cured epoxy resin adhesive used for the pressure injection grouting:

1. Compressive Properties (ASTM D-695) at 28 days
   a. Compressive Strength: 10,000 psi minimum
   b. Modulus of Elasticity: 30 x 10^4 psi minimum at 7 days
2. Tensile Properties (ASTM D-638) at 14 days
   a. Tensile Strength: 7000-psi minimum.
   b. Elongation at Break: 3-5%
   c. Modulus of Elasticity: 35 x $10^{4 \text{ psi}}$ minimum.
3. Flexural Properties (ASTM D-790) at 14 days
   a. Flexural Strength (Modulus of Rupture): 12,000 psi minimum
   b. Tangent Modulus of Elasticity in Bending: 3.0 x 10^5 psi minimum
4. Shear Strength (ASTM D-732) at 14 days: 4500 psi minimum.
5. Total Water Absorption (ASTM D-570) at 7 days: 1.5% (2 hour boil)
6. Bond Strength (ASTM C-882) Hardened Concrete to Hardened Concrete
   a. 2 day (dry cure): 2400-psi minimum.
   b. 14 day (moist cure): 2300-psi minimum.
7. Deflection temperature (ASTM D-648) at 7 days: 108 degrees F min (fiber stress loading = 264 psi)
8. The epoxy resin adhesive shall conform to ASTM C-881, and AASHTO M235-90

9. The epoxy resin adhesive shall be approved by the United States Department of Agriculture.

C. Properties of the mixed epoxy resin adhesive used for concrete repair, the sealing of cracks, and porting devices are available in FDOT QPL.

D. Properties of the cured epoxy resin adhesive used for concrete repair, the sealing of cracks, and porting devices:

1. Compressive Properties (ASTM D-695) at 28 days
   - a. Compressive Strength: 10,000 psi min.
   - b. Modulus of elasticity: 7.0 x 10^5 psi min.
2. Tensile Properties (ASTM D-638) at 14 days
   - a. Tensile Strength: 3000 psi min.
   - b. Elongation at Break: 0.3% min.
   - c. Modulus of Elasticity: 6.3 x 105 psi min.
3. Flexural Properties (ASTM D-790) at 14 days
   - a. Flexural Strength (Modulus of Rupture): 3700 psi min.
   - b. Tangent Modulus of Elasticity in Bending: 8.5 x 10^5 psi min.
4. Shear Strength (ASTM D-732) at 14 days: 3800 psi min.
5. Total Water Absorption (ASTM D-570) at 7 days: 1.0% mix. (2 hour boil)
6. Bond Strength (ASTM C-882) Hardened Concrete to Hardened Concrete.
   - a. 2 day (dry cure): 2800-psi min.
   - b. 14 day (moist cure): 2000-psi min.
7. Deflection temperature (ASTM D-648) at 14 days: 104 degrees F min. (fiber stress loading = 264 psi).
8. The epoxy resin adhesive shall conform to ASTM C-881, and AASHTO M235-90
9. The epoxy resin adhesive shall be approved by the United States Department of Agriculture.

2.3 Materials

A. Epoxy resin adhesive for the pressure injection of cracks:
   1. Component “A” shall be modified epoxy resin of the epichlorohydrin bisphenol A type containing suitable viscosity control agents. It shall not contain butyl glycidyl ether.
   2. Component “B” shall be primarily a reaction product of a selected amine blend with an epoxy resin of the epichlorohydrin bishpenol A type containing suitable viscosity control agents and accelerators.
   3. The ratio of Component “A”: Component “B” shall be 2:1 by volume.

B. The epoxy resin adhesive for concrete repair, the sealing of cracks, and porting devices:
   1. Component “A” shall be modified epoxy resin of bisphenol A type, containing suitable viscosity control agents and pigments. It shall not contain butyl glycidyl ether.

2. Component "B" shall be primarily a reaction product of a selected amine blend with an epoxy resin of the epichlorohydrin bisphenol A type, containing suitable viscosity control agents, pigments, and accelerators.
3. The ratio of component "A": Component "B" shall be as recommended by the Manufacturer by volume.
4. The material shall not contain asbestos.

C. Porting devices as required for either manual or automated application. Porting devices for automated application shall be supplied from the Manufacturer of the pressure injection equipment.

2.4 Mixing And Application Of Crack Injection Epoxy

A. Mixing the epoxy resin adhesive for concrete repair, sealing the cracks, and porting devices. Premix each component. Proportion Component "A" to Component "B" by volume as recommended by Manufacturer into a clean dry mixing pail. Mix thoroughly as per the manufacturer recommendations. Only that quantity of material that can be used within its pot life shall be mixed.

B. Mixing of the epoxy resin adhesive used for the pressure injection grouting:

1. Manual: premix each component by volume as recommended by Manufacturer into a clean dry mixing pail. Mix thoroughly as per the manufacturer recommendations. Only that quantity of material that can be used within its pot life shall be mixed.

2. Automated: the injection equipment is used to meter and mix the two components of the epoxy resin adhesive and dispense the product into the prepared cracks. The unit shall be portable and be equipped with positive displacement-type pumps with interlock to provide positive ratio control of exact proportions of the two components of the epoxy resin adhesive at the nozzle. The pumps shall be air powered or electric, shall provide an in-line mixing and metering system, and shall contain drain-back plugs.

C. Placement procedure:

1. The epoxy resin adhesive for concrete repair, sealing the cracks, and porting devices: set-porting devices as required by the Manufacturer. Spacing of the porting devices shall be accomplished as required to achieve the travel of the epoxy resin adhesive for the pressure injection grouting between ports and fill the cracks to the maximum. On structures open on both sides, provide porting devices on opposite sides at staggered elevations. Apply the mixed epoxy resin adhesive for sealing over the cracks and around each porting device to provide adequate seal to prevent the escape of the epoxy resin adhesive for the injection grouting.

2. The epoxy resin adhesive for the pressure injection:
   a. Manual: load the mixed epoxy resin adhesive for injection into a disposable caulking cartridge of a bulk-loading caulking gun. Inject the prepared cracks with a constant pressure in order to achieve maximum filling and penetration without the inclusion of air pockets or voids in the epoxy resin adhesive. Begin the pressure injection at the lowest port and continue until there is the appearance of the epoxy resin adhesive at an adjacent port, thus indicating

travel. Continue the procedure until all pressure injectable cracks have been filled.

b. Automated: dispense the epoxy resin adhesive for injection under constant pressure in accordance with procedures recommended by the equipment Manufacturer or as required to achieve maximum filling and penetration of the prepared cracks without the inclusion of air pockets or voids in the epoxy resin adhesive. The pressure injection of single or multiple ports, by the use of a manifold system, is possible. This decision should be made by the Contractor, based upon his experience, with the approval of the Engineer. Continue the approved procedure until all pressure injectable cracks have been filled.

C. If penetration of any cracks is impossible, consult the Field Representative before discontinuing the injection procedure. If modification of the proposed procedure is required to fill the cracks, submit said modification in writing to the Engineer for acceptance prior to proceeding.

D. Adhere to all limitation and cautions for the epoxy resin adhesives in the Manufacturers current printed literature.

E. At the Engineer direction the contractor may be required to obtain a core to evaluate the epoxy penetration and epoxy injection quality.

2.5 Cleaning

A. After the epoxy resin adhesive for grouting has cured, the epoxy resin adhesive for sealing cracks and porting devices shall be removed as required by the Field Representative. Clean the substrate in a manner to produce a finished appearance acceptable to the Field Representative.

B. The uncured epoxy resin adhesive can be cleaned from tools with an approved solvent. Disposal of waste materials will be consistent with the Manufacturer’s requirements and local ordinances. The cured epoxy resin adhesive can only be removed mechanically.

C. Leave finished work and work area in a neat, clean condition without evidence of spillovers onto adjacent areas.

2.6 Concrete For Section Restoration

A. Concrete Patching Material:

The material to be used as patching material is Form and Pump latex modified concrete such as SIKA 611, or approved equal, as one-component cementitious pump and pour mortar.

<u>Properties of Cementitious Pump and Pour Mortar:</u>

1. Shelf Life: 1 year
2. Application Time: 30 minutes
3. Color: concrete gray
4. Compressive Properties (ASTM C-109): Compressive Strength at 28 days = 6,500 psi min.

5. Tensile Properties (ASTM C-499): Splitting Tensile Strength at 28 days = 500 psi min
6. Flexural Properties (ASTM C-348): Flexural Strength at 28 days = 720 psi min.
7. Bond Strength (ASTM C-882 Modified): Bond strength 28 days = 2,200-psi min.

B. Bonding and Reinforcement Protection Agent:

The material to be used for this option is SIKA Armatec 110 (or approved equal) as a bonding and reinforcement protection agent.

Minimum Material Properties of Bonding agent:

1. Shelf Life: 1 year
2. Pot life: 90 minutes
3. Color: concrete gray
4. Compressive Properties (ASTM C-109): Compressive Strength at 28 days =8,500 psi min.
5. Tensile Properties (ASTM C-499): Splitting Tensile Strength at 28 days = 600 psi min.
6. Flexural Properties (ASTM C-348): Flexural Strength at 28 days = 1250-psi min.
7. Bond Strength (ASTM C-882) 14 days
    a. Wet on Wet: 2800-psi min.
    b. 24 hr open time: 2600-psi min.

END OF SECTION

SECTION 2200: PILE RESTORATION USING CFRP WRAP

1.0 GENERAL

1.1 Submittals

A. Submit three copies of product data indicating product standards, physical and chemical characteristics, technical specifications, limitations, installation instructions, maintenance instructions and general recommendations regarding each material.

B. The epoxy/composite supplier shall provide a five-year proven record of performance of concrete members strengthening with carbon fiber materials, confirmed by actual field tests and a minimum of 25 successful installations.

C. Submit Health and Safety Sheets and Material Safety Data Sheets (MSDS) of each product used on site and certification that the materials conform to local, state, and federal environmental and worker's safety laws and regulations.

D. The approved Field Representative with approval of the contractor shall prepare and submit for approval three sets of the QA plan including:

The shop drawings and the work plan for the installation of the CFRP pile restoration. The shop drawings and work plan shall contain all of the details of the CFRP wrap, surface preparation, crack and concrete repair materials, joint and end details, lap details and all other information required for the proper installation of the system.

The work plan will describe the testing and inspection requirements and testing equipment to be used.

1.2 Related Sections

A. Section 01200 – Quality Control
B. Section 02100 – Crack and Concrete Repair
C. Section 03100 – Testing Requirements

1.3 Quality Assurance

A. Materials Manufacturer/Supplier: Company specializing in the manufacturing of the products specified in this section with documented experience.

B. Contractor Qualifications: Contractor shall be experienced in the installation of the specified product, who has completed a program of instruction in the use of the specified material, and provides a certified letter from the Manufacturer attesting they are currently qualified to install the materials.

C. The Contractor shall inspect all materials prior to application to assure that they meet specifications and have arrived to the job-site undamaged.

D. The CFRP Reinforcement shall be completely inspected by the Contractor during and immediately following application of the composite materials. Conformance with the design drawings, proper alignment of fibers and quality workmanship shall be assured. Entrapped air shall be released or rolled out before the epoxy sets. Defects shall be noted in the Daily Construction Log, kept by the Field Representative.

E. After CFRP Reinforcement has cured, the Contractor shall inspect all the work to check for voids and/or debonding. Repairs shall be made and noted in the Daily Construction Log.

1.4 Job – Site Conditions

A. Do not apply CFRP Reinforcement materials if raining, or dew condensation is expected, or existing concrete surface is wet or if the ambient or surface temperature are below 45 degrees F or above 95 degrees F.

B. The ambient temperature and temperature of the epoxy components shall be between 50 degrees F (10 degrees C) and 80 degrees F (27 degrees C) at the time of mixing.

C. Precautions should be taken to avoid damage to any surface near the work zone due to mixing and handling of the specified material.

D. The majority of the areas specified to receive carbon wrap are above the mean low water level. Areas where the damage extends below the water level should be protected by an enclosure. The enclosure shall be watertight to allow drying of the concrete section and provide protection during the carbon wrap application. The enclosure shall be maintained in place for a minimum period of time (specified by the manufacturer) after the application of the carbon wrap to allow complete curing of the epoxy. It is recommended that the protective coating be applied prior to the removal of the water protection enclosure.

E. The Contractor is solely responsible for fume control and shall take necessary precautions against injury to Installer personnel during application of primer and resin, etc. Contractor personnel shall use protective equipment and mixing area shall be well vented to the outside. At a minimum, Installer must take the following precautions:

1. Contractor to follow all state, federal, and local safety regulations.
2. Contractor to follow all Manufacturers’ requirements.

1.5 Deliveries, Storage, and Handling

A. Deliver primer, saturant, and protective coating in original, unopened containers with the Manufacturers name, labels, product identification, and batch numbers.

B. CFRP Reinforcement fabric must be stored in accordance with the Manufacturers recommendations. Store in cool (<70 degrees F) dry area away from direct sunlight, flame or other hazards.

C. Store primer, saturant, and protective coating under conditions as recommended by the Manufacturer. Products that are not properly stored or have exceeded their shelf life shall not be used.

D. Manufacturer and Contractor are required to confirm that all materials used in accordance with this Section conform to local, state, and federal environmental and worker's safety laws and regulations.

E. The Contractor shall properly dispose of empty containers immediately.

2.0 PRODUCTS

2.1 Acceptable Manufacturers/Suppliers

A. Carbon fiber reinforced polymer (CFRP) unidirectional wrap.

1. Use SikaWrap Hex 117C System as supplied by Sika Corporation, Lyndhurst, NJ, MBrace CF 130, as supplied by Master Builders Inc., Cleveland, Ohio, or approved equal.

B. Impregnating epoxy resin.

Use the impregnating epoxy resin as specified by the carbon System Manufacturer.

C. Primers and fillers.

Use the Primers and fillers as specified by the carbon System Manufacturer.

E. Coatings.

The coatings shall be waterproof and containing UV resistance component.

2.2 Carbon Fiber Properties

A. Fiber Properties and Data

Property	Minimum Requirement	
1. Ultimate tensile strength	>500,000 psi	(3,450 N/mm^2)
2. Tensile modulus	>35 x 10^6 psi	(234,500 N/mm^2)
3. Elongation	>1.5%	
4. Density	0.065 #/in^3	(1.8 g/cm^3)
5. Weight per square yard	9 oz.	(309 g/m^2)
6. Primary fiber direction	0 (unidirectional)	

B. Cured laminate properties (For each direction)

Property	**Minimum Requirement**
Tensile strength	139,000 psi (960 N/mm^2)
Modulus of elasticity	10.6 x 10^6 psi (73,100 N/mm^2)
Elongation at break	1.33%
Thickness	0.040 in.(1 mm)
Strength per inch width	5.560 lbs/layer (24,700 N)

2.3 Surface Primer

A. Surface Primer shall be a two component, 100% solids, moisture tolerant, high modulus, high strength epoxy.

B. Surface Primer shall meet the following minimum requirements:

Property	**Requirement**	**ASTM Test**
Tensile Strength, 7 Day	3,600 psi	D638
Tensile Modulus, 7 Day	6.5x10^5 psi	D638
Elongation at Break, 7 day	1%	D638
Shear Strength, 14 Day	3,600 psi	D732
Flexural Strength, 14 Day	6,800 psi	D790
Heat Deflection Temp. (HDT)	118F	D648

2.4 Saturant

A. Saturant resin shall be two component, 100% solids, moisture tolerant, high strength, high modulus epoxy.

B. Saturant shall meet the following minimum requirements:

Property	**Requirement**	**ASTM Test**
Tensile Strength	10,500 psi	D638
Tensile Modulus	459 ksi	D638
Elongation at Break	4.8%	D638
Flexural Strength	17,900 psi	D732
Flexural Modulus	452 ksi	D790
Heat Deflection Temp. (HDT)	170F	D648

3.0 EXECUTION

3.1 General

A. Inspect surfaces to receive the work and report immediately in writing to the Engineer as required in the General Conditions any deficiencies in the surface which render it unsuitable for proper execution of this work.
B. Protect vehicles, concrete, and other items surrounding work area from dust or damage due to Work of this Section.

3.2 Concrete Section Preparation

The work under this section consists of restoring delaminated concrete on selected bridge components using polymer/latex modified concrete. Surfaces where the CFRP system is to be applied must be sound. Concrete spalls and delaminations must be repaired according to the following procedure:

A. Concrete restoration shall include the removal of all delaminated concrete from the area to be restored and an additional one to two inches from behind the rebars in delaminated areas. Any loose concrete remaining in the damaged region must be removed, leaving the member with sound concrete.

B. Cracks within solid concrete greater than 0.25mm (0.010 in.) must be stabilized using epoxy injection methods.

C. All exposed steel shall be sandblasted clean near white appearance prior to concrete placement. The exposed reinforcement and concrete surface must be treated by applying a bonding and reinforcement protection such as SIKA Armatec 110 (or approved equal) prior to concrete placement.

D. Concrete restoration shall be performed using an approved Form and Pump polymer/latex modified mortar/concrete such as Sika MonoTop 611 or approved factory bagged mortar/concrete patching material of equal characteristics. Once prepared forms will be placed over an entire area, a port or inlet with valve will be plumbed to the form with a cutoff. A method or route must be established to eliminate air being trapped. This can be quite serious if not installed during the forming process. It may be possible to drill into the base of each location and allow a path for the egress of this air.

E. The selected material shall achieve a minimum compressive strength of 4,500 and 5,500 psi in seven and 28 days, respectively. Proposed material and method of application including manufacture's technical specifications and formulation if applicable shall be submitted to the Engineer for approval prior to commencing work. At locations where due to size of spall or other constrains pre bagged mortar/concrete can not be used, a Class III latex modified concrete may be used as approved by the Engineer upon submittal of design mix.

F. The restored concrete surface shall smooth, uniform and shall match the concrete component original profile. Remove form lines, sharp edges by grinding or filling with putty. Ridges greater than 5mm may need to be ground down as per the field

Representative's inspection. Sharp corners shall be rounded to a radius of greater than 1.0 inch prior to the application of carbon fibers. Filler material, where required, shall be an epoxy.

G. All concrete surfaces shall be dry and free of surface moisture, and tested by the Field Representative to evaluate moisture transmission in accordance with ASTM D4263 "Indication Moisture in Concrete by the Plastic Sheet Method.

H. The adhesive strength of the concrete shall be verified after preparation by random pull-off testing per ASTM 4541-89, or approved equal, at the direction of the FDOT Representative. Minimum tensile strength required is 200 psi. See Section 01300-1.6.B.

I. Final approval of the surface prior to rehabilitation must be received by the Engineer prior to proceeding with the work.

3.3 Mixing Primer And Saturant

A. Mix components in accordance with Manufacturer's recommendations.

B. Viscosity may be adjusted by means of heating. Diluting is not permitted. Pre-condition materials when circumstances dictate as specified herein.

C. Mix only that quantity which can be used within its pot life.

3.4 Primer Application

A. Apply primer in accordance with Manufacturer's recommendations.

B. Primer may be applied with a brush or roller. Apply second coat as necessary after first coat has penetrated into concrete.

C. Surface depressions shall be filled and cured in advance with epoxy filler per Manufacturers instructions.

D. Follow Manufacturer's recommendations pertaining to time between priming and application of FRP Reinforcement.

E. Primer must be covered with fiber within 24 hours of application. If 24-hour window is exceeded due to unforeseen circumstances, the primed surfaces must be solvent wiped with a fast flashing solvent (e.g., MEK) or roughened with sandpaper to break the amine blush.

F. Surface irregularities caused by the primer application shall be ground and removed by disc sanding.

3.5 CFRP Reinforcement Application (Dry Lay-Up Method)

A. Apply CFRP Reinforcement in accordance with Manufacturer's recommendations.

B. FRP Reinforcement sheets shall be cut beforehand into prescribed lengths. Sheets shall be lapped in the longitudinal direction 4 inches or as indicated on the Drawings.

Note: No lapping is required of the sheets parallel to the direction of fiber orientation.

C. Follow Manufacturer's recommendations regarding primer open times.

D. Apply a primary saturant coat uniformly by roller brush.

E. Apply CFRP Reinforcement sheets fiber side down to the concrete over the fresh saturant using a ribbed roller to remove any air bubbles.

F. Apply secondary saturant coat with roller over installed sheets in order to impregnate and replenish primary saturant.

G. For succeeding CFRP Reinforcement sheets as specified in the Drawings repeat application procedures.

H. To ensure complete bond between layers, any successive layer of fabric shall be placed before the onset of complete gelation of the previous layer of epoxy.

I. The cured composite shall have uniform thickness and density, bond between layers and lack of porosity. Undulations in the completed wrap surface shall not exceed ¼ inch per foot in any direction

J. Fibers of the fabric shall not deviate from a horizontal or a vertical line by more than ½ inch per foot.

K. Carbon wraps will be computed for payment in place in square feet based on the surface area measurements of the pile and the number of layers indicated in the shop drawings.

3.6 Curing

A. Protect finished installation of CFRP Reinforcement from rain, sand, dust, etc. using protective sheeting or other barriers. Do not allow protective sheeting to come in contact with finished application.

B. Curing of finished application shall be a minimum of 24 hours prior to application of protective coating.

3.7 Repair of Defects

A. Upon completion of the curing process the installed system shall be checked for areas where saturant has not penetrated or where saturant has not completely cured. Such areas shall be epoxy injected to reestablish bond subject to the approval of the field Representative.

B. Repair procedures shall be performed in accordance with Manufacturer's recommendations and as specified by the Engineer. All repairs shall be subject to the same application, curing, and quality control specifications as the original work.

C. Small delamination less than 2 sq. in. each (1300 sq. mm) does not require corrective action, as long as the total delaminated area is less than 5% of the applied surface area per each face of the pile. No more than two such delaminations on each face of the pile will be allowed.

D. Large delaminations, greater than 25 sq. in. (1600 sq. mm) shall be repaired by selectively cutting away the affected sheet, reapplying primer and resin layers, and applying an overlapping CFRP patch of equivalent plies and fiber orientation.

E. Moderate delamination less than 25 sq. in. (1600 sq. mm) may be repaired by filling the delamination by low-pressure injection of the saturant or by the previous procedure specified for large delaminations.

F. Repair procedures for conditions that are not specifically addressed in this specification shall be submitted and approved by the Engineer prior to proceeding with the work.

3.8 Protective Coating

See Section 02300.

3.9 Cleaning

1. Uncured saturants may be cleaned from tools with an approved solvent and properly disposed.
2. Cured saturants shall be removed by mechanical means and properly disposed.

4.0 INSPECTION, TESTING, AND SAMPLING

4.1 Testing

For Testing Requirements see Section 03100

4.2 Sampling

A. Record lot number of fiber used for wrapping. Ten samples shall be prepared randomly throughout the duration of the project. The FDOT Representative shall select the time and location of these samples. Samples shall consist of two 12 inch by 12 inch (30 cm x 30 cm) layers of fiber (flat).

B. Mix samples of epoxy resin according to Manufacturer's recommendations. All materials used for the samples shall be from the same products (lot number) being used at the site on a daily basis. On a smooth, flat, level surface covered with sheet polyethylene, prepare sample by placing two layers of the composite oriented in the same direction. Cover with sheet polyethylene, squeegee out all bubbles. The prepared, identified samples shall be tested as per FDOT's requirements. All testing shall be at the contractor's expense, by an independent laboratory, in accordance with ASTM 3039. Two copies of the test results are to be submitted to the FDOT and the Engineer within five days of testing. As a minimum the testing shall consist of the following:

   1. Ultimate tensile strength.
   2. Tensile modulus.
   3. Percent Elongation.

4.3 Inspection

A. The approved Field Representative shall observe all aspects of preparation, mixing, and application of materials, including the following:

   1. Surface preparation.
   2. Material container labels.
   3. Mixing of epoxy.
   4. Application of epoxy to the fiber.
   5. Curing of composite materials.
   6. Testing of samples.
   7. All aspects related to the “Carbon Fiber Reinforced Polymer (CFRP) Reinforcement”.

END OF SECTION

SECTION 2300: PROTECTIVE COATINGS

1.0 GENERAL

1.1 Summary

This specification describes the coating of the wrap system and adjacent concrete surfaces with a non-vapor-barrier, flexible, protective, waterproofing coating. Material shall be either a polymer-modified Portland cement coating or a polymer based latex coating. Final appearance is to match, within reason, the color and texture of the adjacent concrete. Prior to application, the contractor shall submit, for approval, a representative mock-up of the final product. The mock-up shall be no smaller than 12" x 12".

1.2 Quality Assurance

The contractor must install materials in accordance with all safety and weather conditions required by Manufacturer or as modified by applicable rules and regulations of local, state, and federal authorities having jurisdiction. Consult Material Safety Data Sheets for complete handling recommendations.

1.3 Delivery, Storage, and Handling

A. All materials must be delivered in original, unopened containers with the Manufacturers name, labels, product identification, and batch numbers. Damaged material must be removed from the site immediately.

B. Store all materials off the ground and protect from rain, freezing, or excessive heat until ready for use.

C. Condition the specified product as recommended by the Manufacturer.

1.4 Job Conditions

A. Environmental Conditions: Do not apply material if it is raining, or if such conditions appear to be imminent. Minimum application temperature 40˚ F (5˚C) and rising.

B. Protection: Precautions should be taken to avoid damage to any surface near the work zone due to mixing and handling of the specified material.

1.5 Submittals

A. Submit three copies of Manufacturer's literature, to include: Product Data Sheet, System Data sheet, Application Guide, and appropriate Material Safety Data Sheets (MSDS).

B. Submit letter from Manufacturer stating that the Contractor is currently qualified to install materials.

C. Sample mock-up (minimum 12" x 12") of the coating product to be inspected for color matching and approved by the Engineer.

1.6 Warranty

A. Provide a written warranty from the Manufacturer against defects of materials for a period of (____) years, beginning with date of substantial completion of the project.

2.0 PRODUCTS

2.1 Acceptable Manufacturers/Suppliers

A. Polymer based latex coating.

   1. Sikagard 670W, as manufactured by Sika Corporation, Lyndhurst, NJ. MBrace Topcoat ATX, as manufactured by Master Builders Inc., Cleveland, Ohio. AMERCOAT 385, AMERON 450HS, or approved equal.

3.0 POLYMER BASED LATEX COATINGS

3.1 Performance Criteria

A. A minimum of a one-year shelf life.

B. Typical Properties of the mixed polymer based latex coating.

   1. Pot Life: indefinite
   2. Color: gray

C. Typical Properties of the cured polymer based latex coating (gray):

   1. Solids Content: 60% by weight 46% by volume
   2. Water Vapor diffusion (at 5 mils = 120 microns dry film thickness)

      μ - Value H_2 O (diffusion coefficient) = 3,140

      SdH_2 O (equivalent air thickness) = 1.3 ft.

   3. Carbon Dioxide Diffusion (at 5 mils = 120 microns dry film thickness)

      μ - Value CO_2 (diffusion coefficient) = 1,100,000

      SdCO_2 (equivalent air thickness) = 433 ft.

      Equivalent concrete thickness (Sc) = approx. 13 inches

   4. Flame Spread and Smoke Development (ASTM E-84-94)

      Smoke Spread: 0

      Smoke Development: 5

      Class Rating: A

   5. Weathering (ASTM G-26)

      2000 hours Excellent, no chalking or cracking.

3.2 Materials

A. Polymer Coating

   1. Material shall prevent moisture ingress.
   2. Material shall be water vapor permeable and provide carbonation barrier.
   3. Minimum dry film thickness shall be 4-6 mils.

3.3 Execution

A. Surface Preparation

1. All CFRP surfaces to be coated must be clean and dry. An open textured sandpaper sandpaper-like surface as per CSP-3 is required. On the plain concrete that is to be coated surfaces should be prepared mechanically by appropriate approved surface preparation techniques following manufacturer's recommendations. Contractor is to submit preparation technique for approval.

B. Mixing and Application

1. Mixing: Stir thoroughly to ensure uniformity using a low speed (400-600 rpm) drill and paddle. To minimize color variation when using multiple batches, blend two batches. Use one pail and maintain the second pail to repeat this procedure for the entire application.
2. Application: Recommended application temperature (ambient and substrate) 45-95F. Material can be applied by brush, roller, or spray over entire area moving in one direction. Allow a minimum of 20-90 minutes prior to recoating. At lower temperatures and high humidity, waiting time will be prolonged. At higher temperatures, work carefully to maintain a wet edge. To achieve a dry film thickness of 4-6 mils, two uniform coats should be anticipated. On porous substrates, a third coat may be necessary.
3. Adhere to all limitations and cautions for the material in the Manufacturer's printed literature.

C. Protection and Cleaning

1. The uncured polymer based latex coating can be cleaned from tools with water. The cured polymer based latex coating can only be removed mechanically.
2. Leave finished work and work area in a neat, clean condition without evidence of spillovers onto adjacent areas.

END OF SECTION

DIVISION 3 – TESTING

SECTION 01300: TESTING REQUIREMENTS

1.0 GENERAL

A trained field Representative shall observe all aspects of on-site field-testing.

1.1 References

ASTM A 370 - Test Methods and Definitions

ASTM C802 – Practice for Conducting an Inter-laboratory Test Program to Determine the Precision of Test Methods for Construction.

ASTM C1021 – Standard Practice for Laboratories Engaged in the Testing of Building Sealants.

ASTM E329 – Practice for Use in the Evaluation of Inspection and Testing Agencies as Used in Construction.

ACI 440.2R-01 – Externally Bonded FRP Systems for Strengthening Concrete Structures.

1.2 Quality Assurance

B. Comply with requirements of ASTM A 370, ASTM C802, ASTM C1021, ASTM E329, ASTM E543, ASTM E548, ASTM E699, ACI 440.2R-01.

C. Testing Equipment shall be calibrated at reasonable intervals with devices of accuracy traceable to either National Bureau of Standards or accepted values of natural physical constants.

1.3 Contractor Submittals

Prior to start of work, submit, for approval, a proposed testing plan. Include with the plan the name, address, and telephone number of the trained Field Representative.

1.4 Testing Requirements

A. Inspection for voids/delaminations (General):

1. After allowing at least 24 hours for initial resin cure to occur, perform a visual and acoustic tap test, in accordance with ACI 440.2R-01 (Part 3, Chapter 6 (6.2.3)) of the layered surface. The acoustic tap test coverage will be, at a minimum, one sounding tool strike per square foot of area coated with the CFRP. Additional testing shall be performed if an area is deemed to be suspect.
2. Other methods for detecting voids must be submitted and approved by the engineer prior to proceeding.
3. Voids requiring corrective action shall be marked and repaired in accordance with Section 02200 (3.7) of these specifications.

B. Bond Testing (CFRP fabric):

All tests under this section are to be performed by the Field Representative in the presence of the Contractor and Engineer.

1. Direct pull-off testing shall be conducted per ASTM D 4541-89, or approved equal.
2. Direct pull-off tests shall be conducted under the following test conditions:
   a. A representative area incorporating the proposed surface preparation procedure shall be tested to determine the pull-off strength of the concrete substrate only. Failure at the bond line at tensile stresses below 200 psi (1.4 MPa) is unacceptable. If the results are unacceptable the engineer has the discretion to order the contractor to remove the unacceptable materials and performing proper repair.

   b. Prior to the first CFRP installation the Contractor shall conduct a pull-off test on an installed sample of the CRFP (12"x12") to verify the tensile bond between the CRFP and the existing concrete substrate. Test location to be prescribed by the FDOT Representative. After testing, inspect the failure surface of the coupon specimen. Failure at the bond line at tensile stresses below 200 psi (1.4 MPa) is unacceptable.

      1. The CFRP system (exclusive of the protective coating) shall be allowed to cure a minimum of 24 hours before execution of the direct pull-off test.

      2. The location of the pull-off tests shall be representative of the general conditions and on a flat surface.

   c. During CFRP installation the Contractor will conduct pull-off tests to verify the tensile bond between the CFRP and the existing concrete substrate. Test location to be prescribed by the FDOT Representative. After testing, inspect the failure surface of the coupon specimen. Failure at the bond line at tensile stresses below 200 psi is unacceptable.

      1. The CFRP system (exclusive of the protective coating) shall be allowed to cure a minimum of 24 hours and a maximum of 48 hours before execution of the direct pull-off test.

C. Test Frequency (CFRP fabric):

1. Direct pull-off testing shall be conducted at the following frequency:

   a. Pull-off tests must be performed on the concrete substrate prior to application of the CFRP system. Ten tests will be required throughout the duration of the project. The FDOT Representative will select the locations where the testing will be performed.

   b. One initial CFRP pull-off test sample (12"x12") is required. The FDOT Representative will select the location where this initial testing will be performed.

   c. The installed CFRP shall be tested prior to the application of the protective coating. Perform a minimum of one direct pull-off test per pile of installed CFRP. The CFRP system shall be allowed to cure a minimum of 24 hours (maximum 48 hours) before execution of the direct pull-off test. The FDOT Representative will select the location where the testing will be performed.

D. Conditions of Acceptance (CFRP fabric):
   1. The failure mode must be cohesive failure within the concrete for tests on the CFRP.
   2. The tensile bond strength must be in excess of 200 psi (1.4 MPa).
   3. If required, repair the tested area in accordance with Section 02200(3.7) of this specification.

1.5 Material Samples

A. If requested by the FDOT, samples will be taken in amounts necessary for quality control testing of epoxy injection material, CFRP sheets, bonding resins and topping products.

2.0 TEST REPORT

A. After each test, promptly submit one copy of the test report to the FDOT Representative and Engineer.

B. Include with each daily report:
   1. Date issued.
   2. Project title.
   3. Name of inspector.
   4. Date and time of inspection or testing.
   5. Identification of product and specifications section (including batch numbers).
   6. Location of tests in the Project
   7. Type of inspection or test
   8. Results of tests.
   9. Conformance with Contract Documents and Specifications.

END OF SECTION

3 QUALITY ASSURANCE PROGRAM

The primary objective of the Process Control Manual is to ensure that bonded repair and retrofit of concrete structures using FRP composites is constructed in a manner, which conforms to contractual and regulatory requirements. Determination of conformance of the Contractor's work to the requirements is verified on the basis of objective evidence of quality.

This section describes how the Quality Assurance (QA) Program is designed to assure that all quality and regulatory requirements are recognized and that a consistent and uniform control of these requirements is adequately established and maintained.

3.1 QUALITY ASSURANCE POLICY

The QA Program main objective is to assure that project is carried out in a planned, controlled and correct manner. It includes procedures for scheduling and assigning work; recording, retention and retrieval of records for all construction activities; identifying and resolving deficiencies affecting the work; and verifying compliance with the requirements of the QA Program.

The QA Program can be modified, if necessary, to meet the needs of individual projects, or to comply with any specific requirements or agreements. The program will implement those requirements and agreements by applying them to specific activities, and will identify the items and services to which the program applies.

The QA Procedures (QAP) define the organizational structure within which the program is to be implemented, and delineate the responsibility and authority of the various personnel involved.

3.1.1 QA/QC GOALS

- Develop staff understanding and acceptance of QA philosophy and procedures.
- Develop staff understanding of their particular role in implementing QA/QC Procedures.
- Meet the Owner's need for quality product.
- Ensure that appropriate procedures are followed at each step of the process from the inspection of incoming raw materials to the application of final coating to achieve specified performance.

3.2 PREPARATION OF A PROJECT-SPECIFIC QUALITY ASSURANCE PLAN

3.2.1 Project Start-up Considerations

An important QA element before starting a construction project is becoming fully familiar with the intent and details of the plans and specifications. Identifying any apparent errors, omissions or ambiguities early in the project will help insure quality, and will limit change orders and contractual disputes. The project startup duties for the project team include:

1. Review the contract for the performance of Construction Related Services and list those administrative, inspections, observation duties and procedures for which the Contractor is responsible. A sample of those duties, which the Contractor is solely responsible for, includes:

    a. Contractor’s means and methods for construction.
    b. Safety of Contractor’s work force.
    c. Contractor’s adherence to schedule, etc.

1. Assisted by the Chief or Senior Construction Inspector, review the Construction Contract Documents (Plans and Specifications) between the Owner and the Contractor. List all administrative, procedural, inspection, and field testing responsibilities to be performed. At this stage, any discrepancies and ambiguities in the duties and responsibilities should be identified and resolved prior to proceeding with the project.
2. Understand the impact of any imposed environmental, phasing or operational limitations or constraints on the construction processes.
3. Review the proposed Project Staff and Organization for:
    a. Adequacy of staff positions needed to cover the contractual obligations.
    b. Required staff licensing and certification.
    c. Technical qualifications and experience of assigned personnel, Contractors and its Subcontractors for specialty services.
    d. Need for staff training in specific inspection procedures, safety awareness, and limitations in the authority, relations with the Contractor, its Subcontractors, the Owner, and the public.
4. Review the physical aspects of the project work area and adequacy of the facilities provided to house construction site staff.
5. Verify the availability of measuring and testing (M&T) equipment and instruments needed to verify the quality of components to be incorporated into the finished work. Check that licenses needed to own, store, or operate M&T equipment are on file. Determine which testing will be done in-house and which will be done by an independent facility.
6. Ensure availability of all forms needed to document the quality of the constructed project and its administrative processes.

3.3 CONSIDERATIONS RELATED TO THE VERIFICATION OF QUALITY OF THE CONSTRUCTED PROJECT

Review the contract agreement for the specific obligations related to the following:

3.3.1 Constructability Review

Following are important items to consider during the constructability review:

1. Check for realistic scheduling of work activities. Identify need for overtime and double shift work and unusually high peak demand for machinery and construction plant.
2. Check the proposed construction schedule for compatibility with sequencing and phasing of the work, as related to natural phenomena, such as high flood periods, hurricane seasons, high tides and general inclement weather periods.

3. Check for proper sequencing of operations. Identify any operations that are on the critical path, and could cause delays and possible loss of a construction season.
4. Check for adequate rights-of-way and access to the construction areas. Verify adequacy of areas reserved for Contractor’s work, lay down and storage areas.
5. Check for interference with traffic, utilities and other on-going or sequential contract work by others.
6. Identify long-lead items and the need for unusual construction materials and equipment.
7. Check for use of appropriate materials and up-to-date designs and technology. Identify use of unconventional or highly specialized designs or expensive materials, which could limit competition and result in high bids. Verify that new materials are being used in the manner intended by the manufacturer.
8. Check contract documents for ambiguities and inconsistencies that could lead to schedule delays, contractual disputes and possible legal actions. Verify that details shown are adequate to assure proper erection and construction sequencing.
9. Check for community impacts such as noise, dust, and release of toxic or otherwise unsafe materials into the environment.
10. Check for conformance to all governmental regulations, which safeguard the environment, the work place and the public.
11. Check that accessibility for maintenance, repair and in-service inspection has been provided. Review maintenance, repair and inspection requirements and verify that the design shown on the drawing provides adequate access for these activities.
12. Avoid duplication of data in the specifications and the drawings, by ensuring that:
    a) Dimensions are correct and consistent, and tolerances are appropriate.
    b) Drafting practices conform to the standards specified.
    c) Drawings are legible.
    d) Drawings reproduce satisfactorily.

3.4 QUALITY ASPECTS OF CONSTRUCTION SPECIFICATIONS

3.4.1 Contract Documents

Problems related to the specifications that may lead to change orders, claims, arbitration and litigation are broken down into the following categories in descending order of frequency:

“Or Equal” Specifications

To avoid problems with “or equal” specifications it is best to list those physical or functional properties of the name brand product you wish to see duplicated in the “or equal” product.

Constructability

Contract documents may be defective if the work shown is not reasonably constructible. Remember the ordinary sequence of trades in the construction process, and look for bad phasing or details that require a succeeding trade to install something before the preceding trade would normally arrive on the job.

In setting the tolerances, be sure that they follow industry standards or are no more stringent than contained in standard specifications. If more stringent tolerances are required, word the specifications so that the Contractor's attention is alerted to this fact, so that he/she can adjust his/her normal work methods and pricing to achieve the results required.

Overly strict or literal interpretation of the specifications on tolerances beyond the normal industry standards generally results in change order decisions in favor of the Contractor.

Ambiguities and Typographical Errors

Ambiguities in the specifications are usually the result of duplication, which is to be avoided. If there are two ways of reasonably interpreting your documents the courts will usually side against the preparer. The use of standard specifications will help reduce this category of problems.

Conflicts between Plans and Specifications

Specifications usually contain a clause in the general provisions establishing an order of precedence between the various contract document components. The usual order is:

1. Signed Contract
2. Other provisions such as special conditions
3. General provisions
4. Plans
5. Technical Specifications

To minimize conflicts between plans and specifications, it is important to avoid duplication. Avoid repeating the same information in plans and specifications. If an entire specifications section is in the plans, that section should be omitted entirely from the specifications. Construction contracts frequently contain a clause, which in effect says that anything mentioned in the specifications and not shown on the plans or shown on the plans and not mentioned in the specifications shall be interpreted as being shown or mentioned in both. The case of an item mentioned only in the specifications and not shown on the plans can lead to change orders on the basis that the Contractor had adequate information as to quality, but was unable to assess the cost of installing the item because its physical relation to other project components was not defined or was lacking. Leaving something out of the specifications that is shown on the plans leaves open the possibility for the Contractor to supply the cheapest possible alternative.

Inspection Requirements

Overly restrictive tolerances have been discussed above. Inspection or observation of the Contractors' work invariably creates some interference with the performance of the work by the Contractor. Frequency of tests and observations should be in line with the normal industry standards or the standard specifications. Failure to adhere to the industry norms may produce claims. Overzealous or inconsistent inspection, although not part of this general subject, also is a frequent cause for change order claims in this category.

Safety and Health Requirements

Failure to comply with local codes can result in lawsuits, charge-backs, or awards against the Design Engineer.

3.4.2 Product Identification, Traceability and Certification

1. Establish procedures for identifying materials and products, including documentation needed to verify the quality of products and materials, such as batch plant records, laboratory tests, catalog cuts and any other documentation.
2. Comply with storage requirements to prevent deterioration of materials and products at the work site including preventive maintenance while in storage.
3. Comply with requirements for identification, rejection, or segregation substandard or non-acceptable materials or products.
4. Comply with requirements for identification of certified materials – identification of the status of tests and inspections for incorporated materials, including specified marking, tagging, and stamping and/or physical isolation.

3.4.3 Inspection and Testing

The contract plans and specifications should be checked for any testing requirements, sampling frequency, acceptance criteria and tolerances. Easy checklists should be developed to assist the Construction Inspectors in assessing conformance to all testing requirements and to insure proper record keeping. Examples of checklists are provided in Section 3.5.1. Following are the steps required to develop QC procedures for testing and inspection:

1. Study the Plans and Specifications to identify all testing and inspection requirements for the project.
2. Assemble relevant contract documents needed to determine standards to be met for each test or inspection.
3. Develop any necessary checklists and train inspection staff.
4. Monitor for compliance to specified standards to be met according to the plans and specifications for:
    a. In-process tests and inspections
    b. In-plant tests and inspections
    c. Receiving inspections
    d. Final testing and inspections
5. Record results of required tests, inspections and observations in a timely manner on standard forms.

3.4.4 Maintenance of Measuring and Testing (M&T) Equipment

1. Establish a calibration and maintenance program for all M&T equipment used at the work site under the control of the field staff. Program may include specific contractual requirements, industrial or national standards and guidelines, or M&T equipment manufacturer's recommendations.
2. Document actions taken to calibrate and maintain testing equipment used and controlled by the worksite inspection staff.
3. Obtain acceptable calibration and maintenance documentation for testing Subcontractor's M&T equipment.

3.4.5 Certification of Tradesmen

Monitor compliance for contractual requirements relating to the qualifications of tradesmen performing project work, such as specified licenses and certifications. Monitor for compliance with mandated training programs for the Contractor's staff.

3.4.6 Identification of Non-Conforming Work

1. Review for compliance with specified procedures for identification and documentation of non-conforming work.
2. Evaluate and resolve remedial actions according to the options allowed in the plans and specifications such as:
   a. Reworking to meet requirements;
   b. Acceptance of work with or without repair; and
   c. Use of materials or products at an alternative application or location.

3.4.7 Implementation of Corrective Actions

Initiate and monitor corrective actions as governed by the applicable provisions of the plans and specifications:

1. Monitor corrective actions for effectiveness.
2. Pro-actively investigate causes for non-conformance and formulate remedial or alternative processes to prevent recurrences.
3. Implement and document process changes resulting from corrective actions.

3.4.8 Shop Drawings

Implement specified procedures for the handling of shop drawings. Monitor and facilitate the timely review of these documents by the proper party, including documentation of the process.

3.4.9 As-Built or Record Plans

Monitor the performance of contractual requirements for compiling and maintaining a current and updated Record Set of contract drawings and specifications. The Record Set may be compiled and produced by the Contractor or by the Owner's representative field staff, as contractually specified. Identify the reason for field changes in a separate record, and document any time or cost implications.

3.4.10 Record Keeping Considerations

Sufficient documentation and records shall be accumulated to provide objective evidence that the construction process was performed in accordance with accepted engineering practice, as well as in conformance to contractual requirements. The documentation should include not only the final design documents, such as drawings and specifications, but also all construction records and any communications, instructions and directives that have a direct bearing on the project.

A record keeping system should be established prior to starting the project. The system as a minimum should be capable of:

1. Organize project files according to a mandated file index system or one developed for the particular project. Maintain the filing system to permit the timely and accurate retrieval of documents.
2. Establish and maintain separate files for documents to indicate the compliance with the QC system for the project. Records that document adherence to the provisions of the plans and specifications include:
    a. Inspection logs, Daily Inspector's Reports and diaries
    b. Test data, including mill tests and certifications
    c. Qualification reports
    d. Validation and calibration reports
    e. Material review reports
    f. Batch plant records
3. Prepare correspondence on a timely basis. Log incoming and outgoing correspondence. Log general complaints from the public and document environmental issues arising from the general public and governmental agencies. Log pending or follow-up correspondence.
4. Identify the receiving organization for project records and files at completion of project. Establish retention time for project files to be retained.
5. Maintain at the work site required publications, documents and other materials referred to in the plans and specifications needed to properly understand and carry out the work scope, and to comply with the requirements of the Owner, and of those of regulatory entities and standard-setting associations.

3.5 PREPARATION OF PROJECT-SPECIFIC QA CHECKLISTS

After the project-specific responsibilities and duties have been identified, they can be subdivided into the following broad categories:

1. Staffing and Staff Qualifications
2. Contract documents
3. Project Files
4. Project Start-up Requirements
5. Miscellaneous contractual Requirements
6. Claims and Change Orders
7. Schedule Monitoring
8. Estimates and Payments
9. Construction Close-Out
10. Daily Inspection Reports
11. Materials and Materials Certification: General
12. Materials and Materials Certification: Concrete

For each of the first 12 subdivisions, a pro-forma Quality Assurance Program (QAP) checklist has been prepared listing those concerns generally of relevance in assessing the quality of services the Contractor is providing.

The Resident Engineer and the chief or senior Construction Inspector in conjunction must review these draft QAP checklists with the Contract documents for the Construction Related Services. Any additional specific responsibilities imposed on the Contractor in these contract documents should be added to the draft QAP checklist items listed, under the appropriate subdivision. Blank spaces have been provided on each subdivision to “customize” or adapt the draft QAP checklist to the demands of the specific project. Add additional sheets, as needed.

The draft Checklists for Materials and Materials Certification, in particular, are very general and are limited to just a few of the standard technical construction specifications divisions. These Checklists must also be augmented with the quality requirements for those materials in the project’s particular contract specifications. Additional checklists should be prepared for each of the major specifications divisions which are part of the particular contract and which require staff involvement in certifying, observing, inspecting, testing or assessment of results.

To allow further flexibility in the arrangement of checklists, the Resident engineer may wish to reorganize the checklists in accordance with the applicable paragraphs of the General and Special Provisions of his/her particular job.

Some Owners, mostly state DOTs, require their Construction Services providers to formulate their own Quality Assurance Plan. Such QA Plans are based on specific guidelines required by the Owner and are generally similar to the QAP checklists that follow.

3.5.1 QA Checklists for FRP Construction

A comprehensive QA and QC program implemented and monitored by the FRP material suppliers and the FRP installation Contractors should be maintained in order to insure quality repair. The QC is the direct responsibility of the Contractor and should cover all aspects of the strengthening project depending on the size and complexity of the project at hand. The quality assurance during construction is the responsibility of the Owner and can be achieved through a set of inspections, measurements, and applicable tests as specified in the construction specifications. The QAP checklists provided in this section address the most important parameters in the application of FRP systems. These checklists are offered as examples and are designed to assist the Owners in developing their QA requirements. They follow the Construction Specifications for Bonded Repair and Retrofit of Concrete Structures using FRP Composites, and

3.5.1.1 Project Start up Requirements (QAP 1 to 6)

Prior to starting the construction all Contractor shop drawings should be reviewed in light of the design plans and specifications to insure adherence to the contract documents. Any perceived conflicts should be resolved prior to starting the work. The Contractor should submit a material certification and identification of all the FRP materials to be used. The quantity, location and orientation of all FRP reinforcing materials to be used should be specified. The Owner should check if the Project is adequately staffed based on the complexity of the job and the approved construction schedule. The qualification of Contractor Personnel should be evaluated to insure that they meet the skills, ability and experience necessary for FRP strengthening projects.

3.5.1.2 Material Qualification and Acceptance (QAP 7)

The FRP materials should be qualified on the basis of the plans and performance specifications requirements. The Contractor should provide information demonstrating that the proposed FRP material meets all design and specifications requirements such as tensile strength and modulus, durability, bond strength, and glass transition temperature. Performance tests on the supplied materials should be performed according to the QC test plan and should meet the requirements specified in the Engineer's performance specifications. These tests may include measuring parameters such as the tensile strength and modulus, glass transition temperature, gel time, pot life, as well as the adhesive shear strength. The results from independent tests of the FRP constituent materials and laminates fabricated with them should be submitted by the Contractor for approval prior to starting the work. Material property information supplied by the manufacturer or material supplier could form basis for acceptance of the FRP materials if no testing requirements are stated in the construction specifications.

3.5.1.3 Removal of Defective Concrete and Restoration of Concrete (QAP 8)

The work under this section consists of restoring delaminated, or otherwise deteriorated, concrete on selected elements using polymer/latex modified concrete. Concrete restoration shall include the removal of all delaminated concrete from the area to be restored and an additional one to two inches from behind the reinforcement in delaminated areas. Any loose concrete remaining in the damaged region must be removed, leaving the member with sound concrete. Surfaces where the CFRP system is to be applied must be sound. Concrete spalls and delaminations must be repaired according to the procedure identified in the Plans and Specifications.

3.5.1.4 Inspection of Concrete Substrate (QAP 9)

The concrete surface should be inspected before the application of the FRP material. The surface should be prepared in accordance with Engineer's specifications. The concrete surface should be examined for surface smoothness or roughness, holes, cracks, corners, and other imperfections.

3.5.1.5 Application conditions (QAP 10)

The ambient temperature, concrete surface, surface dryness should conform the Engineer's specifications. FRP application should be halted if rain appears to be imminent. If rain is threatening after starting the application process, the Contractor should be instructed to protect the installed areas against contact with surface moisture

3.5.1.6 FRP Application Process (QAP 11 to 13)

Special care shall be taken to keep all records on the quantity of mixed resin during a one-day period, the date and time of mixing, the mixture proportions and identification of all components, the ambient temperature, the humidity, and other factors affecting the resin properties. These records shall also identify the FRP sheet used each day, its location on the structure, the ply count and direction of application, and all other useful information.

Sample FRP plate specimens shall be fabricated according to a predetermined sampling plan, under the same ambient conditions and procedures used to apply the FRP material to the concrete surfaces. Performance tests on these FRP specimens may be conducted as needed. The evaluation of the relative cure of FRP materials can be performed by means of laboratory testing of sample plate specimens or resin samples using ASTM Standard D3418, or at the construction site, by physical observation of resin tackiness and hardness of work surfaces or retained resin samples. Visual inspection of fibre orientation and waviness may be required for specific FRP material systems since poor orientation infers misalignment of the entire system from the angles specified in the drawings. Fibre misalignments of more than 5^o from the specified angle (1/12 slope) may adversely affect the provisional performance of the FRP reinforcement and should be reported to the Engineer. Non-compaction of fiber sheets when multiple plies are applied can result in significant voids, sagging and local areas of debonding, all of which will substantially affect the overall performance of the FRP system, and should be reported to the Engineer. Additional information is provided for pre-cured and near-surface mounted FRP systems.

3.5.1.7 Identification of Defective Work (QAP14)

The inspection program should cover such aspects as the presence and extent of delaminations, the cure of the installed system, adhesion, laminate thickness, fibre alignment and material properties.

3.5.1.8 Post-Application - Quality Control Tests (QAP15)

An inspection of the FRP repair system should be conducted after the full cure. Delaminations if detected should be evaluated considering their size and number relative to the overall application area, as well as their location with respect to structural load transfer. The inspection methods may include visual assessment acoustic sounding (hammer sounding), ultrasonics, thermography. Tension adhesion testing of cored samples should be conducted using known methods such as those described in ACI 503R or ASTM D4541. The sampling frequency should conform to the Engineer's specifications. Cored samples required for adhesion testing can also be used to determine the laminate thickness or number of plies. Approved methods to repair FRP materials having some delaminations may be used depending upon the size, the number of delaminations and their locations. These repairs should be performed in accordance to the Engineer's specifications. The laminate should then be re-inspected following delamination repairs, and the resulting delamination maps or scan compared with that of the initial inspection to verify whether the repair was properly accomplished. All inspection records and test results related to the FRP material should be retained. It should include delamination/repair, on-site bond tests, anomalies and correction reports as well as all mechanical/physical test results from the designated laboratories.

3.5.1.9 General Job Administration (QAP 16 to 20)

QAP 14 to 18 address general job administration conditions such us claims and change, orders, schedule monitoring, estimates and payments and daily inspectors reports (DIR).
The following checklists are pro-forma and must be modified to the particular project.

FORM No. QAP 1
Project Start up Requirements

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Have the Construction Contract Documents been preliminarily reviewed for their overall completeness, obvious errors and omissions, constructability, etc.?
_____	_____	_____	a. Have the provisions been reviewed with the field staff?
_____	_____	_____	b. Have the Design Engineer and the Owner been advised of findings?
_____	_____	_____	2. Have the Contract Documents for Construction Related Services between the Consultant and the Owner been reviewed against the Construction Contract Documents to identify possible conflicts in imposed duties and responsibilities?
_____	_____	_____	a. Have all perceived conflicts been resolved?
_____	_____	_____	3. Has a pre-construction meeting been held?
_____	_____	_____	a. Is the agenda for the pre-construction meeting in accordance with the Owner's requirements?
_____	_____	_____	b. Are minutes of the meeting in the files?
_____	_____	_____	4. Has Contractor submitted all required documents on time:
_____	_____	_____	Insurance certificates?
_____	_____	_____	Bonds?
_____	_____	_____	Construction Inspector qualifications?
_____	_____	_____	Permits?
_____	_____	_____	Equipment calibration?
_____	_____	_____	Quality Control Plans?
_____	_____	_____	Material Safety Data Sheets?
_____	_____	_____	Schedules?
_____	_____	_____	Certificate(s)?
_____	_____	_____	5. Have accident and emergency reporting procedures and documentation been established and reviewed with the field staff?
_____	_____	_____	6. Are all forms needed to document the construction processes and quality of the work on hand at the start of the project?
_____	_____	_____	7. Is the project fully equipped with necessary field measuring and testing (M&T) equipment?
_____	_____	_____	8. Is the measuring and testing (M&T) equipment periodically re-calibrated according to the manufacturers recommendation?
_____	_____	_____	9. Are calibration documents on file?

Remarks:
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Reviewer/Date: ______________________________
Resident Engineer/Date: ______________________________

FORM NO. QAP 2
Contract Documents

Project No: ________________ Project Name: ____________________________________

YES	NO	N/A	
_____	_____	_____	1. Is a complete set of contract documents available?
_____	_____	_____	a. Plans?
_____	_____	_____	b. Specifications?
_____	_____	_____	c. Other?
_____	_____	_____	2. Are all design changes and amendments incorporated in these documents?
_____	_____	_____	3. Are all field changes incorporated in the documents?
_____	_____	_____	4. Are “As-Built” plans being updated to reflect field revisions?
_____	_____	_____	5. Have all design and field changes been signed and sealed by the Design Engineer?
_____	_____	_____	6. Have all design and field changes been included and approved in the Change Orders?
_____	_____	_____	7. Are shop drawings being logged and tracked?
_____	_____	_____	8. Are all support documents required or referenced in the CE&I contract available on site?

Remarks:

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Reviewer/Date: ____________________________________

Resident Engineer/Date: ____________________________________

FORM NO. QAP 3
Specifications Review Checklist

Project No: _________________ Project Name: _______________________________________

3.5.1.10

YES NO N/A

_____ _____ _____ 1. Is the specifications complete and clear to the extent necessary to properly specify construction and performance requirements?

_____ _____ _____ 2. Have duplications or inconsistencies between contract drawings and the specifications been eliminated?

_____ _____ _____ 3. Are proper codes, standards, processes, etc. referenced?

_____ _____ _____ 4. Are requirements for shop drawings properly specified, both as to content and timely submission?

_____ _____ _____ 5. Are new materials employed and installed in the manner approved by the manufacturer?

_____ _____ _____ 6. Is proper test and inspection documentation specified?

_____ _____ _____ 7. Are the acceptance criteria tests (tolerances, etc.) specified, and are they adequate, realistic, and in line with the ordinary construction practice?

_____ _____ _____ 8. Are provisions made for the qualification and approval of special construction processes and for the personnel performing these processes?

_____ _____ _____ 9. Are measuring and test equipment calibration requirements and cleaning, storage and handling requirements properly specified?

_____ _____ _____ 10. Are measurements units and basis of payment properly specified?

_____ _____ _____ 11. Is nomenclature used in the specifications exactly as used on the contract drawings?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 4

Drawing Review Checklist

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A		
_____	_____	_____	1.	Is the scope of the set of contract drawings satisfactory?
_____	_____	_____	2.	Do the structures, equipment or components satisfactorily meet the functional needs and requirements?
_____	_____	_____	3.	Has accessibility for maintenance, repair and in-service inspection been provided?
_____	_____	_____	4.	Are materials properly identified on the contract drawings?
_____	_____	_____	5.	Are the items constructible as shown? Has the normal sequencing of construction trades been followed?
_____	_____	_____	6.	Is construction phasing or staging clearly shown?
_____	_____	_____	7.	Are dimensions and tolerances correct and consistent?
_____	_____	_____	8.	Have duplications and redundancy of information, data and dimensioning been eliminated?
_____	_____	_____	9.	Are the Plans signed and sealed by a Professional Engineer?
_____	_____	_____	10.	Are the drawings legible and reproducible?
_____	_____	_____	11.	Do the titles and drawing numbers agree with the cover sheet list of the drawings?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 5
Staffing and Staff Qualification

Project No: __________________ Project Name: ____

YES	NO	N/A	
_____	_____	_____	1. Is the Project adequately staffed based on its complexity and the approved construction schedule?
_____	_____	_____	2. Are project personnel on the job site during Contractors operations?
_____	_____	_____	3. Are the staff members properly trained and informed regarding:
_____	_____	_____	a. Authority, responsibilities and duties of the Construction Inspector.
_____	_____	_____	b. General rules of project safety.
_____	_____	_____	c. Hazard Communication Employee Training Program.
_____	_____	_____	d. Specific technical inspection and testing requirements.
_____	_____	_____	e. Emergency and Accident procedures.
_____	_____	_____	4. Are the names and qualifications of the Contractor staff on file?
_____	_____	_____	5. Do Contractor staff members have required professional or technical accreditation?
_____	_____	_____	a. Has this been verified?
_____	_____	_____	6. Does the Contractor's staff include a qualified provider of First Aid services?

Remarks:

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Reviewer/Date: ________________________________

Resident Engineer/Date: ________________________________

FORM NO. QAP 6

Miscellaneous Contractual Provisions

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A		
_____	_____	_____	1.	Is a copy of the fully executed Bid Blank in the Prime Contractors file?
_____	_____	_____	2.	Is there an EEO compliance checklist on file?
_____	_____	_____	3.	Have the Contractor's EEO Policy, Affirmative Action and DBE Affirmative Action Plans been submitted?
_____	_____	_____	4.	Have the monthly EEO reports been filed?
_____	_____	_____	5.	Have all needed permits been applied for by the Owner?
_____	_____	_____		a. By the Contractor?
_____	_____	_____		b. Are copies of all required permits in the files?
_____	_____	_____		c. Are the conditions of each permit being adhered to?
_____	_____	_____	6.	Are environmental permits or environmental control plans required? If yes, are they included in the submittal package?
_____	_____	_____	7.	Has a Traffic Control Plan been specified for this project?
_____	_____	_____	8.	Have local law enforcement agencies been notified by the Contractor of the provisions of the Traffic Control Plan?
_____	_____	_____	9.	Has the Contractor submitted names and telephone numbers of the emergency contact personnel to all agencies involved?
_____	_____	_____	10.	Has the Contractor submitted evidence of required bonding and insurance?
_____	_____	_____	11.	Are meetings with the Owner on a scheduled basis?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 7
Material Qualification and Acceptance

Project No: __________________ Project Name: ____

YES	NO	N/A	
_____	_____	_____	1. Have all required samples been collected and submitted?
_____	_____	_____	2. Are all certified mill analyses and third-party test results on file?
_____	_____	_____	3. Are all material acceptance requirements being met?
_____	_____	_____	4. Are the materials that have failed testing requirements disposed of according to the contract requirements?
_____	_____	_____	5. Are "or equal" materials and equipment submitted by the Contractor approved by the Design Engineer?
_____	_____	_____	6. Are all certified materials properly identified according to the contract requirements?
_____	_____	_____	7. Have all relevant documents that are needed to determine if the standards are met provided to the inspection staff?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ________________________________

FORM NO. QAP 8

Removal of Defective Concrete and Restoration of Section

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Did the perimeters of existing spalls have been identified and sawcut to a minimum depth of ¾ of an inch to prevent feathered edges?
_____	_____	_____	2. Are the limits of concrete removal for each member identified in the plans?
_____	_____	_____	a) If yes, did the Contractor remove any concrete beyond the identified areas?
_____	_____	_____	b) Did the Contractor obtain the Engineer approval to remove concrete beyond the identified areas?
_____	_____	_____	3. Are Cracks within solid concrete greater than 0.25mm (0.01 in) have been epoxy injected?
_____	_____	_____	4. After removal of all defective areas, did the Contractor inspect and clean the substrate from any dust, laitance, grease, oil, curing compounds, wax, impregnations, foreign particles and other bond inhibiting materials?
_____	_____	_____	5. Has all exposed steel been sandblasted clean near white appearance prior to concrete placement?
_____	_____	_____	6. Was Mechanical anchorage of the repair material with the substrate specified?
_____	_____	_____	a) If yes, were they installed according to Specifications?
_____	_____	_____	7. Did the Contractor apply a bonding and reinforcement protection to all exposed reinforcement and concrete surface prior to concrete placement?
_____	_____	_____	8. Did the Contractor use the approved material and method of application including manufacture's technical specifications and formulation if applicable?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 9

Inspection - Surface Preparation

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Is the restored concrete surface smooth, uniform and matching the concrete component's original profile?
			a) If No, were the deviations less than 0.8 mm (1/32 in)?
_____	_____	_____	2. Did the Contractor remove form lines and sharp edges by grinding or filling with putty?
_____	_____	_____	3. Have all inside and outside corners and sharp edges been rounded or chamfered to a minimum radius of 25 mm (1 in)?
_____	_____	_____	4. Are there any voids or depressions with diameters larger than 12.7 mm (½ in) or depths greater than 3.2 mm (1/8 in), when measured from a 305 mm (12 in) straight edge placed on the surface?
_____	_____	_____	a) If yes, have surface depressions and voids been filled and cured according to Specifications?
_____	_____	_____	5. Have all cracks in the surface of concrete or the substrate wider than 0.25 mm (0.01 in) been filled using pressure injection of epoxy in accordance with the procedures outlined in Specifications?
_____	_____	_____	a) Was any surface roughness resulting from crack injection alleviated according to Specifications?
_____	_____	_____	6. Was the surface checked and cleaned of any dust, laitance, grease, oil, curing compounds, wax, impregnations, surface lubricants, paint coatings, stains, foreign particles, weathered layers and any other bond inhibiting materials?
_____	_____	_____	7. Was the final preparation of the all surfaces receiving FRP performed according the Specifications?

Remarks:

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Reviewer/Date: ______________________________________

Resident Engineer/Date: ______________________________________

FORM NO. QAP 10

Application Conditions

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Is the ambient temperature and temperature of concrete surface within the range of 50^{o}F-95^{o}F, or as specified by the manufacturer?
_____	_____	_____	2. Are the contact surfaces completely dry at the time of installation of FRP system?
_____	_____	_____	a) Was the moisture level measured using a mortar moisture meter?
_____	_____	_____	b) Was the moisture level less than 10% or the specified limit?
_____	_____	_____	3. Does the rain appear to be imminent?
_____	_____	_____	a) If yes, stop application of the material until dry conditions are assured.
_____	_____	_____	4. If rain is threatening after starting the application process, instruct the Contractor to protect the installed areas against contact with surface moisture.

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 11

FRP Application Process (Wet Lay-Up Systems)

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
			1. *Resin Mix*
______	______	______	- Is the resin prepared according to the mix ratio and procedures recommended by the manufacturer until thorough mixing with uniform color and consistency is achieved?
______	______	______	- Is the resin diluted with organic solvents? (NOT allowed)
______	______	______	- Is the resin mixed in quantities sufficiently small to ensure its use within manufacturer recommended pot life?
______	______	______	- Is the excess resin disposed of when exceeded its pot life, or began to generate heat or show signs of increased viscosity?
			2. *Primer and Putty*
______	______	______	- Is the primer applied uniformly to penetrate all surface pores of concrete substrate where the FRP system is to be installed?
______	______	______	- Does the rate of application of primer follow the manufacturer's recommendations?
______	______	______	- Are the ambient and concrete surface temperatures as specified in the contract drawings and recommended by the manufacturer?
______	______	______	- Is the excess primer disposed of when exceeded its pot life?
______	______	______	- Is the putty, if necessary, applied as soon as the primer becomes tack-free or until non-sticky to the fingers?
______	______	______	- In case of delays longer than 7 days, is the surface of primer cleaned and prepared for the putty, if necessary?
______	______	______	- Does the applied putty meet the surface profile according to the contract drawings?
______	______	______	- Is the excess putty, if used, disposed of when exceeded its pot life?
______	______	______	- Are the surfaces of primer and putty protected from dust, moisture and other contaminants before applying the FRP system?
			3. *Fabric Saturation and Placement*
______	______	______	- Is the saturant applied uniformly on all surface areas of concrete where the FRP system is to be installed?
______	______	______	- Is the viscosity of the saturant sufficiently low, according to the manufacturer recommendations, to fully impregnate the fiber sheets?
______	______	______	- Does the rate of application of saturant follow the manufacturer recommendations?
______	______	______	- Are the ambient and concrete surface temperatures as specified in the contract drawings and recommended by the manufacturer?
______	______	______	- Is the excess saturant disposed of when exceeded its pot life?
______	______	______	- Is the fiber sheet cut to the length specified in the contract drawings (typically in segments shorter than 4.6 to 6.1 m (15 to 20 ft))?
______	______	______	- Is the fiber sheet placed properly and pressed gently onto the wet saturant within its pot life?
______	______	______	- Is any entrapped air between fiber sheet and concrete released?
______	______	______	- Is rolling in the fiber direction for unidirectional fiber sheets?

YES	NO	N/A	
			- Is rolling in the fill direction end to end, and then the warp direction, for bi-directional fiber sheets?
			- Is there excessive force or sharp metal rollers involved that could damage the fibers? (NOT allowed)
			- Is sufficient saturant applied on top of the fiber sheet as overcoat to fully saturate the fibers?
			- Is there any interruption in the application of undercoat, fiber sheets, and overcoat?
			- Is the above sequence repeated properly for each additional fiber sheet, with an overcoat resin 15%-20% greater than a single ply?
			- Is each new fiber sheet applied before the onset of complete gelation of the previous layer?
			- Does the number of plies applied in a single day follow the contract drawings and the manufacturer recommendations?
			- In case of several days of delay between plies, is the surface of previously cured layers of the FRP system prepared properly before applying new fiber sheets?
			4. *Splice and Overlap*
			- Are lap joints constructed when there is an interruption in the direction of the fibers?
			- Are lap splice lengths as specified in the contract drawings, but at least 152 mm (6 in),?
			- Are lap splices staggered on multiple plies and adjacent strips, unless permitted in the contract drawings?
			- Are all lap joints in the fiber directions made in a single day?
			- Is there any lap joint in the transverse direction, if specified in the contract drawings?
			5. *Fiber Orientation*
			- Are the fibers aligned on the structural member according to the contract drawings?
			- Is there any deviation in fiber alignment more than 5^{o}? (NOT allowed)
			- Are fibers free of kink, folds and waviness?
			6. *Anchoring of FRP Sheets*
			- Is anchorage for the FRP sheets installed according to the contract drawings, and in such a way to avoid damage to fibers or concrete?
			7. *Stressing Applications*
			- Are stressing hardware and procedures according to the contract drawings and the manufacturer recommendations?
			- Does the grouting pressure according to the contract drawings?
			8. *Curing and Final Coating*
			- Is the FRP system allowed to cure according to contract drawings and the manufacturer recommendations?
			- Is the resin chemistry field modified for rapid curing? (NOT allowed)
			- Is elevated temperature used for curing follows the contract drawings and manufacturer recommendations?
			- Is the FRP system protected until it is fully cured?
			- Is the FRP system under full load before it is fully cured?
			- Is continuous pressure applied, if necessary, for the cure of FRP system?

YES	NO	N/A	
______	______	______	- Is the surface of FRP system prepared according to the contract drawings and the manufacturer recommendation to receive coating?
______	______	______	- Are solvent wipes used for surface cleaning? (NOT allowed)
______	______	______	- If abrasive cleaning is necessary, is the air pressure at the nozzle limited to avoid any damage to fibers?
______	______	______	- Is the thickness of protective coating for the FRP system as specified in the contract drawings and specifications?
______	______	______	- Does the final appearance match the color and texture of the adjacent concrete?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 12

FRP Application Process (Pre-Cured Systems)

Project No: ________________ Project Name: ____________________________________

YES	NO	N/A	
			1. *Application of Adhesives*
______	______	______	- Is the adhesive prepared according to the mix ratio and procedures recommended by the manufacturer until thorough mixing with uniform color and consistency is achieved?
______	______	______	- Is the adhesive applied uniformly on all surface areas of concrete substrate where the pre-cured FRP system is to be applied?
______	______	______	- Does the rate of application of adhesive follow the manufacturer's recommendations?
______	______	______	- Are thickness and viscosity of the adhesive layer according to the manufacturer's recommendations?
______	______	______	- Are the ambient and concrete surface temperatures as specified in the contract drawings and recommended by the manufacturer?
______	______	______	- Is excess resin that has exceeded its pot life disposed of?
			2. *Placement of Pre-Cured System*
______	______	______	- Is the pre-cured FRP system clean?
______	______	______	- Is the pre-cured FRP system cut to the length specified in the contract drawings?
______	______	______	- Are manufacturer's recommendations on the timing and sequence of stacking, overlap and banding, horizontal and vertical joints, staggering of splices and overlap and butt joints followed?
______	______	______	- Is the pre-cured FRP system placed in the wet adhesive within its pot life?
______	______	______	- Is entrapped air between laminate and concrete released?
______	______	______	- Is excess adhesive between laminate and concrete removed?
______	______	______	- Is the FRP system left undisturbed until the adhesive fully cures?
			3. *Anchoring of Pre-Cured System*
______	______	______	- Are permanent anchorage for the FRP system properly installed according to the contract drawings?
______	______	______	- Are temporary clamping and shoring for the FRP system properly installed according to the contract drawings?
			4. *Grouting of Pre-Cured Shells*
______	______	______	- Is the pre-cured FRP shell around concrete column grouted at least 24 hours after installation?
______	______	______	- Does pressure grouting follow the contract drawings and the manufacturer recommendations?
______	______	______	- Does the grout have a shrinkage strain of less than 0.0005 and a compressive strength greater than 27.6 MPa (4,000 psi)?
			5 *Stressing Applications*
______	______	______	- Are stressing hardware according to the contract drawings and the manufacturer recommendations?
______	______	______	- Are the stressing procedures followed according to the contract drawings and the manufacturer recommendations?

YES	NO	N/A	
			6. *Curing and Final Coating*
______	______	______	- Is the FRP system allowed to cure according to contract drawings and the manufacturer recommendations?
______	______	______	- Is the resin chemistry field modified for rapid curing? (NOT allowed)
______	______	______	- Is elevated temperature used for curing follows the contract drawings and manufacturer recommendations?
______	______	______	- Is the FRP system protected until it is fully cured?
______	______	______	- Is the FRP system under full load before it is fully cured?
______	______	______	- Is continuous pressure applied, if necessary, for the cure of FRP system?
______	______	______	- Is the surface of FRP system prepared according to the contract drawings and the manufacturer recommendation to receive coating?
______	______	______	- Are solvent wipes used for surface cleaning? (NOT allowed)
______	______	______	- If abrasive cleaning is necessary, is the air pressure at the nozzle limited to avoid any damage to fibers?
______	______	______	- Is the thickness of protective coating for the FRP system as specified in the contract drawings and specifications?
______	______	______	- Does the final appearance match the color and texture of the adjacent concrete?

Remarks:

Reviewer/Date: ______________________________

Resident Engineer/Date: ______________________________

FORM NO. QAP 13

FRP Application Process (Near-Surface Mounted Systems)

Project No: ________________ Project Name: ____________________________________

YES	NO	N/A	
			1. *Application of Embedding Paste*
______	______	______	- Is the embedding paste prepared according to the mix ratio and procedures recommended by the manufacturer until thorough mixing with uniform color and consistency is achieved?
______	______	______	- Are all grooves, where the FRP system is to be placed, half filled with the paste?
______	______	______	- Are voids between concrete substrate and the embedding paste removed?
______	______	______	- Are the ambient and concrete surface temperatures as specified in the contract drawings and recommended by the manufacturer?
______	______	______	- Is the excess paste disposed of when exceeded its pot life?
			2. *Placing FRP Reinforcement*
______	______	______	- Is the FRP bar or strip clean?
______	______	______	- Is the FRP bar or strip cut to the length specified by contract drawings?
______	______	______	- Is there any shearing of FRP bar or strip? (NOT allowed)
______	______	______	- Is the FRP bar or strip placed at mid-depth of the groove and lightly pressed to force the paste to flow around it?
______	______	______	- Is the grove fully filled with additional paste and then leveled?

Remarks:

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Reviewer/Date: ________________________________

Resident Engineer/Date: ________________________________

FORM NO. QAP 14

Identification of Defective Work

Project No: __________________ Project Name: ____

YES	NO	N/A	
_____	_____	_____	1. Did you find any voids and air encapsulation between the concrete and the layers of primer, resin and/or adhesive, and within the composite itself?
_____	_____	_____	2. Are there any delaminations between layers of composite fabric?
_____	_____	_____	3. Are there any broken or damaged edges of the composite?
_____	_____	_____	4. Is there any wrinkling and buckling of fiber and fiber tows?
_____	_____	_____	5. Are there any discontinuities due to fracture of fibers, breaks in the fabric, or cracks in prefabricated material?
_____	_____	_____	6. Are there any cracks, blisters or peeling of the surface coating?
_____	_____	_____	7. Are there any resin starved areas or areas with non-uniform impregnation/wet-out?
_____	_____	_____	8. Are there any under cured or incompletely cured polymer?
_____	_____	_____	9. Are there any incorrectly placed reinforcement configurations?

Remarks:

Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 15

Post-Application - Quality Control Tests

Project No: __________________ Project Name: ____

YES	NO	N/A		
_____	_____	_____	**1.**	**Inspection for Debonding** Perform surface inspection for any swelling, bubbles, voids or delaminations after at least 24 hours for initial resin cure.
_____	_____	_____		Is the presence of voids and air pockets suspected? If yes, - Perform an acoustic tap test with a hard object to identify delaminated areas by sound. - Mark all voids and assess them in term of size. - Repair Voids in accordance to the procedures established in the contract drawings and specifications.
_____	_____	_____	**2.**	**Inspection for Adhesion** Perform direct pull-off test according to ASTM D4541 or ACI 503R-93 after at least 24 hours for initial resin cure.
_____	_____	_____		Are test locations representatives and on flat surfaces?
_____	_____	_____		Are the number of tests performed in accordance to the number established in the contract drawings and specifications?
_____	_____	_____		Is the observed failure mode of the core specimen cohesive within concrete? Failure at the bond line at tensile stress below 1.38 MPa (200 psi) is unacceptable.
_____	_____	_____		Repair concrete area after bonding test according to the procedures established in the contract drawings and specifications.

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 16
Claims and Change Orders

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A		
_____	_____	_____	1.	Have any claims been made to date?
_____	_____	_____	2.	Has the Contractor provided written notification of all claims?
_____	_____	_____	3.	Did the notifications include specifics of the claims?
_____	_____	_____	4.	Did the Resident Engineer acknowledge each claim?
_____	_____	_____	5.	Is a separate file maintained for each claim?
_____	_____	_____	6.	Is each claim being processed and tracked according to the requirements of the specifications?
_____	_____	_____	7.	Has the Resident Engineer reviewed each claim, documented findings, and made a recommendation for resolution?
_____	_____	_____	8.	Have Change Orders been issued for satisfactorily resolved claims? Number _______ Est. Value: $_________
_____	_____	_____	9.	Does any resolved Change Order affect scope of work or lengthen the contract time?
_____	_____	_____	10.	Are there any currently unresolved claims? Number _______ Est. Value: $_________
_____	_____	_____	11.	Are there any anticipated claims?
_____	_____	_____	12.	Are there any resolved Change Orders as a result of field conditions? Number _______ Est. Value: $_________
_____	_____	_____	13.	Are there any pending Change Orders as a result of field conditions? Number _______ Est. Value: $_________
_____	_____	_____	14.	Are there any Change Orders related to the extra work authorized by the Owner? Number _______ Est. Value: $_________

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ____

FORM NO. QAP 17
Schedule Monitoring

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Has the Contractor's Work Schedule been approved?
_____	_____	_____	2. Does the Contractor's Work Plan match the established Schedule?
_____	_____	_____	3. Is the Resident Engineer meeting with the Contractor on a regular basis to verify and update the Work Plan and Schedule?
_____	_____	_____	4. What is the current status of the Contract?
			Contract Time Used: _______%, as of Date: ________
			Work Completed: ________%.
_____	_____	_____	5. Was the "Notice to Proceed" issued in accordance with stipulations of the Specifications?
_____	_____	_____	6. Has the Contractor asked for time extensions to the contract?
_____	_____	_____	a. Are time extensions anticipated?
_____	_____	_____	7. Are time extension requests being processed according to the provisions of the Specifications?
_____	_____	_____	8. Are time extension requests based on weather delays in accordance with documented weather conditions in the DIR's?
_____	_____	_____	9. Is there a schedule slippage?
_____	_____	_____	a. Has the Owner been advised?
_____	_____	_____	b. Is there an impact on the schedule and cost?
_____	_____	_____	c. Has the Contractor formulated a "back-on-schedule" plan?
____	_____	_____	d. Have time extensions been granted? Number ______ Total time _____days

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ________________________________

FORM NO. QAP 18

Estimates and Payments

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Is there back-up documentation for all pay items?
_____	_____	_____	2. Are monthly payments for quantities in agreement with the Engineer's estimate of quantities for that month?
_____	_____	_____	3. Are items being paid for according to the method of measurement and basis for payment called for in the Specifications?
_____	_____	_____	4. Is the Mobilization Item being paid according to the provisions of the specifications?
_____	_____	_____	5. Is there a separate payment item for stockpiled materials?
_____	_____	_____	6. Are stockpiled materials re-verified in the following month to reconcile quantities with materials incorporated into the work?
_____	_____	_____	7. Do the records indicate when the stockpiled materials are incorporated into the work?
_____	_____	_____	a. Was a deduction made for any partial payment amount previously issued?
_____	_____	_____	b. Are equipment and materials storage conditions noted on DIRs?

Remarks:

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Reviewer/Date: ____

Resident Engineer/Date: ________________________________

FORM NO. QAP 19

Daily Inspectors Reports (DIR)

Project No: _________________ Project Name: ______________________________________

YES	NO	N/A	
_____	_____	_____	1. Are DIR's current?
_____	_____	_____	a. Are Contractor's equipment and labor force clearly documented on DIR's?
_____	_____	_____	b. Are Contractor's hours of work logged?
_____	_____	_____	2. Is the contract day/date correctly listed on the DIR's?
_____	_____	_____	3. Is the Owner's Project Number listed correctly?
_____	_____	_____	4. Is there a DIR for each Construction Inspector on site?
_____	_____	_____	5. Is the particular operation or location of work clearly identified?
_____	_____	_____	6. Are all work quantities shown for the work performed each day?
_____	_____	_____	7. Are Subcontractor's activities clearly documented on respective DIR's?
_____	_____	_____	8. Is there a separate DIR for each utility, force account crew or DBE working on the project?
_____	_____	_____	9. Is the DBE Contractor identified as DBE on the DIR?
_____	_____	_____	10. Are weather conditions and delays adequately noted?
_____	_____	_____	11. Is the DIR signed and dated by the responsible supervisor?
_____	_____	_____	12. Are DIR's written in a concise, understandable and legible manner?
_____	_____	_____	13. Are delays on the project being specifically accounted for?
_____	_____	_____	14. Are accidents, injuries and damages described on the DIR's?
_____	_____	_____	15. Are unusual conditions noted? (high water, lane closures, icing, etc.).
_____	_____	_____	16. Are disputed items of work listed on the DIR?

Remarks: ____

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Reviewer/Date: ____

Resident Engineer/Date: ____________________________________

FORM NO. QAP 20

Construction Close-out

Project No: __________________ Project Name: ____

YES	NO	N/A	
_____	_____	_____	1. Have items on the final “punch list” been accepted and closed out?
_____	_____	_____	2. Has the final estimate been prepared including pending Change Orders?
_____	_____	_____	3. Have all pay item quantities on the final estimate been cross-referenced from source documents?
_____	_____	_____	4. Have the final record plans been completed in accordance with contractual requirements?
_____	_____	_____	5. Are all incorporated materials and equipment tested and certified according to the requirements of the contract?
_____	_____	_____	6. Have all contractual incentive/disincentive provisions been correctly applied and administered?
_____	_____	_____	7. Has the Resident Engineer followed all contractual requirements in accepting the project?
_____	_____	_____	8. Has the Owner’s staff completed any required inspections prior to final acceptance?
_____	_____	_____	9. Are all project files reviewed before transfer to the required receiver?

Remarks:

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Reviewer/Date: ______________________________________

Resident Engineer/Date: ______________________________________