THESIS

# EVALUATING THE BOND DURABILITY OF FRP-CONCRETE SYSTEMS SUBJECTED TO ENVIRONMENTAL EXPOSURES

Submitted by

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ABSTRACT

EVALUATING THE BOND DURABILITY OF FRP-CONCRETE SYSTEMS SUBJECTED TO ENVIRONMENTAL EXPOSURES

The poor current condition of transportation infrastructure in the U.S. is well documented. With traffic volumes on the rise, as well as limited funding available to maintain and rehabilitate aging bridges, cost effective means of improving the performance and durability of these structures must be employed. Fiber Reinforced Polymers (FRPs) offer one potential solution. Their use has been progressively growing in the field of civil engineering as the material's high strength to weight ratio, non-corrosive nature, and ability to conform to existing geometry make it appealing in the reinforcement of existing reinforced concrete structures.

In most applications of FRP to strengthen an existing structure, the FRP-concrete bond is essential. Bond is needed for proper transfer of stresses among interfaces. From a durability standpoint, the long-term bond performance is also a major concern. As a result, a long-term durability study was conducted in the laboratory to evaluate the behavior of the bond between the FRP and concrete. Small concrete specimens were prepared, reinforced with FRP material, and subjected to various environmental scenarios such as wet-dry cycles, freeze-thaw cycles, and constant immersion in water, as well as deicing agents. Direct tension pull-off tests and three-point flexural tests were conducted on these specimens to determine any degradation in bond strength over time. Finally, the pull-off test method was evaluated by means of previous research studies and recommendations about preparation procedures were made.

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# Chapter 1. Introduction

## 1.1 Background and Motivation

### 1.1.2 State of Bridge Infrastructure in the United States

America's transportation infrastructure is in need of attention. With traffic volumes on the rise, means of improving the performance and extending the service life of these structures are needed. Also, in today's economy and political climate, finding sufficient funding to efficiently maintain and rehabilitate bridges is a great challenge and available funds must be used wisely. According to ASCE, the total investment need on bridge infrastructure is $930 million over a 5-year period. With an estimated spending of only $380.5 million over this period, the projected shortfall results in $549.50 million, close to 60% of the total amount that is needed (ASCE, 2009). These figures give a strong idea of the limited funding present in this area, which makes maintenance and repair of these structures a much more difficult task to achieve.

Clearly bridges are used every day to offer drivers the ability to get places. Therefore, it is a matter of huge importance when the safety of citizens is at stake due to the deteriorating conditions of these assets. Take for example, the tragic 2007 collapse of the Interstate 35W Bridge in Minneapolis, which drew attention to the issues associated with aging infrastructure and the urgent need for rehabilitation of these assets. Not only did it cost millions of dollars to replace the bridge, but it caused deep sorrow and pain to the families of the people that lost their lives during the collapse.

In regard to the number of bridges in need of attention, the U.S Department of Transportation (USDOT) determined that out of the 600,905 bridges in the country, 14.8% (89,024) can be classified as functionally obsolete, and 12.1% (72,868) as structurally deficient.

This high percentage in need for repair gave bridge transportation infrastructure a “C” grade in 2009, according to the ASCE Report Card.

In the U.S, the majority of the bridges in service today were constructed in the 1930s, 1950s and 1960s (Rens et.al, 2005). With a current average bridge age of 43 years old (ASCE, 2009), many of these structures are in need of repair and rehabilitation. A national goal has been set to lower the 25% of structurally deficient and functionally obsolete bridges down to 15% by 2013. As a result, new repair alternatives have been researched with a target of extending the life of these existing structures. Many Departments of Transportation around the country, including the Colorado DOT have shown strong interest in the repair, rehabilitation, and reinforcement of bridges using externally-bonded fiber reinforced polymers (FRP).

### 1.1.3 Fiber Reinforced Polymers

FRP is a composite material made of a polymer matrix reinforced with fibers. These fibers can be made of glass, carbon, and other materials. Carbon is most common for externally bonded applications. Fiber reinforced polymers have a wide range of applications in industries such as aerospace, automobiles, sports equipment, and others. In addition, over recent years these composite materials have gained popularity in the field of civil engineering. FRP is applied on the existing concrete surface by means of epoxy adhesion and provides tensile reinforcement to the structural member. The material’s high strength to weight ratio, non-corrosive nature, and ability to conform to existing geometry make it very appealing in the reinforcement and repair of existing reinforced concrete structures. However, these advantages may become useless if preparation procedures are not carefully followed, including improper adhesion of the FRP onto the concrete surface.

In many applications of FRP to an existing structure, the FRP-concrete bond is essential for proper transfer of stresses among interfaces. If a structure is repaired by means of FRP reinforcement, long term bond performance from the durability standpoint is a major concern. Regardless of the FRP’s ability to resist high tensile loads, the reinforcement becomes useless if deterioration of the FRP material is witnessed over time, or if de-bonding from the concrete member takes place. Therefore, if FRP is to be used as means of external reinforcement, one must understand the long term performance of it and its bond to concrete.

## 1.2 Research Objectives

To better understand FRP materials as a reinforcement option on concrete structures, one must identify the long-term behavior of these materials when exposed to various environments. While many studies have considered durability of the material, for example, Shan et. al. (2010) and Bahari & Nasiri, (2008), fewer studies have investigated the durability of the bond. Field conditions such as rain, snow, heat, and deicing agents are some of the reasons why durability of a FRP-concrete bond must be studied and evaluated. Various laboratory test methods that assess the quality of the bond are available. While laboratory tests alone are not enough because of the difficulty to correlate accelerated aging to realistic field conditions, they are a good starting point. Therefore, two research objectives were targeted in this thesis:

1. To conduct a durability study in the laboratory to characterize the potential deterioration of a FRP-concrete bond under different environments.
2. To investigate the suitability of laboratory tests, more specifically direct tension pull-off tests, when characterizing the behavior of a FRP-concrete bond, and how these results can be interpreted in the long run.

## 1.3 Methods and Approach

A bond durability study was conducted in the lab using direct tension pull-off tests and three-point beam bending tests of FRP reinforced concrete specimens. A total of one-hundred pull-off tests were conducted following guidelines from ASTM Standard D7522. In addition, twenty-nine small beams, reinforced with FRP, were cast and tested in flexure using ASTM Standard C78 as a supporting guideline. Specimens were tested at three different times. More specifically, Stage 0 constitutes the control specimens, whereas Stage 1 and Stage 2 represent the tests conducted following six months and twelve months of exposure, respectively. These specimens were subjected to various environmental exposures such as wet-dry cycles, freeze-thaw cycles, and exposure to deicing agents, and then tested during their respective stage.

Since pull-off tests were chosen as the main test method for this research, a thorough study regarding this method was taken into consideration. A literature review on past laboratory and field studies involving pull-off tests was conducted. The results from these previous studies were analyzed. Special observations regarding the interpretation of these results are discussed, as well as the limitations of this test method and its suitability for application in the laboratory and the field.

### 1.4 Organization of Thesis

This thesis is divided into five main chapters. Following the Introduction in Chapter 1, Chapter 2 contains a literature review of FRP durability studies that have been completed in the past and the different test methods that previous researchers have used to evaluate the FRP-concrete bond. Chapter 3 is focused on the laboratory tests and their results. It contains the details of the specimens, including preparation and FRP reinforcement. In addition, this chapter describes the various environmental scenarios chosen for the long term study, as well as the testing procedures and discussion of results from both direct tension pull-off tests and bending tests.

Chapter 4 evaluates the pull-off test method as a means to collect bond data. More specifically, it summarizes past pull-off testing procedures and discusses the advantages and limitations of conducting these tests in the laboratory and how useful the information would be for situations in the field. Chapter 5 contains the conclusions based on the results obtained from the laboratory tests and evaluation from chapters 3 and 4. Finally, further recommendations are given based on the findings from the laboratory testing and the literature review.

# Chapter 2. Literature Review

## 2.1 Importance of FRP – Concrete Bond

The use of Carbon Fiber Reinforced Polymers (CFRP) has progressively gained popularity in the reinforcement of aging and deteriorating concrete structures. Other than cost, the reason why this repair method has not yet been more widely used in the field is due to the lack of knowledge about the long term behavior of the CFRP material itself and of the bond between the CFRP and concrete. A strong bond is vital for proper transfer of stresses between the concrete and the reinforcement. If a structural element is poorly reinforced with CFRP, premature de-bonding is likely to occur, leading to failure of the structure at load capacities much lower than what the reinforcement was designed to provide (Karbhari & Ghosh, 2009). In addition, environmental exposure may significantly affect the bond performance over time. Natural conditions such as rapid temperature changes, fires, snow and rain, as well as manmade conditions including application of deicing salts on roads and bridges, are some of the factors involved in the deterioration of a bond.

Over the years, research has been conducted to study the behavior of the bond between FRP and concrete using different testing methods and testing exposures. The following sections of this chapter provide descriptions of the various methods used by previous researchers to test bond, and reviews durability studies that have been conducted in the past.

## 2.2 Bond Tests

The strength and behavior of the bond between CFRP and concrete can be determined through various testing methods, depending on the nature of the study. Factors such as size, geometry, and quantity of specimens are taken into consideration when choosing an appropriate bond test. Sections 2.2.1 through 2.2.5 describe the different bond testing methods that were evaluated, and the various reasons why two methods were specifically chosen for the purposes of the durability study, described in Chapter 3.

### 2.2.1 Direct Shear Tests

Previous studies have employed direct shear tests in order to test bond strength under pure shear. Pan and Leung (2007) used direct shear tests to "study the crack-induced de-bonding failure in reinforced concrete members flexurally strengthened with FRP composites". Figure 2.1 illustrates a direct shear test using a simple diagram. For testing procedures, the concrete specimen, already bonded with the FRP composite, was placed vertically on the material testing system. In order to perform direct shear on the bond, a steel frame was designed to hold the specimen in its vertical position. When aligned properly, the concrete specimen was held in place, while the FRP plate was subjected to an upward tensile force, causing direct shear between the concrete beam and the composite (Pan & Leung, 2007). The primary disadvantage of this method is the complexity of having to build a custom frame to hold the specimen in place. The slightest error in alignment could have caused eccentricity on the specimen which would have decreased the accuracy of the results. For this reason, direct shear tests were not used in this study.

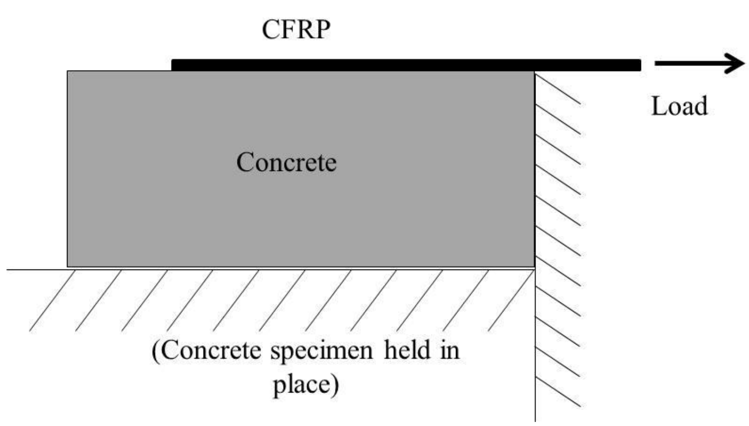


Figure 2.1 Direct Shear Test Representation.

### 2.2.2 Double- Face Shear Tests

Double shear tests have some similarity with single shear tests in the sense that they both determine the strength of the CFRP-concrete bond under pure shear. However, double shear tests have two bonded regions being tested at the same time. More specifically, two blocks of concrete of the same type and dimensions are attached together by two strips of CFRP on opposite sides. Testing of these specimens has been conducted in different ways. Ko and Sato (2007) performed a study in which a steel bar was internally fixed in the concrete block, and cut in the middle to allow the stress to be distributed into the concrete and the composite. Uniaxial tension was applied by gripping the steel bar, and measurements were recorded with the use of strain gauges. (Ko & Sato, 2007). In addition, a variation of a double-face shear test can consist of pushing two specimens away from each other. Both types put the CFRP-concrete bond in pure shear. Figure 2.2 shows the two different kinds of double- face shear tests. The main issue that was noticed with this test type was making sure that both concrete blocks were properly aligned when the two FRP sheets were bonded. Also, handling of the specimens seemed difficult, particularly when moving them was necessary. As a result, double shear tests were not conducted in this study.

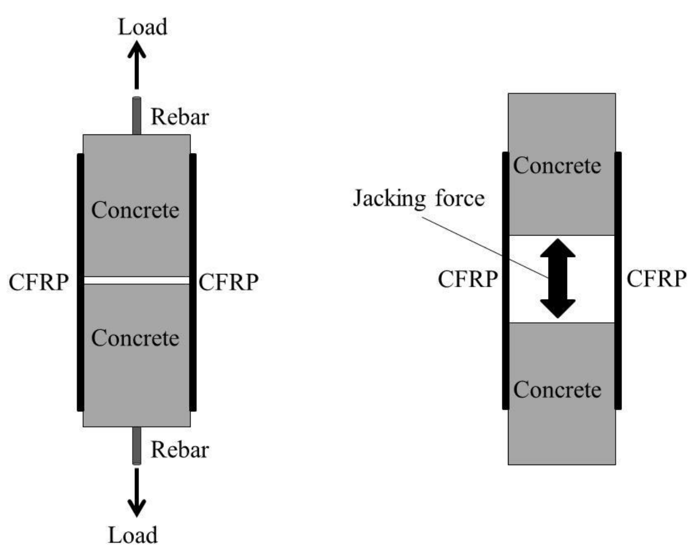


Figure 2.2. Types of Double-Face Shear Tests

### 2.2.3 Direct Tension Pull-Off Tests

The pull-off test is a test method that determines the greatest tension force (applied perpendicular to the bond) that the FRP-concrete bond can resist. The method consists of adhesively bonding a metallic circular loading fixture, also referred to as a dolly or puck, to the surface being tested. The dolly contains a threaded hole in the center that allows for attachment of the fixed alignment adhesion testing device, also known as a pull-off tester. Once attached, the tester slowly applies tension to the bond until a partial or full detachment of the dolly is witnessed, at which point the load is regarded as maximum bond force. Figure 2.3 illustrates a pull-off test scenario.

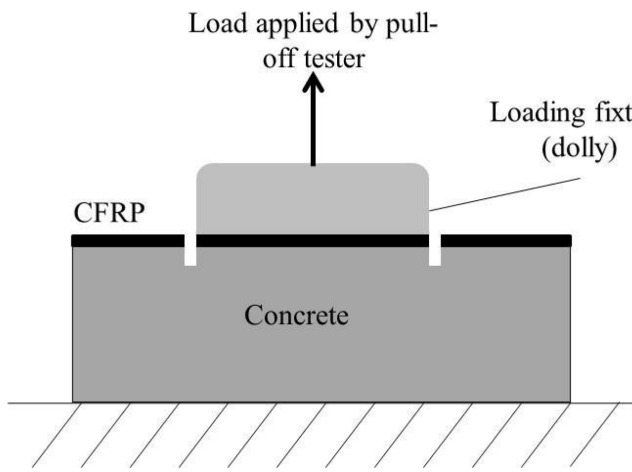


Figure 2.3. Pull-off test representation

The main instruments needed to perform pull-off tests consist of the pull-off tester, loading fixtures (dollies), epoxy adhesive to attach the dollies to the surface, and a core drill or circular hole cutter. The circular hole cutter is used to isolate the area being tested from the rest of the surface. This hole must be the same diameter as the loading fixture, commonly taken as 50 mm (2.0 in). These instruments are shown in Figure 2.4.

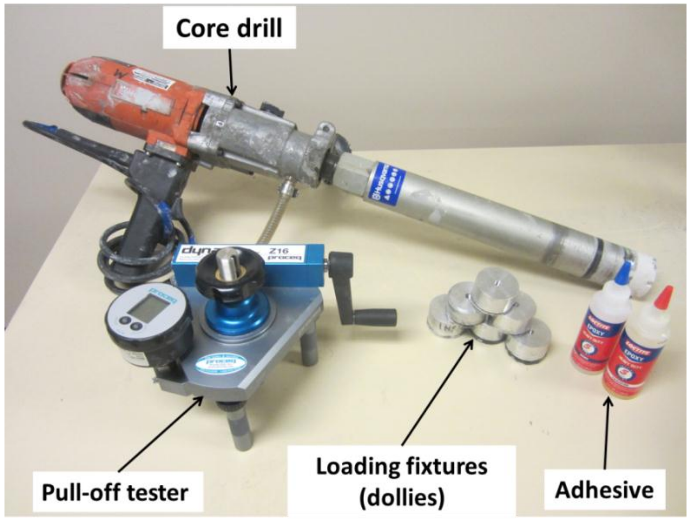

Figure 2.4. Instruments needed to conduct pull-off tests

Prior to 2009, the standard used as guidance for pull-off tests was ASTM D4541.

This standard was primarily created as a test method for the pull-off strength of coatings. However, due to similarities in specimen preparation and testing procedures, the standard was used by previous studies as a method for testing pull-off strength of FRP materials bonded to concrete. With the increase in popularity of this specific test application, ASTM D7522/D7522M was created in 2009, specifically to determine the pull-off strength of FRP bonded to concrete. The standard is applicable to both wet lay-up and shop-fabricated or pultruted laminates bonded to concrete. The test cannot be classified as non-destructive, but due to its relatively small scale, surface repairs are minimal. The test procedures, including preparation of the specimens, are described in Section 3.4.3.

The maximum force recorded during each pull-off test is used to calculate the pull-off bond strength, as shown in Equation 2.1, where $\sigma_p$ is the pull-off strength, $F_p$ is the maximum pull-off force, and D is the diameter of the dolly.

Equation 2.1

$$\sigma_p = \frac{4F_p}{\pi D^2}$$

Following completion of the test, different failure characteristics may be witnessed at the bond surfaces. ASTM D7522/D7522 (2009) classifies these failure modes into seven types, labeled from Mode A through Mode G. These failure modes are summarized in Table 2.1.

Table 2.1. Pull-off Test Failure Modes (ASTM D7522/D7522M, 2009)

| Failure Mode | Failure Type | Causes of Failure |
|---|---|---|
| A | Bonding adhesive failure at dolly | Improper adhesive bonding of dolly. Not an acceptable failure mode. |
| B | Cohesive failure in FRP | Improper saturation of the FRP, environmental degradation. |
| C | Adhesive failure at FRP/adhesive interface | Contamination of adhesive during application, incomplete adhesive cure. |
| D | Cohesive failure adhesive | Contamination of adhesive, incomplete cure, environmental damage of material. |
| E | Adhesive failure at FRP/concrete interface | Contamination of adhesive during application, incomplete adhesive cure. |
| F | Failure mode E and G combined | Inconsistent FRP-concrete adhesion. Failure is partly adhesive and partly on substrate |
| G | Cohesive failure in concrete substrate | Proper adhesion of FRP-concrete. Desirable failure mode |

Figure 2.5 shows the interfaces which these seven failure modes represent. The image is not to scale. The adhesive and FRP layers have been magnified for clarity.

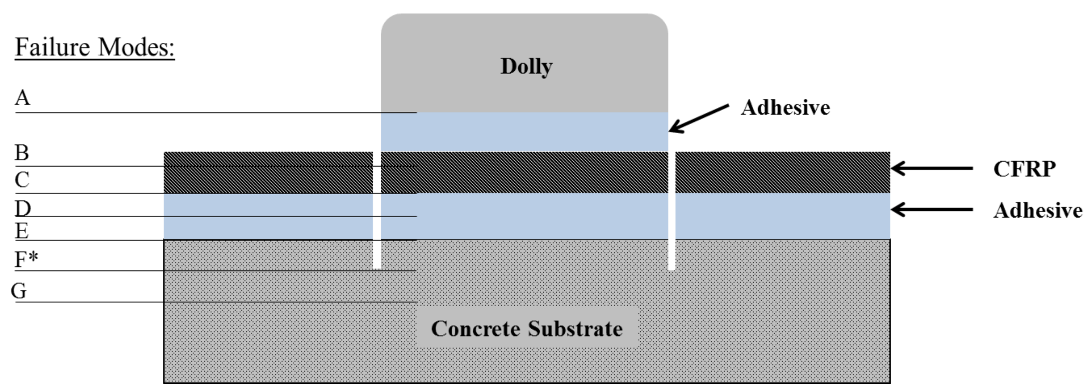


Figure 2.5. Pull-off test failure modes. Mode F failure represents a combination of Mode E and Mode G failures

For the purposes of the durability study conducted in this research, a variety of environmental scenarios were considered, which made it necessary to fabricate a large number of specimens. In addition, convenient handling of the specimens was needed, especially for the groups that underwent wet-dry cycles and freeze-thaw cycles. As a result, due the low cost, small scale, and convenient procedures, pull-off tests were chosen as the primary test method.

### 2.2.4 Three-Point Beam Bending Tests

Beam bending tests represent the second FRP-concrete bond test method chosen for this study. In relation to pull-off strength tests, bending tests provide a more realistic behavior of the FRP concrete bond when subject to flexural loads. Gartner, Douglas, Dolan, and Hamilton (2011) recently presented a study in which this new bond test method is introduced. The authors wanted a new testing procedure that was simple to perform, easy to understand, and that could allow for fabrication of a large number of specimens for statistical validation. Their test method was primarily based on ASTM C78/C78M (2010), a standard test method for the flexural strength of concrete.

Three modifications were made by Gartner et. al. (2011) to ASTM C78 in order to adapt the test method for determining bond strength between CFRP and concrete: a saw cut was added at the mid-span of the beams, CFRP sheets were added to the tension face, and loading was modified from four-point bending to three point bending. The test is based on flexure because as tension develops at the bottom of the beam where the CFRP is located, shear stresses develop to transfer forces between the concrete and FRP. Three-point bending puts the bond in shear, and allows for calculation of the bond shear strength. Three-point bending was also used because with this loading configuration there is less chance of a concrete shear failure outside of the CFRP reinforcement area, helping to ensure that the test actually measures bond strength. The saw cut is placed at mid-span to ensure that failure starts to develop at the top of the cut, which forces the CFRP reinforcement to fully mobilize its development length. The development length represents the length of the composite that undergoes the bond shear strength during loading; more specifically, it constitutes the length starting from the edge of the saw cut to the end of the carbon fabric, as shown in Figure 2.6.

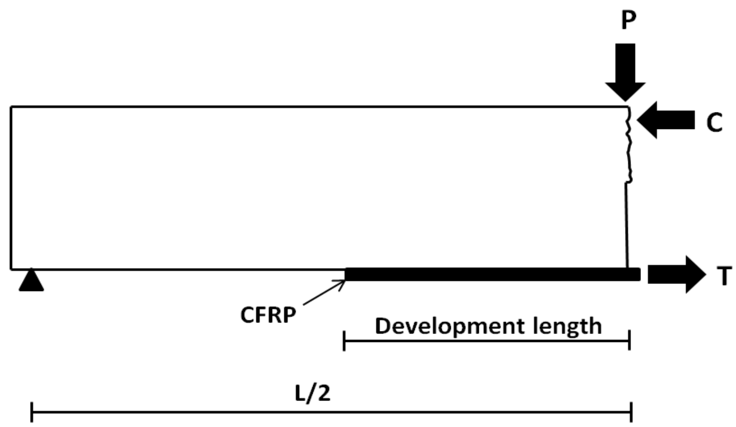


Figure 2.6. Development length and forces under three-point bending. This figure shows half of the beam. P is the load applied at mid-span, T is the tension force generated at the bottom of the beam, and C is the compression force at the top of the beam

As mentioned in Section 2.2.3, pull-off tests were chosen as the primary test method. Pull-off tests, however, apply the load perpendicular to the surface. When the FRP repair is in service, perfectly perpendicular loads are never experienced by an FRP-concrete bond, meaning it can be difficult to understand what the test results actually mean in terms of structural performance, As a result, three-point beam bending tests were chosen as a supplementary test method for this research study, since it provides a more realistic scenario of the behavior of FRP reinforcement on actual concrete structural members.

## 2.3 Previous Research

This section describes previous studies that have investigated FRP-concrete bond behavior under various types of environmental conditioning. The two main types of conditioning include freeze-thaw exposure and moisture exposure. Characteristics of previous experimental designs such as number of freeze-thaw cycles, temperature ranges, number of wet-dry cycles, duration of exposure and the type of solution used in the study, (e.g. plain water, salt water, deicing agents) are described below.

### 2.3.1 Exposure to Freeze-Thaw Cycles

Freeze-thaw exposure is a matter of great importance when considering FRP reinforcement. There are a significant number of reinforced concrete structures that are located in regions where freeze-thaw cycling is common. The large temperature changes cause thermal expansion and contraction in both the concrete and FRP. Since these two materials possess different coefficients of thermal expansion, stresses in the FRP-concrete interface may be introduced during this process and therefore cause premature de-bonding (Yun and Wu, 2010). Studies regarding freeze-thaw conditions of FRP- reinforced concrete have been conducted in the past. However, there is a limited amount of research that focused mostly on the bond behavior, as most of the completed research targets FRP tensile strength in general.

Green et. al (2000) developed a study focused on the evaluation of the bond durability of fiber reinforced polymer bonded to concrete. With 16 hours of freezing and 8 hours of thawing in water, a total of 50, 150, and 300 freeze-thaw cycles were applied, resulting in a 300-day long study. ASTM C310 (1961) was followed in place of the common standard ASTM C666, since the latter was not yet created at the time of this research. For ASTM C310, the temperature ranges were much larger than those in ASTM C666. Temperatures varied from 0$^{o}$F (-18$^{o}$C) in the freezing stage to 59$^{o}$F (+15$^{o}$C) in the thawing stage. Two test methods were considered in this specific study: single-shear tests and four-point flexural tests. A total of twelve 6 in x 6 in x 16 in (150 mm x 150 mm x 400 mm) blocks were tested under single shear, and nine small beams 4 in x 6 in x 48 in (100 x 150 x 1220 mm) were tested under bending, all with a 28-day concrete strength of 4.5 ksi (31 MPa). CFRP reinforcement on the blocks had dimensions of 2 in x 12 in (50 mm x 300 mm), one strip placed in the center of the blocks for shear testing, and two strips placed on the ends of the beams, leaving the center region un-bonded to encourage a de-bonding failure rather than an FRP tensile failure (Green et. al., 2000). In general, an increasing number of freeze-thaw cycles would be expected to cause degradation of the specimen to some degree. However, direct shear test results in this study indicated an increase in the ultimate load and maximum strain with increasing number of cycles. Green et. al. (2000) explains that this may have been due to the enhanced curing process on the specimens that underwent freeze-thaw exposure, since these were left in a water bath for the entire cycling period, as opposed to the control specimens which were left dry and in room temperature conditions.

As far as failure mode goes, as the number of cycles increased, the failure plane was transferred into the FRP-epoxy layer. In other words, control specimens showed a full failure in the substrate, whereas blocks undergoing conditioning experienced a more adhesive failure. As a result, Green et. al. (2000) formulated the hypothesis that a decrease in the shear modulus of the adhesive could have lowered the stress concentrations in the concrete substrate, which would result in an increase in the bond strength. Regarding the beam test results, a similar behavior was observed. Values for ultimate load and deflection at mid-span increased with larger number of freeze-thaw cycles, as well as an identified increase in bond strength and FRP strain. Failure characteristics in all of the beams were fairly consistent as shear failure on the FRP-concrete interface was generated from flexural cracks at the end of the strips. Also, these beams were significantly affected by freeze-thaw cycling. Overall, the failure modes were similar for both testing methods in the sense that as the number of cycles increased, the failure was more predominant in the FRP-adhesive interface.

In addition to the study conducted by Green et. al. (2000), Colombi et. al. (2010) developed a study in which freeze-thaw cycles were applied to concrete elements reinforced with CFRP to evaluate the effects of this environmental conditioning on the CFRP-concrete interface. Two different kinds of CFRP reinforcement were used: CFRP plates, also known as pultruded strips, and CFRP wraps applied by means of the wet- layup process. In addition, two different reinforcing lengths of 4 in and 16 in (100 mm and 400 mm) were used, half for each of the reinforcement types. Twenty concrete blocks with dimensions of 6 in x 6 in x 24 in (150 mm x 150 mm x 600 mm) were fabricated in wooden molds and their respective CFRP reinforcement were adhered following proper curing and preparation of the concrete surface. Using ASTM C666, the freeze-thaw exposure was characterized by varying the temperature from $0^{o}$ F to $40^{o}$ F ($-18^{o}$C to $+4^{o}$ C), with a cycle duration of 5 hours. Stages were divided into 100 and 200 cycles, and single-face shear tests were conducted following completion of freeze-thaw exposure. In this study, the bond was not significantly influenced by the different freeze-thaw cycles applied. In other words, the de-bonding force resisted by the conditioned specimens was relatively similar those that were not exposed. Therefore, it was concluded that for the study freeze-thaw cycles had a marginal effect on the bond strength (Colombi et. al, 2010).

Whilst Colombi et. al (2010) investigated the FRP-concrete bond reaction to freeze-thaw exposure in a dry environment, other studies actually introduced this conditioning combined with the presence of moisture. Yun and Wu (2010) explored the durability of the CFRP-concrete bond under freeze-thaw exposure by means of single-face shear tests. The main parameters under consideration were concrete strength, freeze-thaw solution, and number of freeze-thaw cycles. Thirty concrete blocks of sizes 6 x 6 x 13 $in^3$ (150 x 150 x 340 $mm^3$) were created using two different concrete strengths: 4.4 ksi (30 MPa) and 6.5 ksi (45 MPa). ASTMC672/C672M and ASTMC666 were used as standard guidelines. The specimens were placed in a 0.25 in (6 mm) depth solution of salt water or tap water. Temperatures ranged from $0^{o}$ F to $40^{o}$ F ($-18^{o}$C to $+4^{o}$ C), and the total length of one cycle was 240 min. Exposure stages were divided into 17, 33, 50, and 67 cycles. Based on single shear testing, the results indicated a decrease in bond strength and bond stiffness as the number of cycles increased. All failure modes were located at the concrete level, no adhesive failure was shown. Even though all of the failures were in the concrete substrate, it was clearly identified that the depth of the concrete that detached with the FRP varied significantly in relation to the number of cycles, type of solution, and strength of concrete. The specimens with higher concrete strength showed heavier deterioration in the bond for both salt water and tap water. For these higher strength concrete specimens, damage was more severe on those exposed to salt water under a larger number of cycles. On the other hand, the lower grade concrete specimens showed no significant difference in the bond strength when comparing salt water and tap water exposure. It is important to emphasize the fact that on the higher strength concrete samples, the depth of the concrete that failed was equivalent to the depth of the salt water solution, which likely indicates that failure was mostly due to concrete degradation and not necessarily bond degradation. However, such high degradation on the concrete just below the FRP-concrete interface can over time affect bond stiffness and strength (Yun and Wu, 2010).

Similar studies involving freeze-thaw cycles of FRP reinforced concrete include the work done by Green et. al. (1997), and Chajes et. al (1995), and Hu. Et. al. (2007). Green et. al. (1997) found that exposure to 50 freeze thaw cycles did not appear to damage the CFRP-strengthened concrete beams. On the other hand, Chajes et. al. (1995) identified an average decrease in beam strength of 21% after being exposed to 100 freeze-thaw cycles in a calcium chloride solution. Other studies not directly involving freeze-thaw but rather focusing on extreme temperature exposure include the work done by Gamage, Mahaidi and Wong (2009). The research targeted higher cyclic temperatures of $68^{o}$F to $122^{o}$F ($20^{o}$C - $50^{o}$C) and 90% relative humidity applied on the CFRP reinforced concrete specimens. This scenario was not considered in this specific study but provides an insight on the different kinds of durability tests that have been developed in the past. Finally, the work done by Hu et. al. (2007) was focused on conducting direct shear tests to specimens that underwent 25 and 50 freeze-thaw cycles. The cycle consisted of 2 hours of freezing and 1 hour of thawing. Results showed a decrease in bond strength of 20% in the specimens exposed for a longer period. Hu et. al. (2007) concluded that freeze-thaw cycling had an adverse effect on those specimens exposed for 50 cycles. However, little deterioration was seen on the specimens exposed to 25 cycles.

### 2.3.2 Exposure to Moisture and Wet-Dry Cycles

An additional durability type of study that should be paid careful attention involves the exposure of CFRP and concrete to the effects of moisture, which includes constant immersion in water or other kinds of solutions such as salt water or deicing agents. Studies where wet-dry cycles were considered are also described in this section.

Toutanji and Gomez (1997) studied the effects of moisture on the interfacial layer of the FRP and concrete. With 4 hours of wetting and 2 hours of drying, 300 wet-dry cycles were applied on 28 prepared specimens, for a total of 75 days for completion of the exposure. The cycles were characterized by immersion in salt water during the wetting period, followed by air at 95$^{o}$F (35$^{o}$C) and 90% relative humidity during the drying period. In comparison to the specimens kept as control, the ones exposed to wet-dry cycling exhibited lower performance during testing. Failure modes were mostly characterized by de-bonding at the FRP-concrete interface. In addition, three different epoxies were used on the specimens and then strength degradation rates were compared among the three. Finally, they concluded that the degradation in the bond strength may have been due to the deterioration of the epoxy during exposure.

Malvar, Joshi, Beran, and Novinson (2003) studied the effects of moisture and chloride content on the CFRP bond to concrete by means of pull-off tests. Their study involved the testing of a square concrete pile that had been exposed to saltwater and marine conditions for 48 months. The pile was reinforced with CFRP and aluminum dollies were then attached to the surface. Tests were split into sections where a primer and hydro blasting was used on the surface, and compared to a testing surface where neither primer nor hydro blasting was used. It was found that hydro blasting helped remove some of the chlorides already present on the surface, and application of primer enhanced the adhesion of the reinforcement, which contributed to higher bond strength. Even though the lack of control specimens or baseline values make it difficult to provide comparisons in results from the durability standpoint, this study reflects a realistic scenario of the moisture exposure that structural members reinforced with CFRP can face.

Wet-dry cycling has been considered by several studies. Research in the past has evaluated the effects of wet dry cycles on the interfacial bond performance. Soudki et. al. (2007) conducted and experimental program to investigate the durability of concrete beams reinforced with CFRP material, after being exposed to a corrosive environment. Eleven large-scale concrete beams with dimensions of 4.7 in x 6.9 in x 7.9 ft. (120 mm x 175 mm x 2.4 m) and 28-day compressive strength of 5 ksi (35 MPa) were cast, eight of which were pre-cracked and then reinforced with two types of FRP (CFRP sheets and CFRP strips). The remaining three beams were kept as control. The eight reinforced beams were subjected to a series of wet-dry cycles in a sodium chloride solution with 3% concentration.

The number of cycles was divided to 0, 100, 200, and 300 cycles. Following the exposure, four-point bending tests were conducted on the beams. Inspection of the beams after testing showed a premature de-bonding failure which started at the end of the CFRP strips and then propagated towards the entire length of the reinforcement. It was also observed that for these beams, except the one not exposed to wet-dry cycles, the load capacities decreased rapidly as soon as the de-bonding was witnessed. At this point, the behavior of the beam was similar to those seen in an un-strengthened beam. In addition, the number of wet-dry cycles had a direct effect in the degradation of bond strength. More specifically, the specimens that were exposed to 100, 200, and 300 cycles showed a decrease in strength of 19, 25, and 28%, respectively. In regard to the beams strengthened with CFRP sheets, the failure mode was similar to those strengthened with the strips. However, the strength reduction was less noticeable, with a decrease of 2, 6, and 11% following 100, 200, and 300 cycles.

An additional study involving exposure to sodium chloride includes the work done by Pan et. al. (2010). The effect of chloride content on the behavior of bond between concrete and FRP was studied. Fourteen concrete blocks with dimensions of 6 in x 6 in 6 in (150 mm x 150 mm x 550 mm) were cast using two different concrete strengths: ten blocks with and average compressive strength 2.50 ksi (17.25 MPa), and four blocks with a strength of 2.33 ksi (16.06 MPa). These specimens were reinforced with two layers of FRP and were fully immersed in chloride solution to ensure that the concrete-FRP interface was eroded (Pan et. al. 2010). Four solutions with varying levels of sodium chloride concentration were used (3%, 6%, 10%, and 15%). In addition, exposure times were 0, 15, 30, 60, 90, and 120 days. Direct shear tests were conducted on these specimens to determine any possible bond deterioration over time.

Following testing, all specimens showed a de-bonding failure. Correlations between the ultimate debonding load and the immersion time and chloride concentration were made. It was determined that the ultimate load decreased in the specimens that were immersed for 0 to 30 days, mainly due to the deterioration of the adhesive when exposed to the chloride solution. In addition, this load also decreased in those specimens immersed for longer than 90 days due to degradation of both the concrete and the adhesive. On the contrary, the specimens immersed between 30 and 60 days showed an increase in the ultimate load, which was caused by the increase in concrete strength resulting from further hydration (Pan. et. al. 2010). Regarding the levels of chloride concentration, these seemed to have virtually no effect on the ultimate loads. However, further studies involving were recommended to be conducted for longer exposure times to corroborate the effects of chloride concentration in the bond strength.

Additional studies in this area include the work done by Qiao et. al. (2004), whom conducted an experimental program involving 50 and 100 wet-dry cycles applied to 12 concrete specimens strengthened with FRP. The cycle consisted of immersing the specimens in a calcium chloride solution for 16 hours, and then removing them from the solution and letting them dry in air for 8 hours. ASTM C672 was used as a guideline in the development of the conditioning. Following exposure, three-point beam bending tests were conducted. Qiao. et. al. (2004) determined that as the number of cycles increased, the adhesive failure was noticeable over time. However, no further discussion of results was seen in this research.

Finally, Dai, Yokota, Iwanami and Kato (2010) conducted a series of pull-off and flexural tests on FRP strengthened concrete blocks and beams after having them exposed to 8, 14, and 24 months of wet-dry cycles. The wet dry conditioning consisted of four days of immersion in sea water followed by a three-day drying period at room temperature. The sea water was heated to 140$^o$F (60$^o$C) in order to accelerate the diffusion of moisture into the concrete substrate (Dai et. al. 2010). They also concluded that the bond strength was degraded as the number of cycles increased, and identified that bond strength could be significantly enhanced if the right type of primer was used.

As seen in this chapter, previous research has focused mostly on the influence of freeze-thaw and wet-dry conditions in dry air, water, salt water (sodium chloride), or calcium chloride. Magnesium chloride or other popular deicing agents have not been thoroughly tested in this field of study. As a result, special attention was given to the effect of deicing agents commonly used on Colorado roads because they have received less attention in previous studies. Chapter 3 presents the entire laboratory testing plan that was conducted for this research, as well as discussion of the results.

# Chapter 3. Durability Tests

## 3.1 Testing Plan Overview

The purpose of this study was to evaluate the behavior of the bond between the concrete and the CFRP when subjected to various environmental scenarios. These environmental conditions include freeze-thaw and wet-dry cycles, as well as immersion in deicing agents over two testing stages. The testing stages consisted of keeping the specimens exposed to these scenarios over a period of 6 months, and 12 months. Section 3.3 describes the stages in more detail.

In order to test the bond between the CFRP and concrete, various testing methods were evaluated with the purpose of finding a test that was most suitable for this study. Tests such as single shear and double face shear were considered. However, these types of tests showed to have some inconveniences in relation to the goal of this study. Since various environmental scenarios were considered, a large amount of specimens was needed. Therefore, practical specimen sizes were necessary for easy handling of the blocks, as well as lower cost for materials and testing devices. As a result, two different testing methods were chosen: pull-off tests, and small three-point bending tests.

## 3.2 Environmental Exposure Scenarios

During the winter months in Colorado and other northern regions, roads and bridges are faced with various adverse weather conditions that may affect their performance over time. These conditions include exposure to rain and snow. In addition, the use of deicing products to improve driver's safety on bridges is also a factor. As a result, various environmental exposures were considered in the study to evaluate the FRP-concrete bond durability. These exposures include:

immersion in deicing agents, wet-dry cycles, freeze-thaw cycles, and immersion in water. Each exposure is described in more detail in the subsequent sections.

### 3.2.1 Exposure to Deicing Agents

To evaluate long term bond durability under deicing exposure, concrete blocks and beams reinforced with CFRP were placed face down in a 0.25 in - 0.50 in (6 mm - 13 mm) depth of deicing solution. ASTM C672/C672M (2003) was used for guidance on the depth of solution needed for the conditioning of the beams and blocks. The standard specifies the exposure procedure using a solution of calcium chloride, but for the purposes of this study, two different deicers were used: Meltdown Apex and Apogee, provided by Envirotech Services. The first one is characterized as a performance enhanced Magnesium Chloride solution, while the second one is described as a non-chloride deicer.

For preparation of the deicers, both products were diluted with water at a 1:1 weight ratio to achieve a concentration more representative of field conditions. Exposure was carried out for all of the testing stages described in Section 3.3. Since a constant depth was desirable, specimens were monitored to make sure a minimum depth of 0.25 in (6 mm) was maintained. When the depth was lower than the recommended 0.25 in, the plastic bins containing the specimens were refilled to the desired depth. However, all of the containers were fully covered which prevented rapid evaporation of the solution. As specified by Envirotech, concentration of the solutions is shown to decrease over time. As a result, samples of the solutions were collected and taken to the facilities at Envirotech to determine the rate at which the concentration decreased. With this rate, it was determined how often a new batch of Meltdown Apex and Apogee were needed to be mixed, in order to keep a constant concentration and avoid discontinuities in the long term exposure.

### 3.2.2 Wet-Dry Cycles

A series of wet-dry cycles were applied on some specimens for all of the testing durations. One complete cycle was as follows: specimens remained soaking in a 0.25-in depth of Magnesium Chloride solution for 4 days, then were removed from the containers and allowed to dry for 3 days. The week-long cycles were repeated for 6 and 12 months.

### 3.2.3 Freeze-Thaw Cycles

Freeze-thaw exposure was applied to some of the specimens. Since there is no specific standard for testing FRP-concrete bond under freeze thaw conditions, the exposure used by Yun and Wu (2011) was followed in this study. Two ASTM standards for concrete were used as guidance to develop this exposure: ASTMC666 (2003) and ASTMC672/C672M (2003). A total of four blocks underwent freeze-thaw exposure: two for Stage 1 and two for Stage 2, resulting in a total of 6 pull-off tests per stage. No small beams for bending tests were considered for freeze-thaw exposure due to limited freezer space. Specimens were placed in the Magnesium Chloride solution with the FRP side down at a depth no smaller than 0.25 in (6 mm) (ASTM C672/C672M, 2003), as illustrated in Figure 3.1.

Figure 3.1. FRP Reinforced Concrete specimens undergoing free-thaw exposure

To ensure that the depth was kept constant, specimens were monitored often and refilled with solution to prevent considerable loss due to evaporation. One freeze-thaw cycle was characterized as follows: the temperature was held constant at 40 $^{0}$F (4.4 $^{0}$C) for 8 hours; the temperature was then decreased to 0 $^0$F (-17.8 $^0$C) in 30 min and was held constant for 15 hours. Finally, the temperature was increased back to 40 $^0$F (4.4 $^0$C) in 30 min. The total time for one cycle was 24 hours.

The temperature ranges were obtained from the ASTMC666/C666M Standard recommendations. However, this standard targets rapid cycles, in which it is advised that one cycle be no longer than 5 hours, with 300 being the maximum number of cycles during a test. For the purposes of this study, freeze-thaw exposure was chosen to last the same time as the other environmental scenarios, for proper comparison of the different results. For this reason, freeze-thaw exposure was continued for 183 cycles for testing stage 1, and 365 cycles for testing stage 2, 6 months and 12 months respectively.

## 3.3 Long Term Testing Stages

Three testing stages were executed in the study. Stage 0 consisted of testing 3 beams under three-point bending and 2 blocks after the application of the CFRP. With 3 pull-off tests per block, a total of 6 pull-off tests were performed during this stage. Stage 0 specimens were characterized as control, kept in dry conditions and at room temperature. These specimens provided a basis for comparison for the later testing stages. Stage 1 was the testing stage following 6 months of environmental exposure. A total of 15 blocks and 13 beams were tested during this stage. Finally, Stage 2 represented 12 months of exposure, in which 15 blocks and 13 beams were tested. Tables 3.1 and 3.2 summarize the entire testing plan, showing the different types of specimens and environmental scenarios at each stage.

Table 3.1. Stage 1 (6-month) Tests

| Pull-off | | | Beam Bending | | |
|---|---|---|---|---|---|
| # of Blocks | Exposure | CFRP Layers | # of Beams | Exposure | CFRP Layers |
| 1 | Dry | 2 | 2 | Dry | 2 |
| 2 | Water | 2 | 2 | W/D in Chloride Deicer | 2 |
| 2 | W/D in Chloride Deicer | 2 | 3 | Water | 2 |
| 2 | Non-Chloride Deicer | 2 | 3 | Non-Chloride Deicer | 2 |
| 2 | Non-Chloride Deicer | 3 | 3 | Chloride Deicer | 2 |
| 2 | Chloride Deicer | 2 | | | |
| 2 | Chloride Deicer | 3 | | | |
| 2 | F/T in Chloride Deicer | 2 | | | |

Note: W/D = Wet- Dry cycles, F/T = Freeze-Thaw cycles

Table 3.2. Stage 2 (12-month) Tests

| Pull-off | | | Beam Bending | | |
|---|---|---|---|---|---|
| # of Blocks | Exposure | CFRP Layers | # of Beams | Exposure | CFRP Layers |
| 1 | Dry | 2 | 2 | Dry | 2 |
| 2 | Water | 2 | 2 | W/D in Chloride deicer | 2 |
| 2 | W/D in Chloride deicer | 2 | 3 | Water | 2 |
| 2 | Non-chloride deicer | 2 | 3 | Non-chloride deicer | 2 |
| 2 | Non-chloride deicer | 3 | 3 | Chloride deicer | 2 |
| 2 | Chloride deicer | 2 | | | |
| 2 | Chloride deicer | 3 | | | |
| 2 | F/T in chloride deicer | 2 | | | |

Note: W/D = Wet- Dry cycles, F/T = Freeze-Thaw cycles

## 3.4 Fabrication and Testing of Specimens

With pull-off and three-point flexural tests chosen for testing the bond, two types of concrete specimens were manufactured according to the quantities shown in Tables 3.1 and 3.2. The concrete specimens were then reinforced with the carbon fiber fabrics and finally exposed to their respective environmental exposures. Sections 3.4.1 through 3.4.4 explain in detail the procedure for the concrete casting, CFRP application, and specifics for the pull-off and bending tests.

### 3.4.1 Concrete Specimens

The concrete mix was obtained from Lafarge North America, located north of Fort Collins. Specifications for the concrete were taken from the Colorado Department of Transportation 2011 Specifications Book, Section 601: Structural Concrete. For the purposes of this study, Class D concrete was chosen. Class D concrete was chosen to represent a common concrete type that is used in bridges. The mix specifications included a slump of 4 inches, air entrainment of 5-8%, a water to cement ratio of 0.45, and a 28-day compressive strength of 4500psi (31.0 MPa). As stated in the specifications book, Class D concrete is a dense medium strength structural concrete, required to be made with AASHTO M 43 sizes No. 57, or No. 67 coarse aggregate.

Wooden molds were fabricated prior to casting the concrete. Once the forms were finished, the mixing truck arrived at the Colorado State University Engineering Research Center to proceed with the pouring of concrete. A total of 45 blocks for pull-off tests, 42 beams for bending tests, and 17 cylinders for compressive strength tests were cast, as shown in Figure 3.2. The details on these specimens (including dimensions) are specified in the following sections. The concrete specimens were allowed to cure for 5 days before removal from the forms. During this 5-day period, curing was aided by sprinkling water on the surface and covering them with sheets of transparent polyethylene plastic to help prevent water from evaporating.

Figure 3.2. Casting of concrete specimens

Following the removal of the blocks and beams from the forms, it was observed that some of the small beams showed a considerable amount of voids/ air pockets on the sides. Since Class D concrete has a significant amount of large aggregate, the workability of the mix was low which made it difficult to achieve proper compaction in small molds. Such behavior was not anticipated as previous related studies did not specify the maximum aggregate size used in their concrete samples. As a result, a total of 9 beams were repaired and patched using Class S mortar mix, and allowed to harden for 30 minutes before they were placed in water containers with the other specimens for curing. Figure 3.3 shows the characteristics of the specimens before and after mortar patching. All specimens were fully submerged in water for the remaining 23 days of the curing stage to provide them with as much moisture as possible and prevent cracking during this period.

Figure 3.3. Concrete beams before (left) and shortly after (right) patching

### 3.4.2 Application of CFRP to Concrete Specimens

Following proper curing of all the blocks and beams, the CFRP was applied. The carbon fiber fabrics were obtained from HJ3 Composite Technologies, characterized as unidirectional fabrics with design strength of nearly 119 ksi (821 MPa). Table 3.3 presents the physical properties of the composite provided by HJ3.

Table 3.3. Composite Properties from HJ3

| | Typical Values | Design Values |
|---|---|---|
| **Tensile Strength, ksi (MPa)** | 150 (1,034) | 118.6 (814) |
| **Modulus of Elasticity, ksi (MPa)** | 12,380 (85,357) | 10,433 (71,933) |
| **Ply Thickness, in (mm)** | 0.047 (1.194) | |
| **Strain at Rupture** | 0.0117 | |

All of the specimens were first sandblasted to the level of aggregate using 30 Grit silica based sand. The purpose of sandblasting was to eliminate any loose particles, debris, and uneven surfaces. Specimens were then air blasted to remove any dust and dirt accumulation from the surface. A surface that is sandblasted and air blasted ensures improved bonding of the carbon fabrics to the concrete. Figure 3.4 illustrates the concrete surface before and after sandblasting.

Figure 3.4. Concrete surface before (left) and after (right) sandblasting

Following proper cleanup of the surfaces, a primer coat provided by HJ3 Composite Technologies was applied on the surface of the specimens using a roller, spreading uniformly along the entire surface to avoid any excessive buildup. The goal of the primer as stated by the manufacturer was to promote higher bond strength between the CFRP and the concrete. The primer was allowed to cure for 48 hours and then a wet-layup process was used to manufacture the composite directly on the surface of the concrete, following the steps suggested by the manufacturer. The process consisted of first saturating the fabrics with the two-component epoxy and then pressing the carbon fiber strips onto the concrete surface. SRW-400 was the saturating epoxy used in the application of the CFRP. This two-part epoxy contained a resin and a hardener that were thoroughly mixed at a 2:1 ratio by volume. Rollers were used to ensure uniform pressure during CFRP application and to remove significant air pockets. Finally, any minor air pockets were removed by smoothly but firmly applying pressure on the specimens, starting from the center and moving towards the edges. The wet-layup procedure was repeated on the appropriate specimens to complete the double-layer and triple-layer CFRP reinforcement. Figure 3.5 shows the concrete blocks and beams reinforced with CFRP.

Figure 3.5. Concrete specimens reinforced with CFRP for pull-off (left) and flexural (right) tests

### 3.4.3 Pull-Off Test Specimens and Test

For the pull-off strength tests a total of 45 concrete blocks were cast. With three pull-off tests per block, this resulted in a total of 135 pull-off tests for the entire study. The blocks were 14 in x 6 in x 3.5 in (356 mm x 152 mm x 89 mm). These dimensions allowed a clear spacing of 2 in (50 mm), equivalent to the diameter of one dolly, between each pull-off. This allowed enough space between dollies to prevent any influence on one test from adjacent tests on the same block (Karbhari and Ghosh, 2009). The blocks were reinforced with carbon fiber fabric cut into sheets with dimensions of 13 in x 5 in (330 mm x 127 mm). Figure 3.6 illustrates the specimen dimensions and dolly spacing.

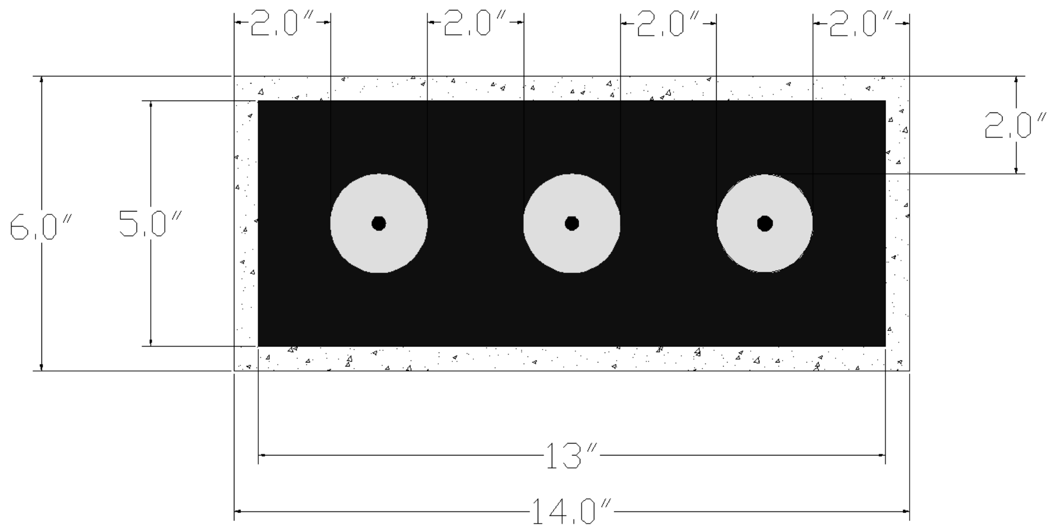


Figure 3.6. Specimen dimensions and dolly spacing

Two layers of CFRP were applied on 33 blocks. The remaining 12 blocks were reinforced with three layers of fabric. After environmental conditioning and shortly before testing, three 2 in (50 mm) diameter cores were made at the location where the dollies were going to be placed in each concrete block. The depth of the core cut was no less than 0.25 in (6 mm) but no more than 0.50 in (12 mm), as recommended by ASTM D7522. When using a core drill, it was likely that the drill bit would rapidly skip off the desired cut area and cause damage to the surface. A fixed wooden frame with three holes at the desired locations was built in order to keep the drill under control. Following the cutting of the cores and prior to adhesion of the dollies, both the CFRP surface and the dollies were roughened with medium-grit sandpaper, then cleaned with water and isopropyl alcohol, and allowed to dry for 24 hours. A roughened surface allows for improved bonding of the dollies on the FRP material. The dollies were then fixed onto the specimens using a Devcon high strength epoxy, with strength of 1500 psi (10.34 MPa). This was a two part epoxy containing a resin and a hardener with a curing time of 5 minutes. As a result, the dollies needed to be placed on the surface rapidly after the mixed epoxy was applied on the dollies. This epoxy was advertised as reaching maximum strength one hour following application. To ensure that the dollies were properly fixed to the surface, the epoxy was allowed to cure for a minimum of 24 hours before performing pull-off tests. Figure 3.7 shows the dollies adhered to the concrete specimens.

Figure 3.7. Concrete specimens following adhesion of dollies

Once the dollies were fully adhered, pull-off testing took place using a Proceq pull-off tester, model: Dyna Z16. The load was applied at the ASTM D7522 recommended rate of less than 150 psi/min (1 MPa/min) by slowly rotating the crank of the pull-off tester until the maximum force was reached. The machine contained a digital force indicator that displayed the pull-off strength as the crank was being rotated, until failure. Maximum strength was recorded, and failure modes were reported (Refer back to Section 2.2.3 for more information about failure modes). Figure 3.8 shows the setup for the pull-off testing.

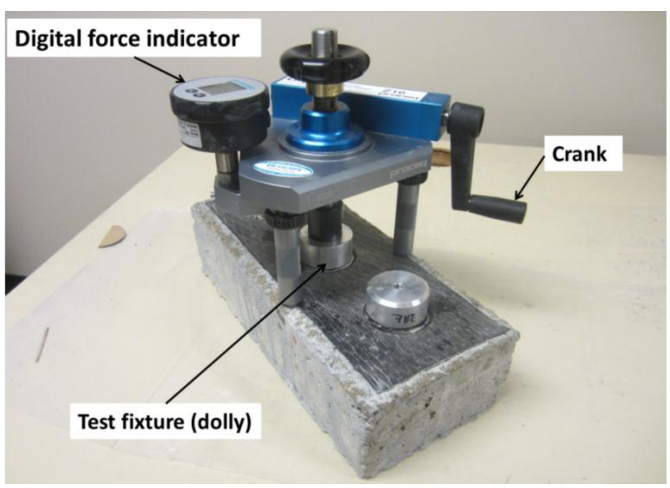


Figure 3.8. Pull-off testing setup

### 3.4.4 Beam Specimen and Tests

For the beam bending tests a total of 42 small beams were cast, with dimensions of 14 in x 4 in x 4 in (356 mm x 102 mm x 102 mm), as illustrated in Figure 3.9. Following the 28-day cure, the beams were saw cut at mid-span on the tension face to a depth of 2 in (50 mm), equivalent to half of the beam height. Prior to beam reinforcement, the fabrics were cut into strips of 8 in x 1 in (203 mm x 25 mm). Two layers of CFRP sheets were applied on all 42 specimens to evaluate the possibility of failure not only between the concrete and the fibers, but also in between the sheets. Prior to CFRP reinforcement, the surface was prepared using sandblasting and air blasting procedures, as described in Section 3.4.2.

Figure 3.9. Concrete beams reinforced with CFRP

For the purposes of testing, the loading rate was set at 0.01 in/min (0.25 mm/min) in order to follow the study by Gartner, Douglas, Dolan and Hamilton (2011). Their research indicated that this loading rate allowed for failure 1-2 min after half capacity was achieved. The main parameters recorded were peak load and cross head position. Peak load, as well as the geometrical properties of the specimens allowed for calculation of the bond shear strength. This is the stress developed on the CFRP-concrete interface during loading. Since the saw cut is equivalent to half the depth of the beam, compression during loading was shown on the upper half of the beam. Assuming a linear stress distribution in the concrete, the resultant of this compression stress distribution is located at one-sixth of the total beam depth, which leaves a distance of five-sixths the depth of the beam between the compression and the tension resultant at the level of the FRP, or 5h/6. The tensile force T was found to be 3PL/5h, which is then divided by the CFRP area to determine the bond shear strength.

Equation 3.1 demonstrates the derivation of the bond shear strength (τ), derivation obtained from Gartner et. al. (2011):

Equation 3.1

$$T = \frac{\frac{PL}{2}}{\frac{5h}{6}} = \frac{3PL}{5h}, \qquad so\ \tau = \frac{T}{wS} = \boldsymbol{\frac{3PL}{5hwS}}$$

Where:

τ = Bond shear strength (ksi)

T = Tensile force (kips)

P = Peak load (kips)

L = Span under testing = 12 in

h = Height of specimen = 4 in

w = Width of CFRP reinforcement = 1 in

S = Length of CFRP reinforcement = 8 in

A United Testing Machine Model SFM – 300 kN was used to conduct the flexural tests. The main parameters recorded by the testing machine were time, force, and crosshead position. In order to be able to place the beams on the machine properly for testing, a loading fixture was built. This fixture consisted of a 13.5 in x 6 in x 2 in (343 mm x 152 mm x 50 mm) steel plate with a 3 in x 2 in x 0.5 in (76 mm x 50 mm x 13 mm) steel plate welded at the bottom in the shape of a “T” to allow proper gripping in the machine. Two smooth bars were welded along the 6 in length which served as simple supports of the beam. These bars were separated by 12 in, equivalent to the span under analysis. Figure 3.10 shows the fixture in the machine, and the beam setup for testing.

Figure 3.10. Beam fixture and specimen

## 3.5 Test Results

### 3.5.1 Stage 0 Cylinder Tests

A total of four cylinders were tested in compression during Stage 0. Cylinders were of the standard size of 6 in x 12 in (15.2 cm x 30.4 cm). A high average strength of 6.26 ksi (43.16 MPa) was seen in this group. Table 3.4 shows the individual and average values for compressive load and compressive strength. Figure 3.11 shows the characteristic failure corresponding to the four specimens.

Table 3.4. Cylinder tests for Stage 0

| Cylinder | Load | | Compressive Strength | |
|---|---|---|---|---|
| | kip | kN | ksi | MPa |
| 1 | 180 | 801 | 6.37 | 43.89 |
| 2 | 178 | 792 | 6.30 | 43.41 |
| 3 | 176 | 783 | 6.22 | 42.92 |
| 4 | 174 | 774 | 6.15 | 42.43 |
| **Average** | **177** | **787** | **6.26** | **43.16** |

Figure 3.11. Characteristic failure for compression tests

### 3.5.2 Stage 0 Pull-off Tests

A total of six pull-off tests were conducted for stage zero. Block #1, equivalent to test labels 1, 2, and 3 showed results of 374 psi (2.58 MPa), 442 psi (3.05 MPa), and 391 psi (2.70 MPa), respectively. Out of the three pull-off tests planned on Block #1, test 1 showed an adhesive failure between the dolly and the CFRP, labeled as failure Mode A by ASTM D7522/D7522M (2009). As stated in Table 2.1, this was not considered an acceptable failure mode, mainly because the CFRP-concrete bond did not show failure. Figure 3.12 below illustrates the adhesive failure mode from test 1.

Figure 3.12. Pull-off Mode A failure

As illustrated in Figure 3.13, the dollies from tests #2 and #3 show mostly concrete failure but with some black spots in which the CFRP did not adhere to the concrete during the reinforcement process. In the case of block #2, equivalent to tests 4, 5, and 6, all three showed a Mode A failure, with strengths of 444 psi (3.06 MPa), 270 psi (1.86 MPa), and 266 psi (1.83 MPa).

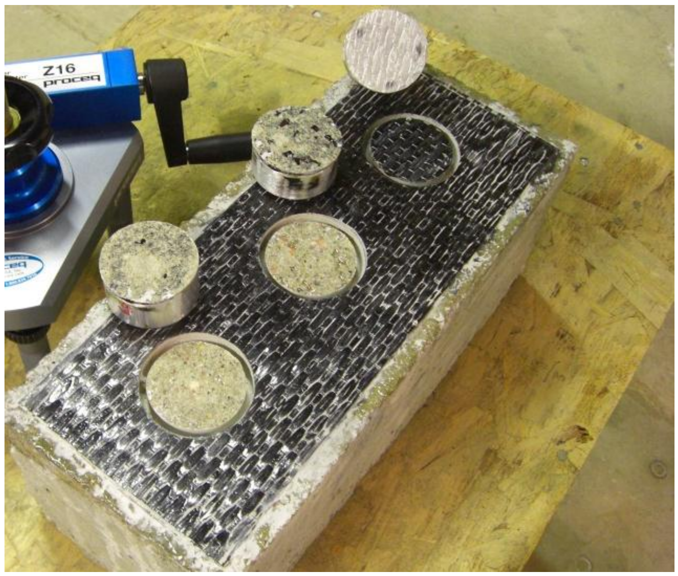


Figure 3.13. Pull-off Tests 1, 2, and 3 failure modes

Out of the six pull-off tests performed, four showed a fully adhesive failure. As a result, four additional core circles were drilled, two for each block, with the purpose of performing four more pull-off tests for statistical validation. Tests 7 and 8 were performed on block #1. The same procedure was followed in the adhesion, including the sanding and cleaning of the surface and dollies. The Devcon high strength epoxy was applied on the dollies for adhesion and allowed to cure for 24 hours before testing. Even though strength was shown to be 405 psi (2.79 MPa) and 401 psi (2.76 MPa), failure still occurred in the adhesive layer.

The persistence of the adhesive failure modes was attributed to two main reasons. Twisting of the dolly during adhesion could have caused minor air voids in the dolly-CFRP interface which decreased the suitability of the bond. The second possibility for adhesive failure could be the curing time of the Devcon epoxy. For all previous tests the epoxy was allowed to cure for 24 hours, and failure mode "A" was still showing. Therefore, tests 9 and 10 on block #2 were allowed to cure for 5 days to ensure that the epoxy was fully cured. For these last two tests the dollies were not twisted, they were slowly placed on the surface and uniform pressure was applied for 30-90 seconds to avoid slippage. As a result, tests 9 and 10 showed strengths of 433 psi (2.98 MPa) and 439 psi (3.03 MPa), respectively, with acceptable Mode F failures. Figures 3.14 and 3.15 illustrate the strengths and failure modes of all the pull-off tests performed during stage zero. The notations on Figures 3.14 and 3.15 represent the test number, the ASTM D7522/D7522M failure mode letter, and the strength in psi. These results are also summarized in Table 3.5.

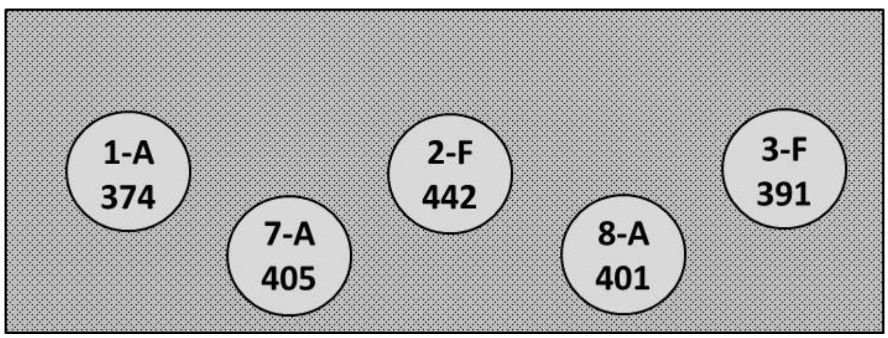


Figure 3.14. Block #1 Results

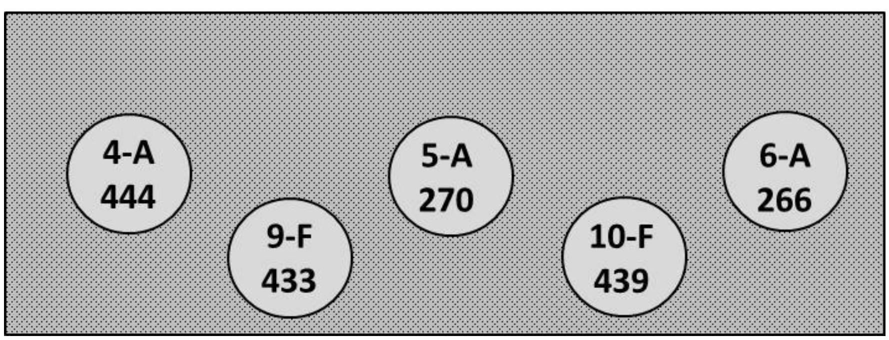


Figure 3.15. Block #2 Results

Table 3.5. Stage Zero Test Results

| Test Label | Pull-off Strength psi | Mpa | Failure Mode (ASTM D7522) |
|---|---|---|---|
| 1 | 2.58 | 374 | A |
| 2 | 3.05 | 442 | F |
| 3 | 2.70 | 391 | F |
| 4 | 3.06 | 444 | A |
| 5 | 1.86 | 270 | A |
| 6 | 1.83 | 266 | A |
| 7 | 2.79 | 405 | A |
| 8 | 2.76 | 401 | A |
| 9 | 2.98 | 433 | F |
| 10 | 3.03 | 439 | F |

### 3.5.3 Stage 0 Beam Tests

Stage 0 contained three small beams to be tested under three-point flexure. Due to an error with the testing machine, one specimen was prematurely damaged so the remaining two were used as the control specimens. As previously mentioned, a United Testing Machine Model SFM- 300 kN was used and the parameters recorded were time, force, and deflection as measured by the crosshead position. Figure 3.16 shows failure of one of the beams and how a single flexural crack was developed at the top of the saw cut, as desired.

Figure 3.16. Beam following crack failure

Both beams from this stage showed a fairly similar failure behavior, in which one side of the beam de-bonded before the other. However, the peak force witnessed on Beam 1 was considerably higher, 0.563 kips (2.50 kN) greater than for Beam 2. The larger force resisted by Beam 1 also resulted in a displacement of 0.029 in (0.74 mm) greater than for Beam 2. One possible reason for Beam 2 having a lower peak force is a weaker CFRP-concrete bond coming from the CFRP application. The darker spots on the failure surface for Beam 2 show that the CFRP did not completely adhere to the concrete substrate during application. Beam 1 contained little to no adhesive layer on top of the concrete surface after failure, which indicates that the CFRP was fully bonded to the concrete. Figure 3.17 shows the failure surfaces for both specimens and it is noticeable how Beam 2 contained a significantly higher amount of adhesive on the concrete.


Figure 3.17. Beam 1 (left) and Beam 2 (right) following testing

A Force vs. Displacement plot was created, labeled as Figure 3.18.

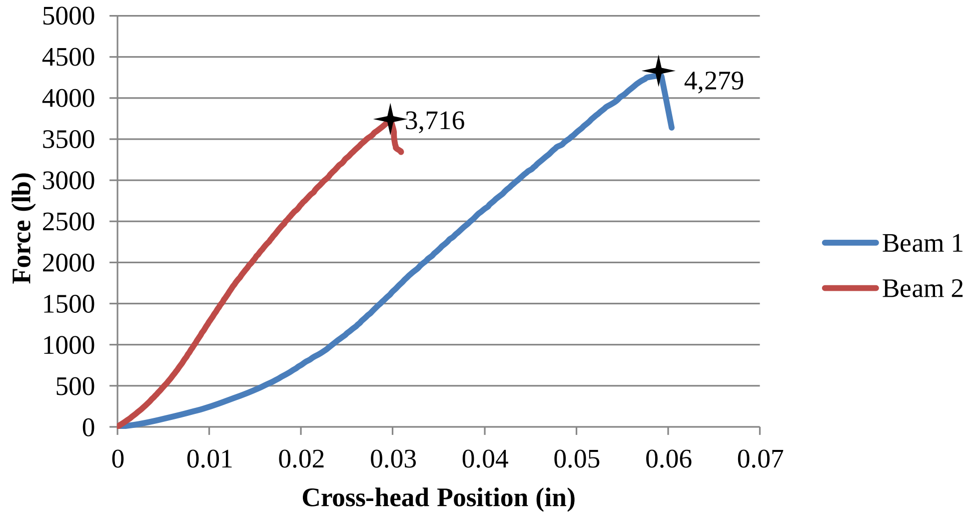


Figure 3. 18. Force vs. Displacement Graphs for Stage 0 Beams

With the parameters recorded by the testing machine and the geometric properties of the specimens, the bond shear stress was calculated using Equation 3.2. A sample calculation is provided below. All variables in the equation are fixed values for all specimens, except for the peak load. The peak load used in the following calculation is from Beam 1. Finally, Table 3.6 summarizes the results obtained from the Stage 0 specimens.

Equation 3.2

$$\tau = \frac{3PL}{5hwS} = \frac{(3)(4.28\ kips)(12\ in)}{(5)(4\ in)(1\ in)(8\ in)} = \mathbf{0.963\ ksi}$$

Table 3.6. Stage 0 Beam Results

| Beam | Maximum Load | | Bond Shear Stress | |
|---|---|---|---|---|
| | Kip | kN | ksi | MPa |
| 1 | 4.28 | 19.04 | 0.963 | 6.64 |
| 2 | 3.72 | 16.55 | 0.837 | 5.77 |

## 3.6 Stage 1 Results

### 3.6.1 Stage 1 Cylinder Tests

A total of four cylinders were tested under compression during this stage. With an average load of 155 kips (690 kN) and average strength of 5.49 ksi (37.83 MPa), these cylinders were weaker by approximately 12% than those tested 6 months prior. These lower values were unexpected and may have been due to calibration of the machine that was made prior to Stage 1 testing. Failure modes for the four specimens were similar to those from Stage 0 (Refer to Figure 3.11 in Section 3.5.1). See Table 3.7 for the summarized results of the four cylinders.

Table 3.7. Cylinder Tests for Stage 1

| Cylinder | Load | | Compressive Strength | |
|---|---|---|---|---|
| | kip | kN | ksi | Mpa |
| 1 | 161 | 716 | 5.69 | 39.26 |
| 2 | 151 | 672 | 5.34 | 36.82 |
| 3 | 151 | 672 | 5.34 | 36.82 |
| 4 | 158 | 701 | 5.57 | 38.41 |
| **Average** | **155** | **690** | **5.49** | **37.83** |

### 3.6.2 Stage 1 Pull-off Tests

Forty five pull-off tests were performed during this stage. Environmental exposures included water immersion, wet-dry (W/D) cycles in Chloride- based deicer (Apex), immersions in both chloride and non-chloride based deicers (Apex and Apogee), and freeze-thaw cycles on chloride-based deicer (Apex). All specimens that underwent conditioning were pulled out of their respective containers and left in dry conditions and room temperature for 5 days to allow for proper air drying before the adhesion of the dollies. In addition, a cloth was used every day for the 5-day period to help increase drying speed. Once fully dried, the specimens were prepared as described in Section 3.4.3, including the core drilling and adhesion of the dollies. Table 3.8 shows the results obtained from the forty five pull-off tests.

Table 3.8. Stage 1 Pull-off Test Results

| Block | Dolly | Exposure | CFRP Layers | Pull-Off Strength psi | Pull-Off Strength MPa | Average Strength psi | Average Strength Mpa | Failure Mode (ASTM D7522) |
|---|---|---|---|---|---|---|---|---|
| 1 | 1 | Dry | 2 | 226 | 1.56 | 172 | 1.19 | F |
| | 2 | | | 152 | 1.05 | | | F |
| | 3 | | | 139 | 0.96 | | | F |
| 2 | 4 | Immersion in Water | 2 | 528 | 3.64 | 372 | 2.57 | A |
| | 5 | | | 513 | 3.54 | | | A |
| | 6 | | | 408 | 2.81 | | | A |
| 3 | 7 | | | 308 | 2.12 | | | A |
| | 8 | | | 304 | 2.10 | | | A |
| | 9 | | | 173 | 1.19 | | | F |
| 4 | 10 | Wet-Dry in Chloride Deicer | 2 | 89 | 0.61 | 292 | 2.02 | F |
| | 11 | | | 132 | 0.91 | | | F |
| | 12 | | | 133 | 0.92 | | | F |
| 5 | 13 | | | 291 | 2.01 | | | F |
| | 14 | | | 579 | 3.99 | | | F |
| | 15 | | | 530 | 3.65 | | | F |
| 6 | 16 | Immersion in Non- Chloride Deicer | 2 | 179 | 1.23 | 450 | 3.10 | F |
| | 17 | | | 458 | 3.16 | | | F |
| | 18 | | | 432 | 2.98 | | | F |
| 7 | 19 | | | 579 | 3.99 | | | F |
| | 20 | | | 475 | 3.28 | | | F |
| | 21 | | | 575 | 3.96 | | | F |
| 8 | 22 | Immersion in Non- Chloride Deicer | 3 | 101 | 0.7 | 208 | 1.43 | F |
| | 23 | | | 403 | 2.78 | | | G |
| | 24 | | | 294 | 2.03 | | | F |
| 9 | 25 | | | 82 | 0.57 | | | F |
| | 26 | | | 142 | 0.98 | | | F |
| | 27 | | | 224 | 1.54 | | | F |
| 10 | 28 | Immersion in Chloride Deicer | 2 | 408 | 2.81 | 385 | 2.65 | F |
| | 29 | | | 405 | 2.79 | | | F |
| | 30 | | | 528 | 3.64 | | | A |
| 11 | 31 | | | 375 | 2.59 | | | A |
| | 32 | | | 355 | 2.45 | | | F |
| | 33 | | | 237 | 1.63 | | | F |
| 12 | 34 | Immersion in Chloride Deicer | 3 | 467 | 3.22 | 417 | 2.87 | F |
| | 35 | | | 627 | 4.32 | | | G |
| | 36 | | | 522 | 3.6 | | | F |
| 13 | 37 | | | 389 | 2.68 | | | F |
| | 38 | | | 80 | 0.55 | | | F |
| | 39 | | | defective | | | | - |
| 14 | 40 | Freeze-Thaw in Chloride Deicer | 2 | 313 | 2.16 | 327 | 2.25 | F |
| | 41 | | | 296 | 2.04 | | | A |
| | 42 | | | 287 | 1.98 | | | A |
| 15 | 43 | | | 422 | 2.91 | | | F |
| | 44 | | | 351 | 2.42 | | | F |
| | 45 | | | 291 | 2.01 | | | A |

The most common failure was identified by Mode F as labeled by ASTM D7522. Thirty three from the 45 dollies tested showed this type of failure, in which a partial adhesive failure of the FRP-concrete interface, combined with a partial concrete failure is seen. Figure 3.19 below shows a typical failure Mode F encountered during testing. The figure illustrates how the CFRP did not fully adhere to the concrete, and it is noticeable by the black spots of CFRP that are still visible, especially on the middle dolly.

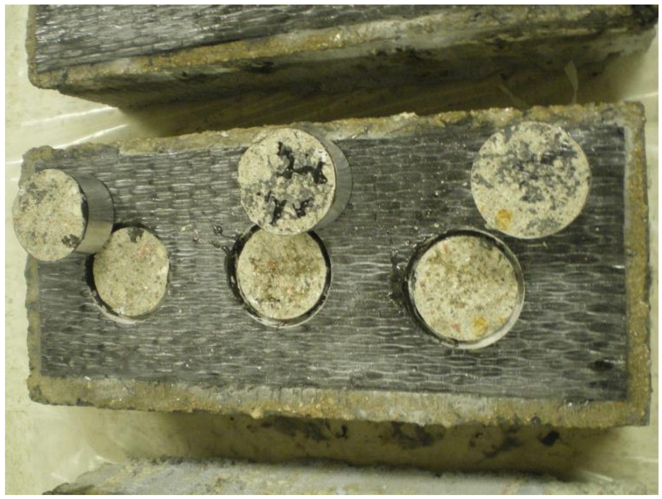

Figure 3.19. Mode F failures (ASTM D7522) during Stage 1 testing

The three pull-off tests corresponding to the dry block showed a representative Mode F failure, with forces ranging from 139 psi to 226 psi (0.96 MPa to 1.56 MPa). These results were used as control to be able to compare dry, room temperature environment with the remaining scenarios. In the case of the two blocks immersed in water for the 6-month period, 5 of the 6 pull-off tests resulted in a fully adhesive Mode A failure. These forces ranged from 173 psi to 528 psi (1.19 MPa to 3.64 MPa), with the lower limit being the only failure in this group characterized as Mode F. A potential reason for which failure Mode A was predominant on the water-exposed blocks could be due to continued curing of the concrete when it was placed in water for six months, making the specimens stronger and forcing the pucks to detach at the adhesive level. The two specimens that underwent wet-dry cycles were more consistent in their failure characteristics, as all six pull-off tests failed at a Mode F level. However, the pull-off force variation was quite large, with forces ranging from 89 psi up to 579 psi (0.61 MPa to 3.99 MPa) among the two blocks. Potential causes for these large discrepancies among the forces will be explained later on in this section.

The next specimens that underwent exposure were those immersed in a non-chloride based deicer. A total of four blocks were exposed, two reinforced with two layers of CFRP and two reinforced with three layers. Eleven of the twelve pull-off tests in this group showed a Mode F failure, with the remaining failure being identified as Mode G. However, in this case, the use of an extra CFRP layer did not demonstrate any improvements to the bond strength. In fact, the average pull-off strength for the six pull-offs performed on the double- CFRP layer specimens was 450 psi (3.10 MPa), which turned out to be 242 psi higher than the average pull-off strength seen on the two blocks reinforced with three CFRP layers.

The next group of specimens consisted of four blocks immersed in a chloride-based deicer. Similar to the previous group, this one consisted on two blocks reinforced with two layers of CFRP and two blocks strengthened with three layers, with a total of 12 pull-off tests. From these twelve tests, eight showed a Mode F failure, two showed a Mode A failure, one showed a full concrete failure Mode G, and the remaining one was categorized as defective due to thread malfunction of the dolly. The average pull-off strength for the double-layer specimens was 385 psi (2.65 MPa), this force being 32 psi (0.220 MPa) lower than those reinforced with three layers of composite.

The last group of specimens consisted of those exposed to lower temperatures. Freeze-thaw cycles were applied on specimens reinforced with two layers of CFRP. Two blocks, or six pull-off tests were conducted in this group. Three pull-offs showed a Mode F failure, and the remaining three failed at the adhesive layer. Forces ranged from 287 psi (1.98 MPa) to 422 psi (2.91 MPa), with an average bond strength of 328 psi among the six pull-offs. ASTM D7522 does not consider failure Mode A as an acceptable mode. For this reason, if these adhesive failures are not taken into consideration, the average strength increases to 365 psi among the three dollies that failed partly at the concrete and partly at the FRP-concrete interface.

There are large variances in the results from the pull-off tests. A special concern is the fact that the control specimens showed the lowest strengths in relation to the other groups. Exact reasons why strengths can vary to this magnitude among specimens that underwent similar conditioning are unknown, but it gives an idea of the extremely localized behavior that pull-off tests can exhibit.

While firm conclusions are difficult, listed below are several potential reasons why such discrepancies in the results would be created:

- Inconsistencies in the depth of the core drilling prior to puck adhesion. The recommended depth per ASTM D7522 is 0.25 in. (6 mm) to 0.50 in. (12 mm). A core drill depth of 0.50 in. could present much different results than a core that is 0.25 in. deep.
- Varying volumes of epoxy used per dolly. Since the dollies are manually adhered onto the surface one by one, a slight difference in the volume of epoxy used per dolly could potentially decrease precision of results.
- Irregularities on the surface of the specimen that would prevent a fully flat adhesion. If a surface is not completely flat, more epoxy would have to be used on the side that is not in contact with the dolly. This would lead to variations in thickness across a bond surface.
- Twisting of the dollies when adhering to the FRP surface. Such twisting during adhesion could create minor air voids and decrease adhesion performance. Therefore, a uniform pressure with no rotation of the dolly is recommended.
- Inconsistencies in the mixing of epoxy. Since the type of epoxy used is only workable for 5-7 minutes, and there were a large number of dollies that needed to be adhered, several mixes of epoxy had to be performed separately. As this is all done by hand, occasions in which an ideal

1:1 ratio of resin to hardener is not used, may decrease the performance of the epoxy and deviate results.

- Improper cleaning and sanding of the FRP surface and/or the aluminum dollies. Accumulation of dust or dirt, as well as a non-roughened, smooth surface would considerably decrease adhesion performance.

All of these sources of error, however, were carefully considered prior to preparation of the specimens for testing and significant effort was made to prevent them. Another cause that may have influenced the results has to do with the type of concrete. Since pull-offs are such a small scale testing procedure, having a strong concrete with large amounts of coarse aggregate can vary pull-off strengths within the same block.

Finally, throughout testing the forty five pull-offs, three special cases were encountered. A special note worth discussing in the chloride based deicer group with triple layer reinforcement is the large discrepancy encountered within the six pull-offs. Test #38 resulted in the absolute lowest value for the entire stage, with a strength of 80 psi (0.55 MPa). On the other hand, test #35 resulted in the absolute highest value among the 45 pull-off tests performed in this stage, with a strength of 627 psi (4.32 MPa). This particular pull-off was characterized as an ideal concrete failure, Mode G in ASTM D7522. It is believed that the reasons for the extremely large discrepancies among the specimens that underwent the exact same conditioning is due to the large aggregate present on test #35, as well as the presence of moisture within the concrete-FRP interface on test #38. This moisture was noticed because of the lighter color of the concrete, as well as a softer and more clayey feel to the touch. Notice in Figure 3.20 the large aggregate on Test #35, as well as the lighter color and thinner layer of concrete on test #38.

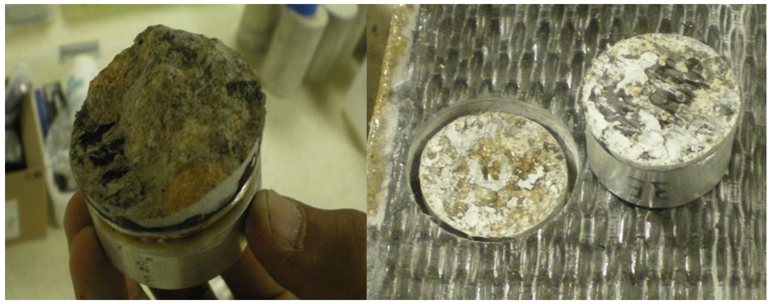

Figure 3.20. Test #35 (left) vs. Test #38 (right)

The second special case encountered during testing has some relation to test # 38 above. The presence of moisture in the CFRP-concrete interface on Blocks #4, #9, and #13. This moisture was not present at the time of fabrication of specimens. Therefore, the deicing solution that these blocks were exposed to actually infiltrated into the bond either through the FRP or the concrete. This moisture was noticed in the specimens with different tonalities of gray in the concrete, as well as a softer, more clayey feel to the touch. It was determined that such moisture did decrease strength considerably. In the case of block #4 corresponding to wet-dry cycles, dollies #10, #11 and #12 showed moisture and did result in the ones with weaker strengths out of the whole group. In addition, dolly # 16 from the non-chloride based deicer group showed a strength of 271 psi (1.868 MPa) lower than the remaining dollies in that group, and it was the only pull-off that exhibited levels of moisture within that group.

The same moisture level – strength relationship was witnessed in dollies #25, #26, and #27 (Block 9) within the non-chloride based deicer group, as well as dollies #33 from Block 11, and dollies #37 and #38 from Block 13. With these patterns identified, one can draw the conclusion that the presence of moisture does in fact reduce the pull-strength. However, reasons why some dollies within the same group may or may not have exhibited moisture are unknown. A characteristic example of a dolly showing sign of moisture is illustrated in Figure 3.21.

Figure 3.21. Presence of moisture at CFRP-concrete interface

The last special case consisted of having a mixed failure mode between Mode A and Mode F. In this special case, the CFRP fibers may not have been fully saturated during application, causing separation of the bundles. Figure 3.22 shows how half of the dolly completely detached from the concrete at an adhesive level on the dolly-FRP interface, whereas the second half was seen as an adhesive failure on the concrete-FRP interface.

Figure 3.22. Combined Mode A and Mode F failure

### 3.6.3 Stage 1 Beam Tests

Following completion of the pull-off tests, three-point flexural tests on thirteen beams were conducted in this stage. Due to improper handling of testing machine, Beam # 7 corresponding to water exposure was characterized as defective. Peak forces ranged from 1.562 kips (6.95 kN) to 4.242 kips (18.87 kN). With the maximum load values and the geometrical properties of the beams, Equation 3.2 was used to calculate the bond shear stress in a similar fashion as described in Section 3.5.3. Results are summarized in Table 3.9.

Table 3.9. Stage 1 Beam Results

| **Beam** | **Exposure** | **Maximum Load** | | **Bond Shear Stress** | |
|---|---|---|---|---|---|
| | | **Kip** | **kN** | **ksi** | **Mpa** |
| 1 | Dry | 4.242 | 18.87 | 0.95 | 6.58 |
| 2 | | 3.843 | 17.09 | 0.86 | 5.96 |
| 3 | W/D in chloride deicer | 3.799 | 16.90 | 0.85 | 5.89 |
| 4 | | 3.345 | 14.88 | 0.75 | 5.19 |
| 5 | Water | 3.696 | 16.44 | 0.83 | 5.73 |
| 6 | | 2.057 | 9.15 | 0.46 | 3.19 |
| 7 | | Defective | - | - | - |
| 8 | Non-chloride deicer | 1.562 | 6.95 | 0.35 | 2.42 |
| 9 | | 3.644 | 16.21 | 0.82 | 5.65 |
| 10 | | 3.203 | 14.25 | 0.72 | 4.97 |
| 11 | Chloride deicer | 3.353 | 14.91 | 0.75 | 5.20 |
| 12 | | 1.718 | 7.64 | 0.39 | 2.67 |
| 13 | | 2.586 | 11.50 | 0.58 | 4.01 |

Two control specimens were kept at dry conditions and room temperature. From the thirteen beams tested, these two possessed the highest peak load values at 4.242 kip (18.87 kN) and 3.843 kips (17.09 kN). The absolute lowest load experienced on the beams was identified on Beam #8, corresponding to immersion in non-chloride based deicer. Figure 3.23 below clearly demonstrates the major differences in the bond between one of the control specimens (Beam #1) and Beam #8. The larger area of darker spots on the concrete surface where the CFRP failed was found noticeable. The dark spots correspond to the cured epoxy adhesive that did not fully bond with the CFRP strip. Although it is possible that the non-chloride deicer may have contributed to degradation of the bond, the other potential cause for this behavior is improper adhesion of the CFRP onto the concrete during preparation.

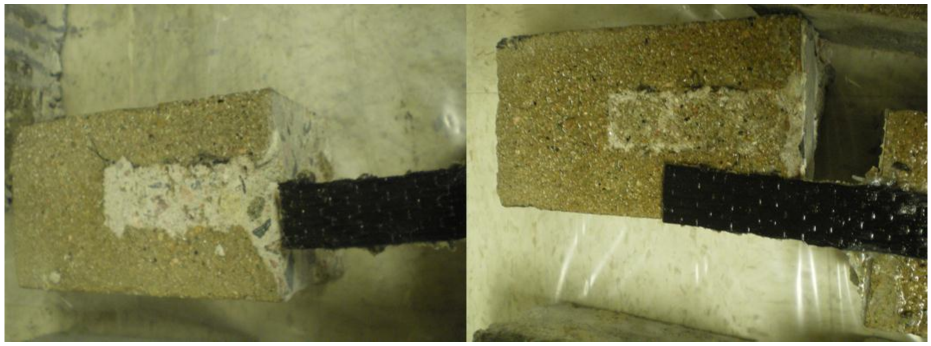

Figure 3.23. Beam #1 (left) vs. Beam #8 (right) failure

Table 3.10 shows the results summarized by exposure. The wet-dry exposure was identified as the group of beams that degraded the least in terms peak load values, with a load of 88% of the control beams. On the other hand, the chloride-based deicer was found to be the exposure that corresponded to the highest strength degradation, with the peak load being 63% of the control specimens.

Table 3.10. Stage 1 Average Results for Beams

| | Average Peak Load | | Average Bond shear Stress | | % of |
|---|---|---|---|---|---|
| **Exposure** | **Kip** | **kN** | **ksi** | **Mpa** | **Control** |
| Dry | 4.043 | 17.98 | 0.910 | 6.271 | 100.0% |
| W/D in chlorde deicer | 3.572 | 15.89 | 0.804 | 5.541 | 88.4% |
| Water | 2.876 | 12.79 | 0.647 | 4.462 | 71.2% |
| Non-chlorde deicer | 2.803 | 12.47 | 0.631 | 4.348 | 69.3% |
| Chloride deicer | 2.552 | 11.35 | 0.574 | 3.959 | 63.1% |

## 3.7. Stage 2 Results

### 3.7.1 Stage 2 Cylinder Tests

A total of four cylinders were tested in compression during Stage 2. The average strength for this group was 5.14 ksi (35.44 MPa). These cylinders turned out to be 6.5% weaker than those tested six months prior, and 18% than the ones tested twelve months prior. The failure mode, however, was similar to those tested in the previous stages. (Refer to Figure 3.11 in Section 3.5.1). Table 3.11 summarizes the results for these cylinders.

Table 3.11. Cylinder Tests for Stage 2

| **Cylinder** | **Load** | | **Compressive Strength** | |
|---|---|---|---|---|
| | **kip** | **kN** | **ksi** | **Mpa** |
| 1 | 147.5 | 656 | 5.22 | 35.97 |
| 2 | 137.5 | 612 | 4.86 | 33.53 |
| 3 | 141 | 627 | 4.99 | 34.38 |
| 4 | 155 | 689 | 5.48 | 37.80 |
| **Average** | **145** | **646** | **5.14** | **35.42** |

### 3.7.2 Stage 2 Pull-off Tests

A total of forty five pull-off tests were conducted during Stage 2. The specimens were subject to the same environmental conditioning as Stage 1. One block was left at room temperature to be used as control. For this stage, all specimens undergoing conditioning were pulled out of the containers seven days prior testing. Specimens were again prepared following procedures explained in Section 3.4.3. Table X shows the results for the forty five pull-offs, including their average strength per group, and failure mode per ASTM D7522.

Table 3.12. Stage 2 Pull-off Test Results

| Block | Dolly | Exposure | CFRP Layers | Pull-Off Strength psi | Pull-Off Strength MPa | Average Strength psi | Average Strength Mpa | Failure Mode (ASTM D7522) |
|---|---|---|---|---|---|---|---|---|
| | 1 | | | 245 | 1.69 | | | F |
| 1 | 2 | Dry | 2 | 249 | 1.72 | 286 | 1.97 | F |
| | 3 | | | 365 | 2.52 | | | F |
| | 4 | | | 382 | 2.63 | | | A |
| 2 | 5 | | | 353 | 2.43 | | | A |
| | 6 | Immersion in Water | 2 | 367 | 2.53 | 356 | 2.45 | A |
| | 7 | | | 209 | 1.44 | | | F |
| 3 | 8 | | | 399 | 2.75 | | | F |
| | 9 | | | 425 | 2.93 | | | F |
| | 10 | | | 192 | 1.32 | | | F |
| 4 | 11 | Wet-Dry in Chloride Deicer | | 152 | 1.05 | | | F |
| | 12 | | 2 | 334 | 2.30 | 222 | 1.53 | F |
| | 13 | | | 275 | 1.90 | | | F |
| 5 | 14 | | | 313 | 2.16 | | | F |
| | 15 | | | 66 | 0.46 | | | F |
| | 16 | | | 340 | 2.34 | | | A |
| 6 | 17 | Immersion in Non- Chloride Deicer | | 308 | 2.12 | | | A |
| | 18 | | 2 | 311 | 2.14 | 357 | 2.46 | F |
| | 19 | | | 372 | 2.56 | | | F |
| 7 | 20 | | | 425 | 2.93 | | | G |
| | 21 | | | 386 | 2.66 | | | F |
| | 22 | | | 330 | 2.28 | | | F |
| 8 | 23 | Immersion in Non- Chloride Deicer | | 310 | 2.14 | | | F |
| | 24 | | 3 | 306 | 2.11 | 260 | 1.79 | A |
| | 25 | | | 131 | 0.90 | | | F |
| 9 | 26 | | | 239 | 1.65 | | | F |
| | 27 | | | 241 | 1.66 | | | F |
| | 28 | | | 95 | 0.66 | | | F |
| 10 | 29 | | | 154 | 1.06 | | | F |
| | 30 | Immersion in Chloride Deicer | 2 | 329 | 2.27 | 203 | 1.40 | F |
| | 31 | | | 289 | 1.99 | | | A |
| 11 | 32 | | | 163 | 1.12 | | | F |
| | 33 | | | 190 | 1.31 | | | F |
| | 34 | | | 139 | 0.96 | | | F |
| 12 | 35 | | | 329 | 2.27 | | | F |
| | 36 | Immersion in Chloride Deicer | 3 | 270 | 1.86 | 257 | 1.77 | F |
| | 37 | | | 281 | 1.94 | | | F |
| 13 | 38 | | | 273 | 1.88 | | | A |
| | 39 | | | 251 | 1.73 | | | F |
| | 40 | | | 330 | 2.28 | | | F |
| 14 | 41 | Freeze-Thaw in Chloride Deicer | | 283 | 1.95 | | | F |
| | 42 | | 2 | 348 | 2.40 | 352 | 2.42 | A |
| | 43 | | | 344 | 2.37 | | | A |
| 15 | 44 | | | 311 | 2.14 | | | A |
| | 45 | | | 494 | 3.41 | | | A |

Once again, the most common failure type corresponds to Mode F. Thirty two of the forty five pull-off tests resulted in this failure mode, equivalent to 71% of the total. Figure 3.24 below shows a representative failure for these thirty two dollies. The black spots on the dolly correspond to sections where the CFRP did not fully adhere to the concrete, followed by partial concrete detachment within the same dolly.

Figure 3.24. Representative Mode F failure for Stage 2

All three pull-offs conducted on the control specimen were Mode F failures. With strength values ranging from 245psi to 365 psi (1.69 MPa to 2.52MPa), the average strength for this group was 286 psi (1.97 MPa). When comparing these tests to the rest of the groups within Stage 2, the control specimens were not the highest in strength but were not the weakest either. The specimens that were immersed in water for twelve months had pull-off forces approximately 24.5% higher than the control specimen. The forces ranged from 209 psi to 425 psi (1.44 MPa to 2.93 MPa), with an average of 356 psi (2.45 MPa). From the six pull-offs conducted in this group, half were Mode A and half were Mode F. The next group of specimens was subjected to wet-dry cycles. These two blocks contained higher variances in the results, with forces ranging from 66 psi to 334 psi (0.46 MPa to 2.30 MPa). The average strength in this group was 222 psi (1.53 MPa). However, there was consistency as far as failure modes go, where all six dollies exhibited a Mode F failure.

Four blocks were exposed to a non-chloride based deicer, equivalent to a total of twelve pull-off tests. Two of these blocks were reinforced with two layers of CFRP, and the remaining two blocks with three layers of CFRP. Failure modes varied from Mode A in three pull-offs, Mode F in eight dollies, and the last one corresponded to an ideal Mode G failure. Figure 3.25 shows the Mode G failure that was witnessed on dolly #20 from this stage, corresponding to one of the blocks reinforced with two layers of CFRP.

Figure 3.25. Mode G Failure in Dolly #20

One observation worth mentioning within this group is the fact that the specimens reinforced with three layers of CFRP turned out to be weaker than those reinforced with two layers by a difference of 37.3%. The average strength for the double-layer reinforced blocks was 357 psi (2.46 MPa), compared to the average strength of 260 psi (1.79 MPa) in the ones reinforced with three layers of CFRP. Reasons for this are unknown, but it gives an idea of the further discrepancies in results that may be found when conducting pull-offs.

The next specimens are those exposed to a chloride based deicer. In this group, forces ranged from 95 psi to 329 psi (0.66 MPa to 2.27 MPa) in those with double-layer reinforcement, and from 139 psi to 329 psi (0.96 MPa to 2.27 MPa) in those with triple-layer FRP reinforcement. This group showed a more logical behavior as far as average strengths, since there was a 26.7%

increase in strength when adding an extra layer of FRP. However, the high variances among individual results make it difficult to draw solid conclusions as to what the values actually mean. As far as failure modes go, ten of the twelve showed a Mode F failure and the remaining two were adhesive Mode A failures. The next and final group of specimens corresponded to two blocks that underwent freeze-thaw cycles in a chloride-based deicer for twelve months. In this case, forces ranged from 283 psi to 494 psi (1.95 MPa to 3.41 MPa). Failure modes, however, were controlled by Mode A. Out of the six pull-offs, only two showed a Mode F failure and the remaining detached at the adhesive level.

In this stage, the presence of moisture in some of the dollies was also witnessed. Two of the dollies within the water exposure group, three within the wet-dry cycle group, three within the non-chloride based deicer group, and nine within the chloride-based deicer group, were the specimens that showed moisture between the concrete and FRP. Once again, the different tonalities of gray as well as a softer more clayey feel to the touch helped identify which dollies showed levels of moisture. The amount of moisture ranged from very small spots around the edges, to larger areas within the dolly. Figure 3.26 shows a characteristic image of the presence of moisture in the specimens. As far as moisture – strength relationship in the water group, no logical pattern was found. In fact, dollies #8 and #9 resulted in the highest bond strength in the group, even though these were the ones that showed some moisture within the group. However, dollies #25, #26, and #27 did turn out to be weakest ones within the non – chloride based deicer group. For the chloride based deicer group, the moisture-strength relationship makes sense in the double-layer reinforcement ones, where dollies #28, #29, #32, and #33 showed the lowest strengths as oppose to the drier ones. These patterns, however, are difficult to interpret, as the moisture was not present within individual blocks, but rather within individual dollies.

Figure 3.26. Presence of moisture on Dollies

Finally, the special case in which a combined Mode A and Mode G failure was witnessed in dolly # 31 from Stage 1, was also seen in Stage 2 in dolly #42. Figure 3.27 shows dolly #42 with this particular failure. The cause may have been once again due to improper adhesion of the fabric, causing separation of the bundles, or due to a weak pattern of concrete on one side.

Figure 3.27. Combined Mode A and Mode F failure in Dolly #42

### 3.7.3 Stage 2 Beam Tests

Thirteen beams were tested to failure in this stage. The beams were classified in the same manner as those from Stage 1. Failure loads ranged from 2.104 kips (9.36 kN) to 4.181 kips (18.60 kN). On average, the control specimens showed the highest strength, as expected. All beams showed a similar failure mode, wherein a single flexural crack is started at the top of the saw cut and shear failure debonding on one of the FRP-concrete interfaces is witnessed. Refer to Figure 3.16 in Section 3.5.3. The lowest strength corresponded to beam #4, in the wet-dry group. It is unknown if that specific environmental exposure might have been the main cause of strength degradation. However, it was observed that those beams exhibiting lower strengths were the ones that showed more adhesive marks on the concrete, meaning the FRP strip did not fully adhere to the surface. Figure 3.28 illustrates the significant difference there is between a proper bond and an improper one. This clearly had an effect in strength, especially in beams #4, #5, #6, and #12. Table 3.13 shows the peak loads with respect to each environmental conditioning. Table 3.13 also shows the values for the bond shear stress, calculated using Equation 3.1.

Table 3.13. Stage 2 Beam Results

| Beam | Exposure | Maximum Load | | Bond Shear Stress | |
|---|---|---|---|---|---|
| | | Kip | kN | ksi | Mpa |
| 1 | Dry | 4.181 | 18.60 | 0.94 | 4.18 |
| 2 | | 3.656 | 16.26 | 0.82 | 3.66 |
| 3 | W/D in chloride deicer | 3.704 | 16.48 | 0.83 | 3.71 |
| 4 | | 2.104 | 9.36 | 0.47 | 2.11 |
| 5 | Water | 2.928 | 13.02 | 0.66 | 2.93 |
| 6 | | 2.295 | 10.21 | 0.52 | 2.30 |
| 7 | | 4.317 | 19.20 | 0.97 | 4.32 |
| 8 | Non-chloride deicer | 3.895 | 17.33 | 0.88 | 3.90 |
| 9 | | 3.780 | 16.81 | 0.85 | 3.78 |
| 10 | | 3.927 | 17.47 | 0.88 | 3.93 |
| 11 | Chloride deicer | 3.640 | 16.19 | 0.82 | 3.64 |
| 12 | | 2.303 | 10.24 | 0.52 | 2.30 |
| 13 | | 4.106 | 18.26 | 0.92 | 4.11 |

Note: W/D = Wet-dry cycles

Figure 3.28. Beam #6 (left) vs. Beam #1 (right)

As far as degradation in relation to the control beams, no significant pattern was encountered compared to the results from Stage 1. In this stage, the beams that underwent wet-dry cycles turned out to be the weakest, whereas in Stage 1 these turned out to be the second strongest, right after the control beams. As a result, it is difficult to draw comparative conclusions among stages, as no clear pattern was seen in relation to strength degradation. See Table 3.14 for a summary of results and the percentage of strength relatively to the dry beams.

Table 3.14. Stage 2 Beam Average Results

| | **Average Peak Load** | | **Average Bond shear Stress** | | **% of** |
|---|---|---|---|---|---|
| **Exposure** | **Kip** | **kN** | **ksi** | **Mpa** | **Control** |
| Dry | 3.919 | 17.43 | 0.882 | 6.079 | 100.0% |
| W/D in chlorde deicer | 2.904 | 12.92 | 0.653 | 4.505 | 74.1% |
| Water | 3.180 | 14.15 | 0.716 | 4.933 | 81.2% |
| Non-chlorde deicer | 3.867 | 17.20 | 0.870 | 5.999 | 98.7% |
| Chloride deicer | 3.350 | 14.90 | 0.754 | 5.196 | 85.5% |

## 3.8 Durability

Figure 3.29 illustrates the average results from the 6-month and 12-month exposures for the pull-off tests, classified per conditioning. For the most part, there was a strength degradation observed in the specimens exposed for an additional six months. The only exceptions were in the groups of freeze-thaw, and the non-chloride specimens with three FRP layers. Also, there was a decrease in strength in the control specimens between 0 and 6 months, but it increased at the end of the twelve month period.

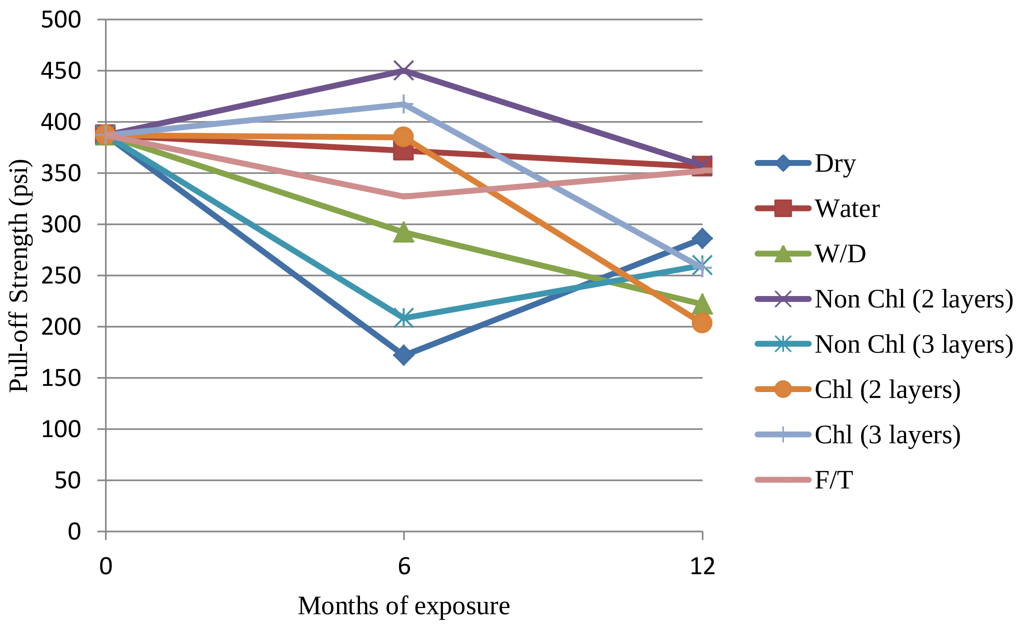


Figure 3.29. Average pull-off results for the 6-month and 12-month exposure

Figure 3.30 shows a comparison plot of the beams tested at all stages. As expected, the dry (control) beams were the strongest ones throughout the entire durability study. In addition, a decrease in strength was witnessed in the water group. However, for unknown reason there was an increase in strength during the last six months of exposure for the rest of the groups. This may have been due to an increase in the concrete strength during conditioning.

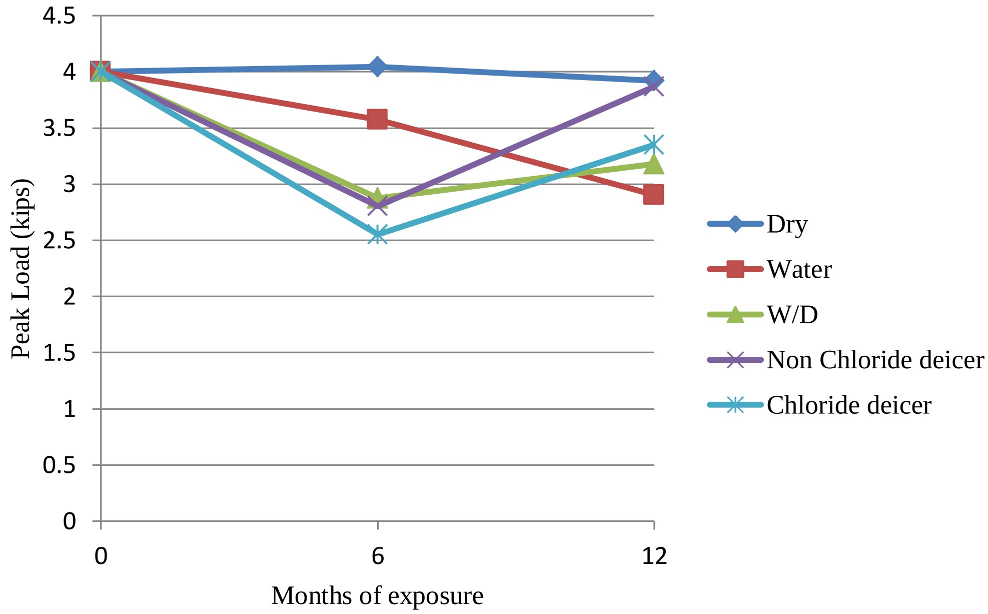


Figure 3.30. Average beam results for the 6-month and 12-month exposure

Due to these unexpected results it is impossible to draw any conclusions regarding the effect of the conditioning. From what was seen in all of the results previously discussed, high variability was witnessed, especially in the pull-offs. Factors such as an increase in strength over time, as well as inconsistencies in failure modes within the same specimens make interpretation of results challenging. It is difficult to know if the type of exposure has any influence in strength degradation. Therefore, the next chapter will be focused on evaluating pull-offs, by examining the challenges that may be encountered when analyzing the data obtained during testing, including the high variances pull-off test results can exhibit in the field and laboratory.

# Chapter 4. Evaluating Pull-Off Tests

## 4.1 Pull-off Tests Limitations Overview

Over the years, various tests have been created in the laboratory and the field in order to characterize the bond behavior. Pull off tests have been popular due to their low cost, small scale, and convenient method of testing bond. Although convenient, this test method does contain certain limitations affecting consistency and interpretation of results. For one, in a direct tension test, load is applied perpendicular to the surface. When the FRP repair is in service, perfectly perpendicular loads are never experienced by an FRP-concrete bond, meaning it can be difficult to understand what the test results actually mean in terms of structural performance. Also, due to the small scale of the testing procedure, drastic variations among results within the same test group can occur. The strength of the concrete substrate plays a large role in the bond strength that an FRP-concrete system can show. However, when bond strength is controlled by the strength of the pre-existing concrete, test results may not necessarily be indicative of the quality of the actual repair. In addition, preparation of the testing surface can introduce factors that may potentially increase variability of results, such as the presence of water as well as torsional and thermal stresses applied during the core drilling process. Finally, variations in the depth of the core cut must be paid close attention, as certain guidelines specify different depths, which could potentially alter the results. In light of these limitations, this chapter seeks to evaluate direct tension tests as a tool for understanding FRP-concrete bond in both the laboratory and the field. The chapter will consider different guidelines for conducting the test, and the results of previous studies in the lab and the field evaluating the effectiveness of pull-off testing.

## 4.2 Variations in the Depth of Cut

In addition to the specifications and procedures described by ASTM D7522, there are additional guidelines that focus on pull-off tests, and the depth of the core cut depths must be paid special attention. Guideline No. 03739 by the International Concrete Repair Institute (ICRI) also targets pull-off tests as a way to evaluate the tensile strength of a concrete surface repair. However, when looking at these different guidelines, the depth of the core cut was found to be inconsistent. The ICRI Technical Guideline No. 03739 (2004) titled Guide to Using In-Situ Tensile Pull-Off Tests to Evaluate Bond of Concrete Surface Materials, recommends a minimum depth of core drill to be 1 in (25 mm) or one-half the core diameter, whichever is larger. For a 2 in (50 mm) dolly, the core depth would be 1 in (25 mm). On the other hand, ASTM D7522 (2009) titled Standard Test Method for Pull-Off Strength for FRP Bonded to Concrete Substrate, recommends that for a 2 in (50 mm) dolly a core depth should be 0.25 in (6 mm) to 0.50 in (12 mm) in depth. Finally, ACI 503R (1993), titled Use of Epoxy Compounds with Concrete, recommends to barely core drill into the substrate. As a result, variations in these figures make it quite difficult to determine which depth would be the most appropriate for use in any field or laboratory setting.

## 4.3 Previous Laboratory Studies involving Direct Tension Pull-Off Tests

In this section, previous research laboratory studies regarding pull-off tests are examined. These past studies are analyzed to determine if the nature of their results provide any useful information for future research. Karbhari and Ghosh (2009) used pull-off tests to study the long term bond durability of CFRP adhered to concrete under various environmental conditions such as immersion in salt water, immersion in water, exposure to freezing conditions, and different humidity levels. A total of 250 pull-off tests were conducted, which were split among the various environmental exposures. The tests were conducted at 6 month intervals for a total of 24 months.

In general, results were fairly consistent, with a gradual increase in the level of deterioration for those specimens immersed for a longer period. They concluded that the specimens immersed in salt water exhibited the largest degree of deterioration, possibly due to infiltration of the sodium chloride into the CFRP-concrete interface.

A recent study conducted by Eveslage et. al. (2009) investigated the effect of variations in the use of ASTM D7522 as a standard pull-off test for FRP-concrete systems. The study included variables such as depth of core cut, shape of loading fixture or specimen, and the effects of retesting specimens that showed an unacceptable failure mode initially (Mode A per ASTM D7522). The experimental program involved a total of 75 pull-off tests. The specimens were prepared in accordance with instructions from the standard. For the specimens that exhibited a Mode A adhesive failure initially, it was determined that, even though the retests did show a Mode G failure, the average strengths were in fact lower, which indicated the possibility that damage to the specimens occurred during the initial testing. However, consistency in results from this group of specimens was witnessed, with a coefficient of variation of about 16%, similar to those that did not require retests. Three different cut depths were investigated: 0.10 in (2.5 mm), 0.25 in (6 mm), and 0.75 in (19 mm), with a total of 5, 21, and 5 pull-off tests conducted, respectively. From the test results, no change in strength was witnessed among the 0.10 in (2.5 mm) and 0.25 in (6 mm) core depths. However, the deeper core cut of 0.75 in (19 mm) showed a decrease in strength of up to 26%. A possible explanation for this lower strength is the likelihood of larger torsional and thermal stresses induced by drilling, as compared to the lower cuts. Table 4.1 shows statistics of pull off results for each cut depth.

Table 4.1. Pull-off Strength Results (Eveslage et. al., 2009)

| Depth of Cut, mm | Sample Size | Mean Bond Strength Mpa | psi | Standard Deviation MPa | psi | COV % |
|---|---|---|---|---|---|---|
| 0 | 31 | 2.72 | 395 | 0.141 | 20.5 | 20 |
| 2.5 | 5 | 2.78 | 403 | 0.094 | 13.6 | 13 |
| 6 | 21 | 2.78 | 403 | 0.110 | 15.9 | 16 |
| 19 | 5 | 2.06 | 299 | 0.125 | 18.2 | 22 |

As seen from Table 4.1 above, the results do not show a consistent pattern. There is no direct correlation between the depth of the cut and changes in strength. Also, the average coefficient of variation among these tests is nearly 18%. This leads to the conclusion that pull-off tests can provide certain inconsistencies in studies conducted in the laboratory.

## 4.4 Previous Field Studies involving Direct Tension Pull-Off Tests

Banthia, Abdolrahimzadeh, and Boulfiza (2009) conducted a field study in which four bridges in Canada were investigated to assess the durability of the FRP repairs applied on the bridges after several years of service. Four structures were selected to represent a range of environmental conditions, lengths of service, and types of FRP reinforcement. Table 6 summarizes some of the characteristics of these structures.

Table 4.2. Bridges Characteristics (Banthia, Abdolrahimzadeh, and Boulfiza, 2009)

| Structure | Location | Year of Construction | Year of FRP Repair | Type of FRP Repair |
|---|---|---|---|---|
| SafeBridge | Youbou, BC | 1955 | 2001 | Sprayed GFRP |
| St-Ètienne Bridge | Quebec | 1962 | 1996 | GFRP and CFRP column wraps |
| Leslie Street Bridge | Ontario | 1960s | 1996 | CFRP column wraps |
| Maryland Bridge | Manitoba | 1969 | 1999 | CFRP sheets at girder ends |

Pull-off tests were conducted on specific sections of these repairs in order to determine the condition of the bond. These tests were conducted following ASTM C1583-04, titled “Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)”. Similar to ASTM D7522, this standard is suitable for both laboratory and field tests and is used to determine the bond strength of the repair. The testing procedures are similar for both standards requiring core drilling, attachment of the dolly, and tensile load application until failure.

The locations of the pull-off tests on these bridges were randomly chosen, except in the case of the girders of the Maryland Bridge, where the cores were made at locations near the supports where maximum shear is witnessed. The depth of the cores was 0.40 in (10 mm) and diameter of the dollies used was 2 in (50 mm), as specified by ASTM C1583. Results from the pull-off tests showed significant variability. These results are summarized in Figure 4.1. The average pull-off bond strength for all four structures ranged from 104 psi (0.72 MPa) to 522 psi (3.60 MPa), both values obtained on different columns of the same bridge. For all structures the COV is very large, where values ranged from 27.7% for the Maryland Bridge, up to 154.2% for column 1 of the St-Etienne Bridge.

Interpretation of these results is challenging for several reasons. Failure modes were not specified in this study. Therefore, it is unknown what material controlled the bond strength; whether it was a concrete substrate failure or an FRP failure. In addition, the strength of concrete at the time of testing, which most likely varied among the different bridges, has significant influence on results and is important to interpret them. Finally, the lack of baseline or control values makes it difficult to understand whether low strengths represent poor application of the repair or degradation of bond strength over time. Figure 6 shows a plot of the values obtained for each structure.

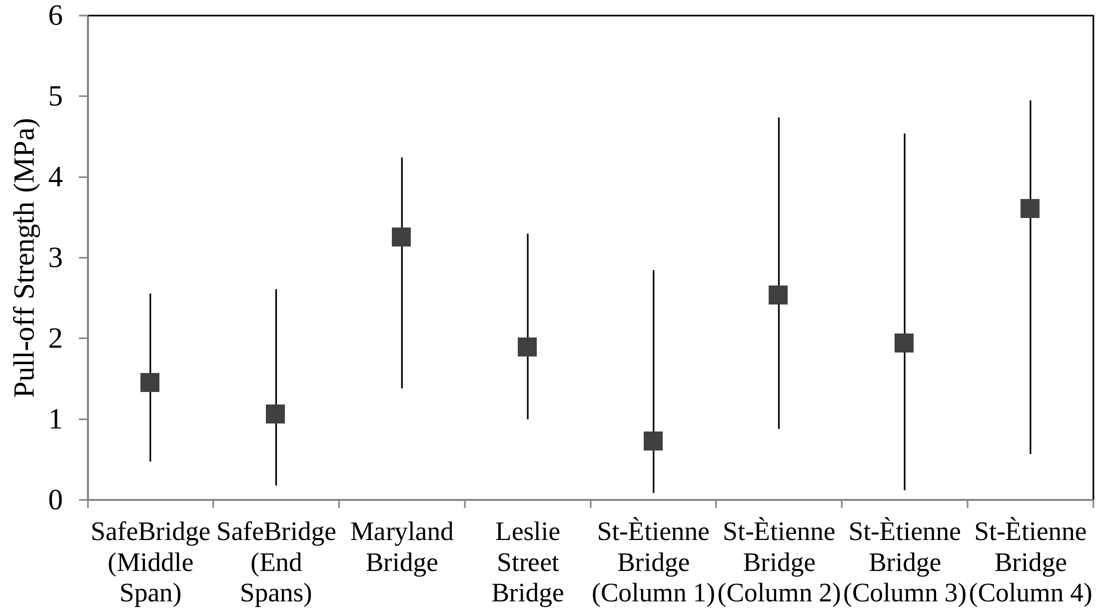


Figure 4.1. Pull-off Strength Results (Banthia, Abdolrahimzadeh, and Boulfiza, 2009). The ends of the vertical lines represent the lowest and highest values, and the boxes represent the mean values.

Another field study using pull-off tests to evaluate an FRP repair was completed in 2011 (Allen & Atadero, 2011) on a concrete arch bridge originally constructed in 1946. The structure was rehabilitated in 2003 by means of internal FRP rods, shotcrete, and external FRP wraps around both arches. At the time of repair, a total of 42 pull-off tests were conducted by a contracted testing firm at various sections of the structure using standard 2 in (50 mm) aluminum dollies. An additional 27 tests were conducted in 2011 to evaluate bond durability. Figure 4.2 shows a plan view of the arches, and the manner in which the bays were labeled. For instance, bay 1SW corresponds to the first bay on the Southwest side of the structure. The circle labels correspond to the locations where pull-off tests were conducted in 2003, while the triangles represent the locations from the 2011 study.

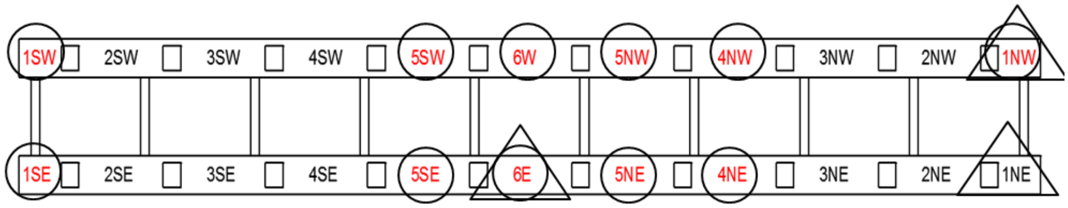


Figure 4.2. Pull-off test locations from 2003 study (circles) and 2011 study (triangles) (Allen, 2011)

The strongest pull-off values were located at the bottom of the arches, labeled as 1SE, 1SW, and 1NW. However, in the remaining eight locations where groups of pull-off tests were conducted, average strength values are relatively scattered along the bridge, showing no consistent pattern. The lack of consistency from a strength-location relationship standpoint among the inner bays and the lack of additional information, for example, location of shotcrete placement, make it difficult to draw reliable conclusions as to what the results represent. When considering the significance of failure modes, bay 1NW is an interesting case. Five of the six pull-offs showed a Mode A failure, unacceptable by ASTM D7522 for statistical validation. What is interesting about this group is that the only other failure was an ideal concrete failure (Mode G) which had a similar strength as the remaining Mode A failures. As Mode G failures are controlled by the concrete strength. This would suggest that the adhesive used was not strong enough for the strength of concrete for most of the tests. This is another difficulty that may be encountered in pull-off testing. Average results, as well as maximum and minimum values from the 2003 quality control tests are shown in Figure 4.3.

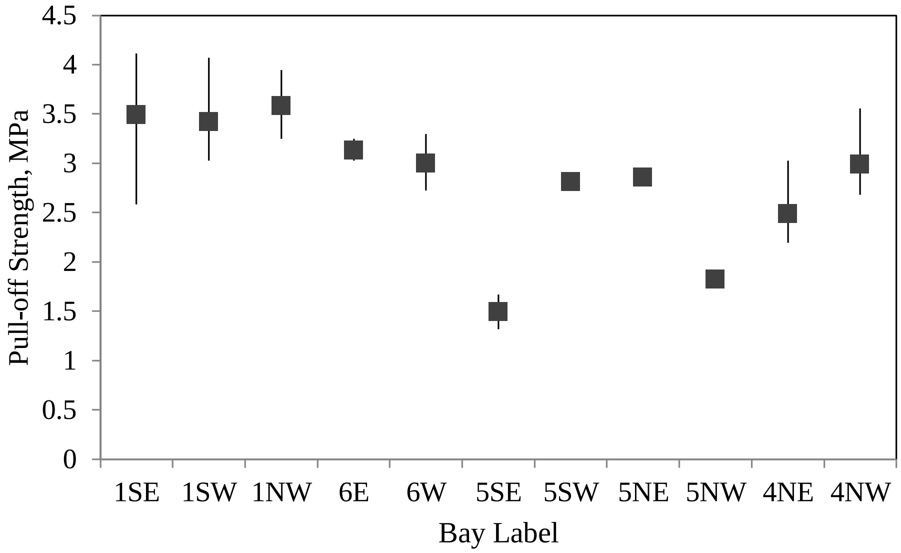


Figure 4.3. Castlewood Canyon Bridge pull-off strength results (CTC-Geotek, 2003)

The 27 pull-off tests conducted in 2011 were made in order to assess the durability of the repair, at three different locations of the bridge: 1NE, 1NW, and 6E, as indicated in Figure 4.4, Figure 4.5, and Figure 4.6, respectively. Fewer tests were conducted because in 2003 scaffolding for the repair was available, making many locations accessible, while in 2011 access to upper bays required climbing gear.

Results from these tests are shown in Figures 4.4, 4.5, and 4.6. Figure 4.6 shows an especially interesting case of pull-off strength variation. At this location, the sixth bay on the East arch, the two outer tests on the bottom now had strengths of 447 psi (3.08 MPa) and 363 psi (2.50 MPa) whereas the middle test, located just in between these two, showed an extremely low strength of 19 psi (0.13 MPa). What is worth noting is that all three pull-off tests showed a similar Mode G failure, which is characterized as the ideal substrate failure and should be in theory the highest in strength. In this case they seem to indicate a weak patch of concrete. This is once again another example of the challenge in figuring out why pull-offs behave the way they do, and the potential for high variances in results within the same test group. At the bottom of the arches, the predominant failure modes were Mode E and Mode F. For Bay 1NW, not including the last two pull-off tests, the average strength was low, approximately 133 psi (0.917 MPa). However, the last two pull-off tests showed an extreme increase in strength, averaging 252 psi (3.62 MPa). The remaining results were basically scattered, with failure modes varying from Mode A through Mode G, and large strength variation among adjacent tests. No consistent patterns were observed.

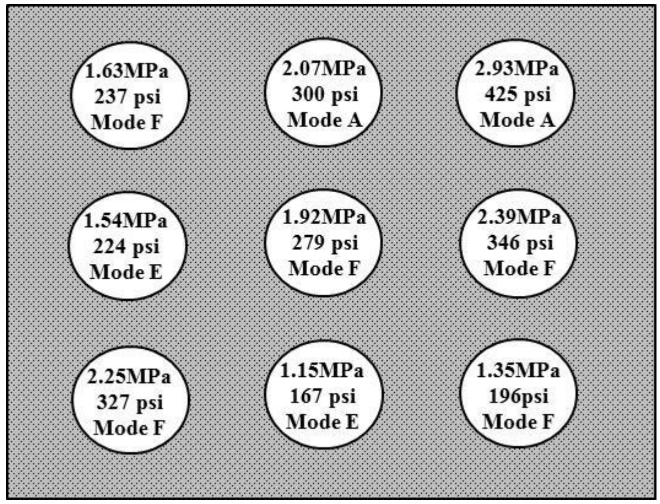


Figure 4.4. Pull-off test results, including strengths and failure modes for north end of east arch, labeled as 1NE. Dollies spaced approximately 6 in (15 cm) center-to-center from each other. (Allen, 2011)

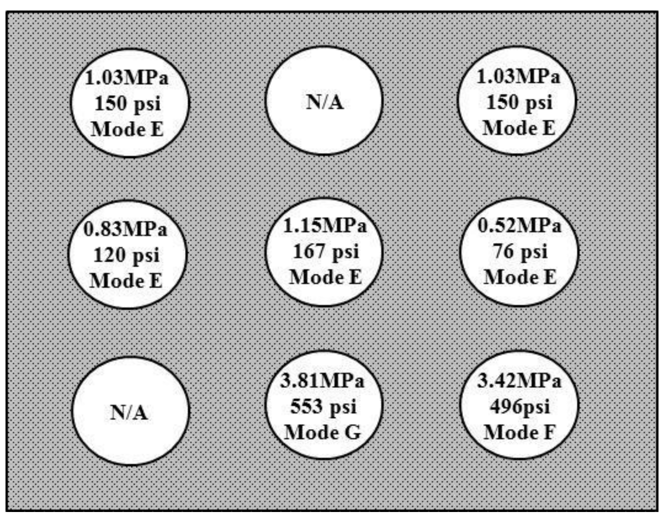


Figure 4.5. Pull-off test results, including strengths and failure modes for north end of west arch, labeled as 1NW. Dollies spaced approximately 6 in (15 cm) center-to-center from each other. (Allen, 2011)

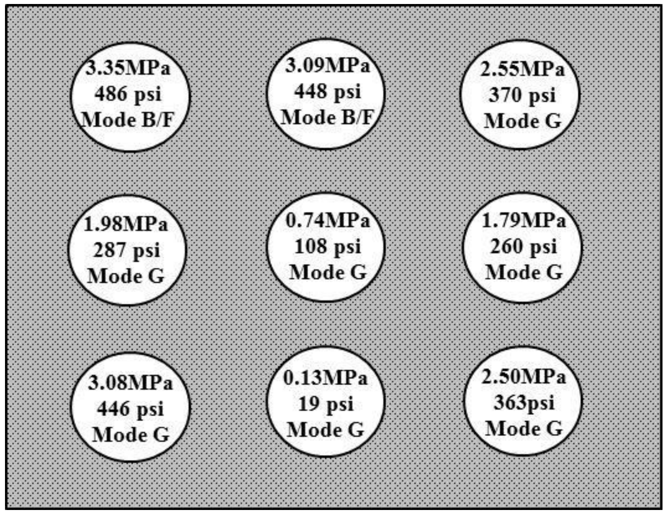


Figure 4.6. Pull-off test results, including strengths and failure modes for center of east arch, labeled as 6E. Dollies spaced approximately 10 in (25 cm) center-to-center from each other (Allen, 2011)

Another example involving pull-off tests in the field consisted of a recent quality control procedure focused on the evaluation of an FRP repair. The repair was made on the pier caps and columns of an Interstate bridge in Colorado. A total of seven pull-off tests were completed in 2011, using steel dollies with 3 in (76 mm) diameters, as opposed to the commonly used 2 in (50 mm) diameter aluminum dollies. Once again, large variations in the bond strength results were witnessed, as seen in Table 4.3. Strengths ranged from as low as 99 psi (0.683 MPa) to as high as 424 psi (2.92 MPa). The predominant failure mode was a cohesive concrete failure, also known as Mode 6, as labeled by the ICRI Technical Guideline No. 03739, equivalent to Mode G per ASTM D7522.

Table 4.3. Bond Strength Results (CTL Thompson Materials Engineers, inc., 2011)

| Location | Load | | Pull-off Strength | | Failure Mode | |
|---|---|---|---|---|---|---|
| | kN | lbs | MPa | psi | (ICRI No. 03739) | ASTM D7522 |
| Pier 4 West side Pier Cap North | 6.23 | 1400 | 1.37 | 198 | 6 | G |
| Pier 4 West side Column North | 9.34 | 2100 | 2.05 | 297 | 6 | G |
| Pier 6 East Side Pier Cap North | 3.11 | 700 | 0.68 | 99 | 5 | F |
| Pier 6 East Side Column South | 8.01 | 1800 | 1.75 | 254 | 6 | G |
| Pier 6 West Side Pier Cap North | 3.56 | 800 | 0.78 | 113 | 5 | F |
| Pier 5 East Side Pier Cap North | 9.34 | 2100 | 2.05 | 297 | 3 | C |
| Pier 5 East Side Pier Cap South | 13.34 | 3000 | 2.92 | 424 | 6 | G |

This example is limited by the small number of tests conducted, only one pull-off per bridge section, which makes statistical validation impossible. When conducting pull-off tests as a quality control procedure in the field, the tests are in fact destructive and repair of the surface is needed. Even though these tests are fairly simple to prepare, in the long run they can take time to complete if the amount of pull-offs becomes large. Therefore, pull-off tests do not become very practical if used as quality control.

As seen in this chapter, pull-off tests can exhibit high variability in results when conducted in the field and laboratory. It is uncertain how reliable these results can be and if there is any effective interpretation of the strength values and failure modes that come from conducting pull-off tests. As a result, part of the next chapter provides conclusions involving the nature of this test method and gives some recommendations in testing procedures if pull-off tests were to be developed in future research.

# Chapter 5. Conclusions

## 5.1 Summary

### 5.1.1 Durability Study

The purpose of this research was to evaluate the long-term behavior of the bond between Fiber-Reinforced Polymers (FRPs) and concrete when exposed to various environmental conditions, by means of a durability study conducted in the laboratory. Small concrete specimens were cast, reinforced with FRP sheets, and subjected to a series of exposures such as immersion in water, immersion in deicing agents, wet-dry cycles, and freeze-thaw cycles. These specimens were tested at different stages. Stage 0 represented the control specimens, while Stage 1 and Stage 2 represented the series of tests completed following six months and twelve months of conditioning, respectively. During these stages, pull-off tests were conducted on the concrete specimens strengthened with FRP. These tests were completed using the guidelines from ASTM D7522. In addition to pull-off tests, twenty-nine small concrete beams were fabricated, reinforced with FRP and tested in flexure. This method was recently introduced by Gartner et. al (2011), as it provides a more realistic scenario of the behavior of FRP reinforcement on actual concrete structural members.

Once testing was completed, results from both pull-off tests and beam tests were thoroughly discussed. No significant pattern was found when comparing the control specimens to the ones exposed for six months and twelve months. In fact, results were scattered among the test groups, including situations such an increase in strength over time, and varying failure modes from pull-off tests, which made it impossible to draw any firm conclusions in relation to the effect of these environmental exposures on the long term FRP-concrete bond.

### 5.1.2 Evaluation of Pull-Off Tests

After obtaining pull-off test results with high variability and which were difficult to interpret, this thesis also investigated the suitability of direct tension pull-off tests when examining the behavior of the bond, in the laboratory and the field. Previous studies were summarized with a focus on describing their different results, and providing a discussion of the nature and interpretation of these results. After analysis of previous lab and field studies, observations regarding reliability of results were made. The scattered results and lack of consistency make pull-off tests a questionable test method for use both in the laboratory and the field. In addition, variations in the depth of core cut prior to adhering the dollies can make a significant difference in the strength of each pull-off, and can potentially increase the variability in the failure modes. The different core depths specified by the pull-off documents such as ACI 503R (1993), ICRI No. 03739 (2004), ASTM D4541 (2009), and ASTM D7522 (2009), make it challenging to identify which in fact is the most appropriate guideline to follow.

### 5.1.3 Recommendations for Future Research

To further increase knowledge in this area, it is recommended that research studies in the future have a stronger focus in the effect of deicing agents on FRP materials bonded to concrete. Several past studies have mostly considered exposure to moisture, and sodium chloride, but have not taken into account other chemicals. Also, larger scale specimens are suggested. Larger beams would provide a better correlation between laboratory studies and actual field conditions, and could give more reliable data. In addition, studies involving both unidirectional and bidirectional carbon fibers is advised in order to increase knowledge in the way these materials provide the reinforcement.

The type of test chosen also plays a big role in the interpretation of results. Standard flexural tests are preferred, as these offer a more realistic behavior of an actual structural beam in a structure. Compression tests on concrete cylinders wrapped with FRP is also advised, as these will provide a better insight in the role of these materials used for confinement.

Pull-off tests are not recommended if conducting a study in the lab involving bond behavior of FRP-concrete systems. This test method contains certain limitations as far as interpretation of results, statistical validation, and. However, if pull-off tests were to be used in the lab, additional recommendations in the preparation of specimens and testing procedures are listed below. These recommendations will decrease the risk of obtaining results that are unacceptable for statistical validation.

- ASTM D7522 recommends at least five pull-offs per condition. It is recommended that more than five tests be used if variability among specimens becomes large. For control specimens, a larger quantity is recommended in order to have more baseline values to compare with other groups.
- If conducting a durability study, use concrete that has been curing for a significant amount of time, so that over time concrete strength gains do not play a role in pull-off strength changes
- Avoid high strength concrete containing large aggregate, as this may increase the possibility of obtaining more adhesive failure from the dolly-FRP interface.
- Conduct pull-off tests on concrete specimens not reinforced with FRP to have an idea of the tensile strength of that concrete and be able to compare to those reinforced with FRP.

## 5.2 Conclusion

This thesis was able to determine how laboratory studies on small scale concrete specimens strengthened with FRP contribute to developing technology used in increasing the durability of aging concrete infrastructure. Laboratory conditions by their definition are not identical to those seen in the field. However, the results of controlled environment testing helps raise important questions that need to be answered before the technology can move forward. Testing methods and result analysis can be developed in the laboratory in parallel with field tests.

As more results are gathered, it will strengthen then validity of our testing methods and allow for the type of analysis that will lead to hard conclusions.

As field and laboratory studies are conducted in the future, special attention needs to be paid to the durability of FRP-concrete systems. Additional questions regarding the influence of deicing agents on concrete and FRP need to be answered. More specifically, the level of degradation that these chemicals can cause in the long run has not yet been determined. Finally, questions regarding the pull-off test method and its suitability in determining the bond behavior of FRP-concrete systems needs further study. The findings of this thesis provide insight to future research related to studying the long-term performance of these systems. With this in mind, the use of FRP materials may continue to grow in the field of civil engineering as an option for the external reinforcement of transportation concrete structures.

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